Lecture Notes in Civil Engineering

Dharamveer Singh · Avijit Maji ·
Omkar Karmarkar · Monik Gupta ·
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Solomon Debbarma *Editors*

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Introduction

The practical problems associated with transportation systems engineering in many developing countries including India are different and complex. Therefore, researchers and agencies have been working to analyse the challenges and to identify implementable solutions for various transportation engineering related problems as per prevailing conditions. The earliest roots of the Conference on Transportation Planning and Implementation Methodologies for Developing Countries (TPMDC) can be traced back to the International Workshop/Conference series started by The Transportation Systems Engineering (TSE) group of IIT Bombay about 26 years ago by making the first announcement in the WCTRS (World Conference on Transport Research Society) Newsletter. The first Workshop of the series was organized in December 1994 on *Impact Evaluation and Analysis of Transportation Projects in Developing Countries*, IEATP-94. Riding on the success of the workshop, the second of the series, Transportation Planning, and Implementation Methodologies for Developing Countries (TPMDC-96) was conducted in December 1996. Starting from year 1996, international conference on TPMDC are conducted biennially. TPMDC 2022 brings an ideal platform for researchers, practitioners, and agencies to share and exchange the experience among the transportation professionals of the developing and developed nations.

TPMDC 2022

The 14th edition of TPMDC was held from 19th to 21st December 2022, at IIT Bombay, Mumbai. The conference covers the national and local level transport planning, traffic operation and management, pavement design, materials characterization, highway safety, geometric design, intelligent transportation systems and freight transport activities in India. The conference themes were designed to facilitate the contribution of research articles along the six following tracks.

Track I: Transport Modes: General Highways, Railways, Airways and Waterways

This theme includes general aspects of all transport modes including air transport, rail-based transport, inland and international waterways, and non-motorized transport modes. The theme covers research on various topics such as air traffic control and management; port, harbor, and fleeting services; railway system planning, design, operation, and management; freight transportation and operations using rail, water, and air modes. Research on infrastructure planning and geometric design aspects of transportation projects are also included in this theme.

Track II: Pavement Systems Engineering

This theme will focus on all aspects of material characterization, analysis, design, construction, evaluation, and maintenance of pavement structures. These aspects include broad areas of characterization of conventional and innovative pavement materials; pavement material modelling; pavement recycling; stabilization of pavement layers; analysis and design of bituminous, concrete, composite, and other kinds of pavements; life cycle cost analysis; pavement maintenance and management systems; pavement drainage; airport pavements and railway track.

Track III: Transportation Planning, Policy and Economics

This theme includes research on theoretical and empirical aspects of travel demand and behavioral modelling and forecasting, network design for passenger and freight transport, planning of urban transport systems, and logistics planning. The theme covers several aspects of transportation economics such as public transport pricing, user impact of transport projects, cost benefit analysis and project evaluation, and transport as a means for economic development. The theme also includes research on transport policy analysis, and social equity in transport.

Track IV: Traffic Management, Operations, and Safety

This theme deals with traffic aspects of highways and urban roads such as traffic flow theory and modelling, traffic control and management, transport network analysis, operations and management of passenger and freight traffic, and ICT for traffic systems. This theme also includes the various aspects of transport safety such as driver and infrastructural factors, externalities and policies for transport safety aspects, vulnerable road user safety, safety in public transit, and other modes of transport. The theme also covers the topic on operations and management of public transport systems.

Track V: Emerging Transportation Technologies

This theme focuses on all aspects of technological advances in transportation systems. The topics broadly include battery and electric operated vehicles, connected and autonomous vehicles; internet of things, V2X communication, and intelligent transportation systems; technology enabled models for mobility services of passenger and freight transport; technology enabled multi-modal integration; robotics, artificial intelligence, human-machine interfaces, computer vision, image processing, and augmented reality in transportation; big data analytics for transportation.

Track VI: Sustainable Mobility in Transportation

This theme includes research related to environmental impact assessment of transport projects and mitigation strategies; various interactions between transport, health and associated policies; emissions from vehicles and other transport infrastructure; policies and planning for sustainability in transport systems; renewable energy applications in transport sector; pollution and environmental issues of all transport modes; strategies, technological interventions, policies and management of sustainable and socially inclusive transport systems.

Acknowledgements

The financial aspect of the conference was primarily supported by our sponsors: Navi Mumbai Municipal Corporation (NMMC), Blacktop Infrastructure Private Limited, Thane Municipal Corporation, Maharashtra State Road Development Corporation (MSRDC), Bitpath Private Limited, Savi Infrastructures and Properties, Dineshchandra R. Agrawal Infracon, City and Industrial Development Corporation (CIDCO), Viatop Premium, AIC Infrastructures, Markolines, Zydex Industries, Samarth InfraENGG Technocrats, KABA Infratech, Yash Innovative Solutions, Bitumen Corporation, Mumbai Port Authority and Balajee Infratech & Constructions.

We would like to acknowledge all the authors for submitting and presenting their recent quality work at TPMDC 2022. We are especially grateful to our delegates including key note speakers and invited speakers for sharing their valuable knowledge with the participants. We extend our appreciation to the renowned researchers who

supported us in the critical review process of the received papers. We thank the members of the conference organizing committee Dr. K. V. Krishna Rao, Dr. Tom V. Mathew, Dr. Gopal R. Patil, and Dr. P. Vedagiri, students of the transportation systems engineering and IIT Bombay administration for their valuable assistance for organizing the conference.

Dr. Dharamveer Singh
Dr. Avijit Maji
Dr. Nagendra Rao Velaga
Dr. Solomon Debbarma
Mr. Omkar Karmarkar
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About the Editors

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Professor Singh's field of specialization is in pavement engineering, focusing on recycling, stabilization, new and innovative technology for road construction, pavement design, and maintenance and preservation of pavements, forensic investigation.

Professor Singh is closely associated with industry and highway fraternity on various fronts including pavement design, construction, training, and implementation of new technologies. Professor Singh is member of IRC committees on pavement design, low volume roads, asset management, and composite pavements. Also, Prof. Singh is associated with Bureau of Indian Standards (BIS), technical committee on Bitumen, Tar and Related Products, PCD 06.

Professor Singh has been conferred with Pt. Jawaharlal Nehru Birth Centenary Award-2018 by Indian Roads Congress Nov. 2019. Professor Singh has published more than 100 papers in international journal and conferences. Prof. Singh is an Associate Editor for International Journal, Innovative Infrastructure solutions, Springer Publisher, and Editorial Board Member of International Journal of Pavement Research and Technology, and International Journal of Road Materials and Pavement Design.

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RILEM. Debbarma served as a reviewer for several international peer-reviewed journals. He is a Young Faculty Award recipient awarded by IIT Bombay. In addition, he was the Entrepreneur Lead for the NSF Innovation-Corps project: High-performance Cementitious Materials, funded by the National Science Foundation, USA in 2021.

Pavement Systems Engineering

Development of Maintenance Priority Index for Urban Road Network

Saurabh Singh Yadav, Aakash Gupta, Sachin Gowda, and Yogesh Aggarwal

Abstract The estimated service life of a pavement is determined by design criteria such as geological considerations, water table movements, structural variations, and existing circumstances such as traffic intensity, drainage, and climate. The analysis of deformations and other variables, which influencing the pavement life, is a difficult task since the events that cause them are unpredictable and random in nature. It is unavoidable yet; these variables have an impact on the quality standards of the road network, resulting in decreased in their usable life. As a result, in order to remedy difficulties, it is important to assess or diagnose the current pavement conditions, both structurally and functionally. As a result, the issue of pavement evaluation, which deals with the mentioned element, is critical for pavement management. The functional testing on road surfaces, as assessed by its strength and durability during its service life, is dependent on several subjective measures of its stiffness and roughness. Structural Evaluation of Pavements is required to measure the structural strength of various layers of pavement. It also helps in evaluating a pavement's remaining life and the thickness of overlay necessary. In the current study, a maintenance priority index has been developed using functional and structural parameters and also, it has been compared with the already available maintenance priority tools.

Keywords Functional parameter · Structural parameter · Prioritization index

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1 Introduction

Pavement distress and their continuous deterioration lead to poor riding comfort and safety concerns. Apart from pavement distresses, many other parameters affect the pavement quality. Since the events that lead to deformations and other factors that affect pavement life are unpredictable and random in nature, analysing them can be challenging. It is nonetheless inevitable that these factors will affect the road network's quality standards and shorten its useful life. As a result, it is critical to evaluate or diagnose the current pavement conditions, both physically and functionally, in order to address issues. The topic of pavement evaluation, which addresses the aforementioned component, is therefore crucial for pavement management. The functional testing on road surfaces, as assessed by its strength and durability during its service life, is dependent on several subjective measures of its stiffness and roughness.

2 Literature Review

This section provides a thorough study of the literature on functional pavement and structural pavement of pavement. Discusses the research done by several experts in fieldwork of pavement inspection related to the different factors that influence pavement roughness, models for predicting pavement efficiency. It also contains indices, researchers came up with to prioritize paving latest techniques used to model the various components of pavement, i.e., functional and structural evaluation, pavement performance prediction models, resource allocation optimization and maintenance prioritization procedures. Biasnchini et al. (2013) determine the relative relevance of various forms of distress in determining pavement quality and finding significant characteristics for establishing maintenance plans. The main goal of the study conducted by Boyapati and Kumar [1] is to calculate Pavement Condition Index (PCI) by collecting and analysing field data in order to priorities pavement maintenance [1]. The main aim of the study conducted by Al-Neami et al. is to evaluation of PCI using GIS data and Paver 5.2 software [2]. In Murshan and Vala [3], the primary goal of this research is to offer a maintenance priority index. In Mohammad et al. [4], the main motive of this research is to regulate a physical survey of 50 lane roads and to conduct a relative study on Pavement Condition Rating Methods (PCRM). In Janani et al. [5], the goal of this study is to provide a new approach for prioritizing pavement maintenance sections based only on the functional parameter of the pavement. The purpose of the study conducted by Afifyet et al. (2020) is to evolve an accurate IRI forecast model for flexible pavements using both (ANN) Artificial neural networks and Multiple Linear Regression. In Zhang et al. [6], this study's overall goal was to evolve a structural indicator, using the Falling Weight Deflectometer data. The structural study of the pavement was completed using Benkelman Beam deflection (BBD) to determine the pavement's strength. In Amin et al. (2018) [7], in this research, only non-destructive test technique to analyse pavements using

FWD data is collected. Thereafter to analyse the structural proportions of the current pavement. KGPBACK software was used for back-calculation. In U. Shaha et al. (2013), pavement condition evaluation, which covers distress, roughness, friction, and structural parameter, is an important part of pavement design, rehabilitation, and maintenance. The majority of the cost-effective maintenance and rehabilitation (M&R) methods are produced using the Pavement Management System (PMS) [8]. The weights are determined by employing the Analytical Hierarchy. In Gupta et al. [9], the aim of this study is that development of rural road maintenance priority index is based on functional and structural condition of rural road network [9].

3 Data Collection

In this current study, 48 km road stretch in Delhi NCR, each with length of 1 km and average width of 3.5 m, was selected to collect the details of distress, roughness, and pavement deflection data. Data were collected on site to develop various models that would help in the maintenance of the urban road network. Field data were collected based on functional parameters predominantly prevalent in urban sections of the Delhi NCR Road network. Road roughness, pavement deterioration such as cracks, patching, ruts, potholes are among the functional parameters evaluated, ranging in severity from low to severe. Falling Weight Deflectometer is used to evaluate the structural behaviour of the pavement as shown in Fig. 1.

3.1 Functional Evaluation Data

The functional evaluation of 48 km of the chosen road section comprised gathering functional metrics of road surface distresses (cracks, spalling, potholes, ruts), and road roughness (IRI), which are mostly found in the selected road section.

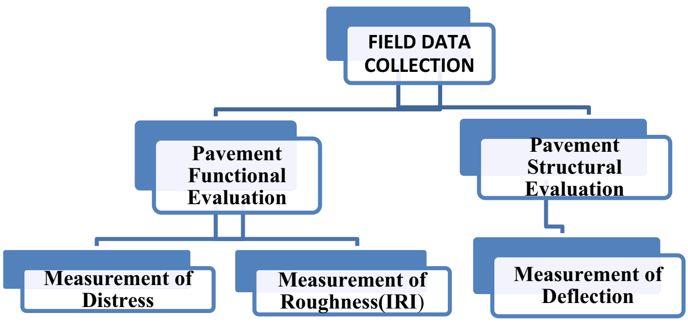
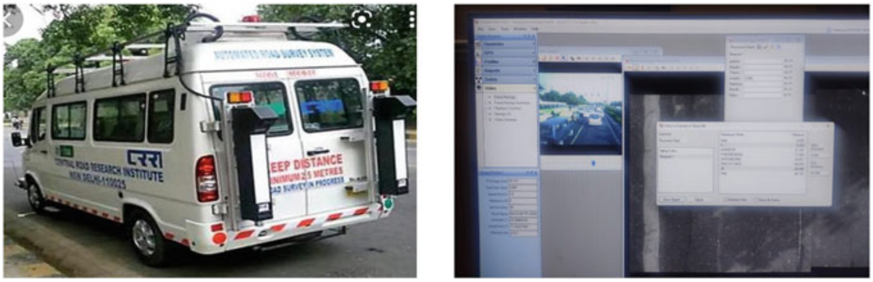


Fig. 1 Flowchart of data collection



(a) Network Survey Vehicle (NSV) (b) Hawkeye Tool

Fig. 2 Network survey vehicle and Hawkeye tool

3.2 Structural Evaluation Data

A structural evaluation of selected road sections in Delhi-NCR was performed. The structural factor, i.e., Pavement Deflection, which is detected using FWD is mainly used for this study.

4 Equipment Used for Analysis of Pavement

4.1 Network Survey Vehicle (NSV)

Network Survey Vehicle (NSV) uses the most up-to-date survey methodologies. Tools for processing video images, laser, GPS, etc. Survey-Vehicle is utilized for the automated collecting road asset management data, pavement maintenance management system data, and inventory data. Surface characteristics of pavement include: Cracking, Rutting, Raveling Potholes, Roughness and Studies on road safety audits. Data collected are analysed using Hawkeye processing tool as shown in Fig. 2.

4.2 Falling Weight Deflectometer (FWD)

Falling Weight Deflectometer (FWD) is an impulse loading instrument that applies a weight to the pavement and measures the deflected shape of the surface. The falling mass is allowed to descend vertically on a set of springs positioned over a circular loading plate that applies the impulse force. Displacement sensors are positioned at varying radial distances from the centre of the load plate to measure the curved shape of the attached surface. The FWD variants are commercially available in both trailer and vehicle mount configurations as shown in Fig. 3. A predetermined amount of

Falling Weight Deflectometer (FWD) FWD Software

Fig. 3 Falling weight deflectometer (FWD)

weight is dropped from a specified height onto a series of springs/buffers installed thereon. At various radial positions, the associated peak load and vertical tip surface deflections are measured and recorded.

5 Methodology

The aim of this study is to produce an overall pavement condition index that can be used for urban road maintenance and makes good use of available resources. For this, firstly the functional data and structural data have been collected. Pavement distress and road roughness are considered as functional parameters while surface deflection is considered as structural parameter. After collecting all the data, Distress Index, Roughness Index, and Deflection Index have been developed individually. This is followed by the Overall Functional Condition Index and the Overall Structural Condition Index. Once, the Overall Functional Condition Index and the Overall Structural Condition Index are calculated, by giving a suitable weightage to them, the Overall Pavement Condition Index is obtained (AHP and Expert Choice software has been used to give the weightage). With the help of the Overall Pavement Condition Index, road maintenance and their ranking can be done. This technique can be used for various types of roads such as state highways, district roads, and rural roads as shown in Fig. 4.

5.1 Development of Functional Condition Indices

To calculate the combined pavement condition index, the following equations given in Table 1 have been used to determine the pavement distress index (PCI Distress).

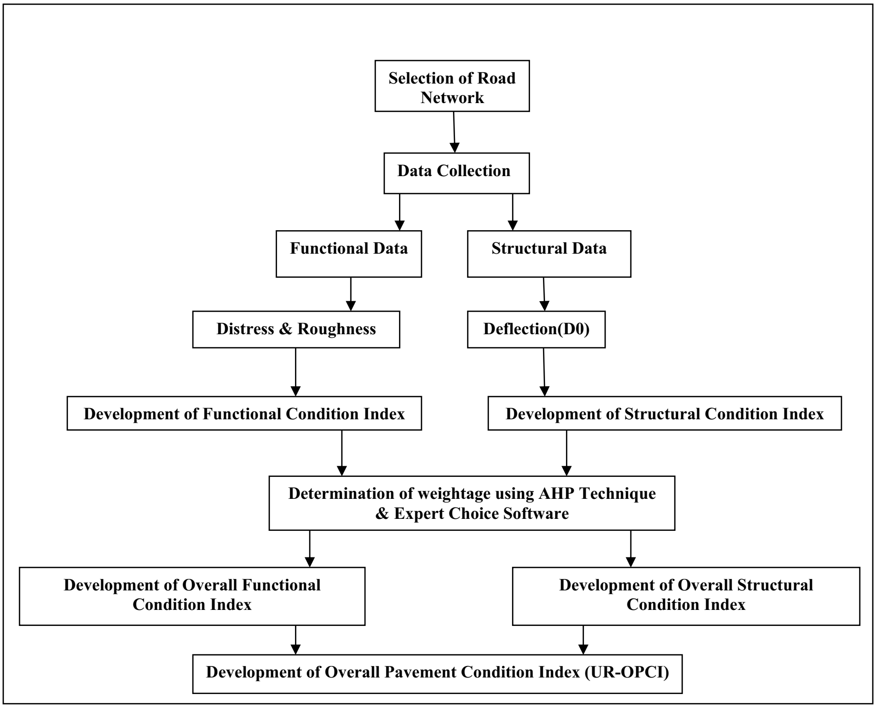


Fig. 4 Overall methodology

The threshold value for all distress has been set to 60, indicating that the pavement needs repairs [9].

The percentages of L, M, and H in Table 1 above represent the proportion of the observed pavement with a given distress among the different severity levels. The denominator numbers are the maximum allowable extent (MAE) for each severity level.

Table 1 Equations used to determine individual distress index

Distress	Low severity index (LSI)	Medium severity index (MSI)	High severity index (HSI)
Longitudinal cracking	100−40* (%L/25)	100−40* (%M/20)	100−40* (%H/10)
Transverse cracking	100−40* (%L/25)	100−40* (%M/20)	100−40* (%H/10)
Alligator cracking	100−40* (%L/50)	100−40* (%M/25)	100−40* (%H/15)
Potholes	100−40* (%L/50)	100−40* (%M/30)	100−40* (%H/10)
Patching	100−40* (%L/50)	100−40* (%M/15)	100−40* (%H/10)
Raveling	100−40* (%L/80)	100−40* (%M/60)	100−40* (%H/30)

5.2 Analytical Hierarchy Process (AHP)

AHP is a rating tool used to assign weights to a number of distress factors to determine their relative importance. The creativity, knowledge, and talent of each individual distress are determined by the AHP. It arithmetically synthesizes the multiple choices or perceptions, checks the coherence of judgements to evaluate each choice, and finally arrives at a result that represents the associated problem statement. The questionnaires are made available to road engineers, scientists, and students to support their individual perception and to support their expertise and knowledge [9].

5.3 AHP Weightage Determination

AHP is a systematic approach that can be used to make complicated decisions. With this method, the problem is first broken down within a hierarchical structure, which is then broken down into several parts. This can assess any complicated decision challenge that involves expert views and perspectives. The AHP analyses any problem with the help of each person's creativity, expertise, and experience. It mathematically combines the various judgments or perceptions, checks the consistency of the judgments to assess each judgement, and then arrives at an output to represent the appropriate issue statement [9].

The analytical hierarchy process approach is used in this study to determine the weighting of several distress criteria that have a significant impact on the International Roughness Index Distress and surface deflection. For this purpose, a questionnaire is shown in the Annexure, a total of 125 questionnaires is distributed, out of which 96 were answered and used to determine the proportional importance of different types of distress. Expert Choice 11 software was used to examine the consistency ratio (CR) of 125. If a response's consistency ratio is greater than 0.1, it has been discarded. The AHP Expert Choice 11 software is used to calculate the Weightage factor of different distress, responses as shown in Table 2.

Table 2 Weightage determined using AHP expert choice 11 software

Pavement distress categories	Weightage (W_i)
Longitudinal cracking (LC)	0.2
Transverse cracking (TC)	0.19
Alligator cracking (AC)	0.39
Potholes (PH)	0.14
Raveling (RA)	0.05
Patching (PA)	0.03

5.4 Combined (PCI Distress)

After assessing each stress index, the Combined PCI stress is calculated using the following Eq. 1. Because the impact of all stresses was not equal, stresses are weighted differently based on the expert choice 11 software to account for their individual impact, which is listed in Table 1.

$$\begin{aligned}(\text{PCI})\ \text{Distress} = 100 * [1-\{1-\text{LC}/100\} * 0.20] * [1-\{1-\text{TC}/100\} * 0.19] * [1-\{1-\text{AC}/100\} * 0.39]\\ * [1-\{1-\text{PA}/100\} * 0.03] * [1-\{1-\text{RA}/100\} * 0.05] * [1-\{1-\text{PH}/100\} * 0.14]\end{aligned} \tag{1}$$

5.5 Roughness Index (PCI Roughness)

A correlation between the IRI (International Roughness Index, m/km) and the Ride Quality Rating (RQR) was discovered by means of regression analysis. The PCI (roughness) is calculated using the polynomials Eq. 2, which gives a perfect result [9].

$$(\text{PCI})\ \text{Roughness} = 1.227 * \text{IRI}^2 - 17.73 * \text{IRI} + 100 \tag{2}$$

5.6 Development of Structural Condition Index (SCI)

Structural Condition Index (SCI) is calculated based on its structure number, which is determined by the rebound deflection of the pavement surface, the layer coefficient and the thickness component of each pavement layer.

(SCI) structural condition index is the ratio of Modified Structural Condition and Effective Structural Condition [9].

$$\text{MSN} = \text{SN} + 3.51\,\text{log10}\,(\text{CBR}) - 0.85(\text{log10 CBR})^2 - 1.43 \tag{3}$$

Here,

$$\text{SN} = 0.0393 \sum_{i=1}^{n} (a_n * d_n) \tag{4}$$

where "a_n" represents the layer coefficients of n layers and "d_n" represents the thickness of n layers of pavement in mm.

"a_n" = 0.3 for Bituminous layer and 0.14 for GSB/subgrade layer.

CBR—denotes the California Bearing Ratio of the pavement sub grade (%)

$$\text{SCI} = (\text{MSN}/\text{SN effective}) * 100 \tag{5}$$

Here,

$$\text{SN effective} = 3.2 * (\text{Characteristic deflection in mm using FWD}) - 0.63 \tag{6}$$

5.7 Evaluation of Overall Functional Condition Index (OFCI) and Overall Structural Condition Index (OSCI)

The Overall Functional Condition Index (OFCI) is influenced by the Functional Condition Index, which includes PCI (distress) and PCI (roughness), while the Overall Structural Condition Index (OSCI) is influenced by the SCI (deflection), respectively.

The questionnaire survey with 125 answers, which corresponded to the questionnaire in the Annexure, will be used to determine the relative importance of OFCI and OSCI by using the Expert Choice 11 tool. The Weightage for the Pavement Distresses Index (PCI, Distresses) and the Roughness Index (PCI Roughness) are achieved with 65% and 35%, respectively. Whereas SCI (deflection) is achieved 100% Weightage. As a result, (OFCI) and (OSCI) were calculated by using Eqs. 7 and 8

$$\text{OFCI} = 0.65 * \text{PCI (distress)} + 0.35 * \text{PCI (roughness)} \tag{7}$$

$$\text{OSCI} = 1 * \text{SCI (deflection)} \tag{8}$$

5.8 Evaluation of Overall Pavement Condition Index (UR-OPCI)

The Overall Pavement Condition Index (UR-OPCI) was determined based on (OFCI) and (OSCI). The weightage was individually applied with structural and functional pavement criteria to define the final overall pavement condition index for best results. The weight values were calculated by Expert Choice 11 software. From the evaluation of 125 surveys, functional factors received a weighting of 60% and, as a result, structural parameters 40%; the final UR-OPCI was calculated using Eq. 9. The calculated value of PCI (distress), PCI (roughness), PCI (deflection), OFCI, OSCI, and OPCI are listed in Table 3.

$$\text{OPCI} = 0.6 * (\text{OFCI}) + 0.4 * (\text{OSCI}) \tag{9}$$

Table 3 UR-OPCI value ranges

UR-OPCI range	Pavement rating	Maintenance and repair strategies
0–15	Very poor	Full-depth reconstruction, reclaimed asphalt pavement recycling
15–30	Poor	Thick overlays, premix, carpet, surface dressing
30–50	Fair	Thick overlays, full-depth patching, pothole filling
50–65	Good	Thin overlays, patching, fog seal
65–80	Very good	Thin overlays, chip seal, micro-surfacing
80–100	Excellent	Routine maintenance that includes micro-crack sealing patching

5.9 UR-OPCI-Based Maintenance and Repair Strategies

The current study proposes specific maintenance and repair techniques for preventive and corrective actions in urban road sections based on different value ranges of the Overall Pavement Condition Index (UR-OPCI). Because the Overall Pavement Condition Index (UR-OPCI) was developed using the functional and structural parameters of urban roads, it is considered the best indicator of road surface condition. Table 3 recommends maintenance and repair procedures according to different UR-OPCI value ranges [9].

6 Results and Discussion

a. Figure 5 shows individual functional condition indices and structural condition indices. Road R13 shows the highest PCI (distress) value and R24, R27, R39 show the lowest PCI (distress) value. R1 shows the highest PCI (roughness) value and R47 shows the lowest PCI (roughness). Similarly, R4 shows the highest SCI (deflection) and R7 shows the lowest SCI (deflection) (Table 4).

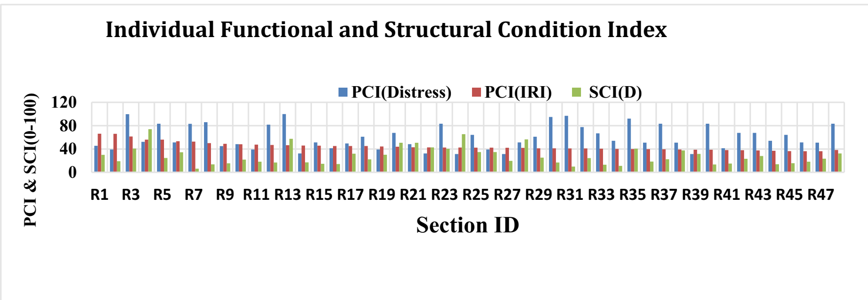


Fig. 5 Individual functional and structural indices

Table 4 Calculated value of PCI (Distress), PCI (IRI), SCI (D), OFCI, OSCI and UR-OPCI, and ranking based on UR-OPCI for selected urban road network

ID	PCI (ASTM)	PCI (distress)	PCI (IRI)	SCI (D)	OFCI	OSCI	OPCI	Ranking based on UR-OPCI
R1	98	45.51	66.29	29.9	52.78	29.9	43.63	26
R2	100	39.15	66.03	19.1	48.56	19.1	36.77	15
R3	68	99.70	61.48	40.7	86.32	40.7	68.07	47
R4	85	52.34	56.05	73.9	53.64	73.9	61.74	46
R5	79	83.41	56.03	24.5	73.83	24.5	54.10	42
R6	79	51.35	53.27	34.3	52.02	34.3	44.93	27
R7	72	83.18	52.56	6.33	72.46	6.33	46.01	30
R8	97	85.95	50.16	13.6	73.42	13.6	49.49	36
R9	89	44.97	48.85	15.4	46.33	15.4	33.96	8
R10	100	48.34	48.11	21.8	48.26	21.8	37.68	16
R11	100	39.16	47.43	18.2	42.05	18.2	32.51	5
R12	78	81.67	46.91	16.8	69.51	16.8	48.42	34
R13	79	99.93	46.50	57.5	81.23	57.5	71.74	48
R14	100	32.29	45.88	17.1	37.05	17.1	29.07	2
R15	100	51.34	45.67	14.7	49.36	14.7	35.50	11
R16	100	41.22	45.16	14.3	42.59	14.3	31.28	4
R17	83	49.40	45.14	31.9	47.91	31.9	41.51	21
R18	88	60.99	45.09	22.1	55.43	22.1	42.10	22
R19	89	39.16	44.28	30.1	40.95	30.1	36.61	13
R20	72	67.52	43.97	50.7	59.28	50.7	55.85	43
R21	100	48.34	42.97	50.7	46.46	50.7	48.15	33
R22	100	32.29	42.69	42.7	35.93	42.7	38.64	18
R23	100	83.39	42.47	40.3	69.07	40.3	57.56	44
R24	100	31.33	42.45	65.3	35.22	65.3	47.25	31
R25	100	64.18	42.39	34.8	56.52	34.8	47.83	32
R26	100	39.15	42.30	34.8	40.16	34.8	38.02	17
R27	100	31.33	42.04	19.6	35.01	19.6	28.85	1
R28	100	51.34	41.86	56.5	47.79	56.5	51.27	39
R29	100	60.97	41.19	25.1	54.03	25.1	42.46	23
R30	78	94.95	41.13	16.8	76.06	16.8	52.36	40
R31	100	96.96	40.99	9.91	77.31	9.91	50.35	38
R32	96	77.55	40.82	24.3	64.58	24.3	48.47	35
R33	100	66.73	40.51	13	57.43	13	39.66	20
R34	100	54.04	40.16	11.1	49.11	11.1	33.90	7
R35	62	92.13	39.94	40.5	73.81	40.5	60.49	45

(continued)

Table 4 (continued)

ID	PCI (ASTM)	PCI (distress)	PCI (IRI)	SCI (D)	OFCI	OSCI	OPCI	Ranking based on UR-OPCI
R36	89	50.88	39.79	18.5	46.92	18.5	35.55	12
R37	76	83.38	39.58	22.4	67.88	22.4	49.69	37
R38	87	50.88	39.10	37.5	46.63	37.5	42.98	24
R39	93	31.33	38.74	31.5	33.72	31.5	32.83	6
R40	73	83.24	38.66	13.5	67.35	13.5	45.81	29
R41	100	41.22	38.16	15.1	39.95	15.1	30.01	3
R42	62	67.52	37.85	23.2	56.79	23.2	43.35	25
R43	68	67.53	37.59	28	56.63	28	45.18	28
R44	70	54.04	36.86	13.9	47.83	13.9	34.26	9
R45	67	64.18	36.39	15.6	54.39	15.6	38.87	19
R46	68	51.35	36.12	18.3	46.02	18.3	34.93	10
R47	88	50.86	36.05	23.3	45.67	23.3	36.72	14
R48	69	83.35	38.27	32.3	67.57	32.3	53.46	41

b. Figure 6 Comparison of PCI calculated using the ASTM method and overall pavement condition index (UR-OPCI) using the methodology used in the present study. R13 shows the highest UR- OPCI and R27 shows the lowest UR-OPCI value.
c. UR-OPCI can also prove to be cost-effective and give correct strategic maintenance priority of urban road network. As a result of the above discussion, it is clear that the Overall Pavement Condition Index (UR-OPCI) is an accurate tool for identifying the priority ranking for maintenance plans of various urban road sections.

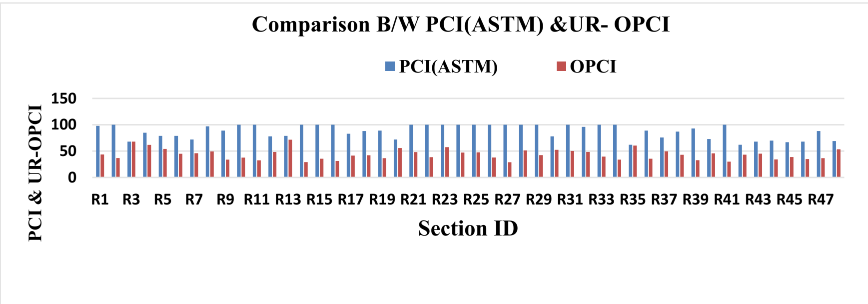


Fig. 6 Comparison B/W PCI (ASTM) and UR-OPCI

7 Conclusions

The current study examined urban roads to prioritize the urban road network and effective use of road maintenance funds. The current study assessed pavement maintenance through functional and structural assessments of urban roads. 48 urban road segments in DELHI-NCR were selected for the development of Roughness Index model, a Deflection model, a Distress model and finally an Overall pavement Condition Index (UR-OPCI). The following conclusions can be drawn from this study—

a. The objectives were achieved after a detailed collection of the data on structural and functional assessment aspects, which aided in the creation of roughness index models, distress models, and pavement surface deflection models, all of which led to the formulation of OPCI.
b. According to study of the Overall Pavement Condition Index (UR-OPCI), R13 has UR-OPCI in the range of 65–80, indicating that pavement is in very good condition, it requires Thin Overlays, Chip Seal, and Micro-surfacing.
c. According to UR-OPCI ranking, R27 has the highest priority and should be serviced first, while R13 is the best road and should be serviced last.
d. The UR-OPCI leads to an effective allocation of maintenance resources and is proving to be a valuable tool for road maintenance engineers and traffic managers. It also contributes to the long-term growth of the country.

Annexure—Format of questionnaire survey

Format of Questionnaire survey for functional and structural parameters of pavement

Questionnaire

Relative importance of parameters corresponding to pavement performance and maintenance

	Fundamental scale
Intensity of importance	Definition
1	Equal importance
3	Somewhat more important
5	Much more important
7	Very much more important
9	Absolutely more important
2,4,6,8	Intermediate value when compromise is required between above

Part-A

Please check the box with a tick (☑) for the relative importance between group 1 and group 2 parameters

Sample

For example: If you think that road rutting is 5 times more important than road ravelling then put (☑) under 5 on left side towards cracking

Group 1 parameter	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Group 2 parameter
Rutting					✓													Ravelling

For example: If you think that road rutting is 5 times more important than road cracking then put (☑) under 5 on right side towards ravelling

Group 1 parameter	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Group 2 parameter
Rutting													✓					Ravelling

For example: If you think that road rutting has equal importance to road ravelling then put (☑) under 1 in the centre

Group 1 parameter	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Group 2 parameter

(continued)

(continued)

Questionnaire																		
Relative importance of parameters corresponding to pavement performance and maintenance																		
	Fundamental scale																	
Intensity of importance	Definition																	
Rutting									✓									Ravelling

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Modelling of Deflection Basin Parameters of Asphalt Pavements Using Artificial Neural Networks and Adaptive Neuro-Fuzzy Inference Systems

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Abstract Non-destructive testing equipment, such as the Falling Weight Deflectometer, offers crucial evaluations of the structural state of the road and enhances pavement management systems. Various approaches based on pavement surface deflection measured using Falling weight deflectometers are widely used around the world for assessing structural stability. The backcalculation of pavement layer moduli has been a widely recognized approach for assessing the structural adequacy of the pavement. However, consistently performing these tests at the network level is laborious, and the subsequent interpretation of the data requires technical expertise, a great deal of time, finance, and other resources. Because of this structural component of roadways, decisions when choosing between maintenance and repair are often neglected. This study uses a variety of structural, functional, environmental, and subgrade soil properties as input parameters to develop a trusted relationship for the estimation of seven different deflection basin parameters such as surface curvature index, Base Curvature Index, Base Damage Index, Area Under Pavement Profile, Deflection Ratio, Shape factors F1 and F2. An effective model was developed using artificial intelligence-based soft computing techniques; Artificial Neural Networks (ANN) and Adaptive Neuro-fuzzy Inference Systems (ANFIS) to predict the output deflection basin parameters from the input variables. The data to train, test and validate the model were gathered through field trials. To achieve the above goal, several models based on ANN and ANFIS were trained by changing number of hidden layers, the neurons in the layer and number of membership functions. Prediction efficiency of the model is assessed based on its root mean square error and the coefficient of determination value.

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Keywords Surface curvature index · Base curvature index · Base damage index · Area under pavement profile · Deflection ratio · Shape factors

1 Introduction

Pavement surface deflection has been utilized by a number of agencies to measure its structural performance. To measure the deflection of the road surface under loading, devices like deflectograph and falling weight deflectometers are frequently employed. In project-level studies, structural assessments and remediation solutions are often provided using mechanistic approaches and back-calculation techniques. Decisions concerning the choice and use of appropriate procedures for pavement restoration are frequently made using data on the functional and structural state of the pavement. Many highway agencies have utilized the La Croix deflectograph and Benkelman beam extensively to measure roadway deflection and determine the need for repair and reconstruction.

According to Kim et al. [1]; Talvik [2], the surface curvature index (SCI), more closely associated to the properties of bound layer, is the difference between the deflection at the centre and the deflection at 300 mm radial offset from the centre (D_0–D_{300}). The parameter (D_{300}–D_{600}) is called the as the Base Damage Index (BDI) and is more closely related to the base layer, while (D_{600}–D_{900}) represents the Base Curvature Index (BCI) and reflects the condition of the base layer and subgrade. Area Under Pavement Profile (AUPP), another property derived from deflection, has been used effectively to quantify pavement stiffness as well as to link the horizontal strain at the base of bituminous layer. The shape parameters F1 and F2 help determine the state of the layer at a given depth by representing the degree of curvature of the deflection basin.

As a result, various rapid analysis techniques are required to quantify the structural health of the pavement using FWD data. Utilizing DBPs is one of the approaches that aids in determining the geometry of the deflection basin under the load. The usefulness of DBP in assessing the structural condition of roads has been highlighted by a number of researchers in the past [1–8]. The commonly proposed DBP includes the base damage index, the SCI, the BCI, the AUPP [1, 6–12]. The area ratio as well as normalized area ratio were used effectively to check the structural health of pavement [13].

In order to draw conclusions on the structural health of pavement layers, this study uses Deflection Basin Parameters (DBPs). Two different AI-harnessed models were developed to indirectly measure the DBP from input variables that are relatively easily collectable. The results of the study are presented in this paper, which supports the use of AI models for the interpretation of pavement condition.

1.1 Objectives

This primary goal of the research was to evaluate the usefulness of Deflection Basin Parameters (DBPs) generated during FWD testing as network-level indicators of highway structural health. To assess the functional condition of the pavement, distress survey was performed using Network Survey Vehicle (NSV).

The output variables in this study are the most commonly used DBPs such as SCI, BCI, BDI, AUPP, DR and Shape Factors F1 and F2, while the input variables are pavement structure-related variables such as Bituminous layer thickness (La), Granular layer thickness (Lb) and; functional performance variables such as the % area of Cracking; and Subgrade soil strength indicators such as Soil type, Plasticity Index, Maximum Dry Density (MDD) and California Bearing Ratio (CBR); and finally environment variables such as pavement temperature and ambient temperature. The choice of input variables is primarily influenced by the accessibility and ease of collecting these data compared to that of FWD. Contrarily, as the entire method—from data collection to back-calculation analysis—must be followed each time with FWD testing, the current situation demands a method that reduces the frequency of FWD testing. This study emphasizes on combining variables from several areas to produce accurate and highly flexible prediction models.

2 Analysis

To find a suitable way to evaluate the correlation between the DBPs and Cracking, PI, MDD, CBR, Bituminous layer thickness, Granular layer thickness, Air temperature and Pavement temperature ANN, and ANFIS analysis were carried out and compared. The associated methods are briefly outlined below.

2.1 Artificial Neural Network

ANN is an analogue of a minimalistic illustration of how the biological human brain is put together. Recently, academics working on advanced technical problems has used ANN to solve these challenges. Layers of interconnected neurons—the basic processing unit constitutes the conventional ANN architecture. The many connections that exist between these neurons and their ability to learn from their input allow the output to be predicted effectively. A set of input nodes, some hidden nodes at one or more levels, and some output nodes form the basic structure of the ANN. The training function, the transfer function, the number of hidden layers, and the number of neurons in those layers all affect how well the model can predict the output. By combining multi-layer perceptron, feed-forward-back propagation, log-sig (log-sigmoid) transfer function and Levenberg–Marquardt learning function, a

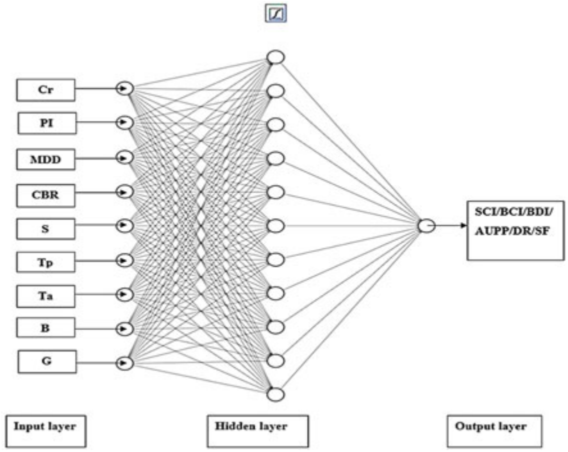

Fig. 1 Architecture of ANN model

suitable ANN model to predict the output DBPs was developed (trainlm). Several combinations of the number of hidden layers and the right number of neurons were examined to get a model with a better R^2 value and a lesser RMS error. The ANN model was developed with single hidden layer for nine inputs: cracking, PI, MDD, CBR, bituminous layer thickness, granular layer thickness, air temperature and pavement temperature and an output deflection basin parameter such as SCI, BCI, BDI, deflection ratio, area under pavement profile, shape factor F1 and shape factor F2. Of the total data points, 70% were used to train the model. 15% were used to test the model and the remaining 15% of data were validated using MATLAB r2022a software. The design of the proposed ANN model is shown in Fig. 1.

2.2 *Adaptive Neuro-Fuzzy Inference System*

Using a hybrid algorithm, the ANFIS model was evolved that incorporated both ANN and fuzzy logic with a series of fuzzy language rules, producing Input-output models resembling human-like knowledge and specific input–output data combinations. Membership functions in ANFIS modelling signify how fuzzy the data set is. Based on its geometry, they are classified as triangular, bell-shaped, trapezoidal and Gaussian membership functions. The trapezoidal membership function (Trapmf) with different epoch numbers was used to identify the model with the lowest RMSE value for the ANFIS study. Takagi–Sugeno type ANFIS with Grid partitioning was employed in the construction of FIS. In addition, the resulting FIS was trained using 70% of the data using a hybrid learning algorithm. The model was validated on 15% of the data and was tested on the remaining 15% of the data using MATLAB r2022a. The structure of the proposed model is illustrated in Fig. 2.

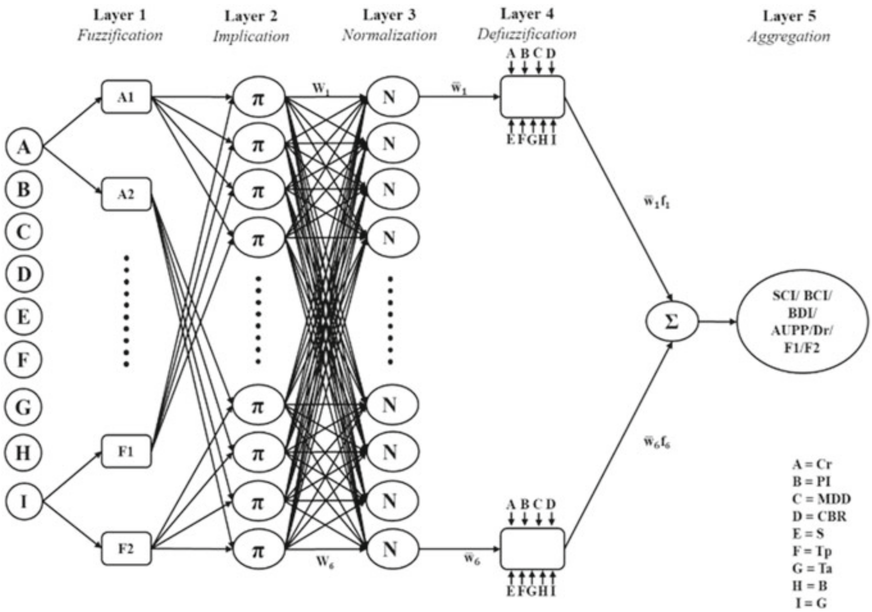

Fig. 2 Proposed architecture of ANFIS model

2.3 Performance Criteria

The potential of the ANN and ANFIS models was assessed statistically by computing the RMSE, R^2 values. These criteria are defined by the equations below.

$$\text{RMSE} = \sqrt{\frac{\sum_{j=1}^{n} (E - A)^2}{n}}$$

$E =$ Predicted Value

$A =$ Observed Value

$$R^2 = 1 - \frac{\sum (A_i - E_i)^2}{\sum (A_i - \overline{A}_i)^2}$$

$\overline{A} =$ Average of A values

$n =$ Number of Observation

3 Results and Discussions

3.1 ANN Results

The ANN model was developed with single hidden layer for nine inputs: cracking, PI, MDD, CBR, bituminous layer thickness, granular layer thickness, air temperature and pavement temperature and an output deflection basin parameter such as SCI, BCI, BDI, deflection ratio, area under pavement profile, shape factor F1 and shape factor F2. To create an effective model with the lowest RMSE and highest R^2 value, the architecture of the models was varied by altering the number of neurons in the hidden layer. A logsig transfer function was applied to this hidden layer of different number of neurons. A total of seven different models were developed for different outputs with same inputs. The test of the model for predicting the output was established through training and validation data. Model performance of training, validation, testing for RMSE and R^2 are given, respectively, the following Table 1 gives the results and in Fig. 3, the R^2 values obtained for the network structure SCI (9-10-1).

3.2 ANFIS Results

The ANFIS model was developed using two trapezoidal membership functions (trapmf) for each input variable to determine the deflection basin parameters. The input parameters such as cracking, PI, MDD, CBR, bituminous layer thickness, granular layer thickness, air temperature and pavement temperature and different deflection basin parameters such as SCI, BCI, BDI, deflection ratio, area under pavement profile, shape factor F1 and shape factor F2 as output resulted in seven different models. Table 2 summarizes their performance in terms of RMSE and R^2 value and Fig. 4, shows the regression results for SCI using ANFIS.

4 Discussion

The artificial intelligence harnessed soft computing techniques; ANN and ANFIS used in this work have successfully shown how input and output variables correlate with each other. The R^2 values in the range of 0.66–0.93 for the ANN model and 0.45–0.86 for ANFIS model depict of strong correlation between the input and output parameters. The RMSE error for the models ranged from 0.014–0.335 to 0.007–0.416, respectively. Although it is observed that the R^2 values are satisfactory, it should be highlighted that the data used for modelling came from meticulous field testing and might as well involve human errors in the data collection. The robustness of these models is ensured by the big dataset and successful field testing, and their application has useful field consequences. Even though this research limited the

Table 1 Test results of ANN model

Architecture	Neurons in hidden layer	Output parameters	Model performance	RMSE	R^2
9-10-1	10	SCI (mm)	Training	0.014	0.927
			Validation	0.016	0.912
			Testing	0.015	0.901
9-14-1	14	BCI (mm)	Training	0.008	0.906
			Validation	0.009	0.872
			Testing	0.009	0.900
9-11-1	11	BDI (mm)	Training	0.019	0.837
			Validation	0.021	0.815
			Testing	0.020	0.776
9-14-1	14	Deflection ratio	Training	0.022	0.846
			Validation	0.023	0.787
			Testing	0.025	0.812
9-13-1	13	AUPP (mm)	Training	0.079	0.815
			Validation	0.092	0.770
			Testing	0.096	0.762
9-15-1	15	Shape factor F1	Training	0.072	0.779
			Validation	0.075	0.710
			Testing	0.082	0.754
9-14-1	14	Shape factor F2	Training	0.301	0.708
			Validation	0.305	0.720
			Testing	0.335	0.658

number of hidden layers to unity for the convenience of modelling for a very big database, the R^2 values can still be enhanced in the case of ANN models by adjusting the number of hidden layers. When dealing with ANFIS, experimenting with various combinations of membership functions and increasing the number of membership functions can lead to a higher R^2 value. However, for two membership functions, this research employed trapezoidal membership function, which provided higher coefficient of correlation and least RMSE value compared to other membership functions.

The research successfully demonstrates the ease and capability of artificial intelligence-based soft computing techniques to surpass complex challenges of modelling pavement reactions, where a variety of diverse components play important roles. By doing such comparable, reliable models would help respective authorities speed up the decision-making processes for pavement maintenance and rehabilitation measures.

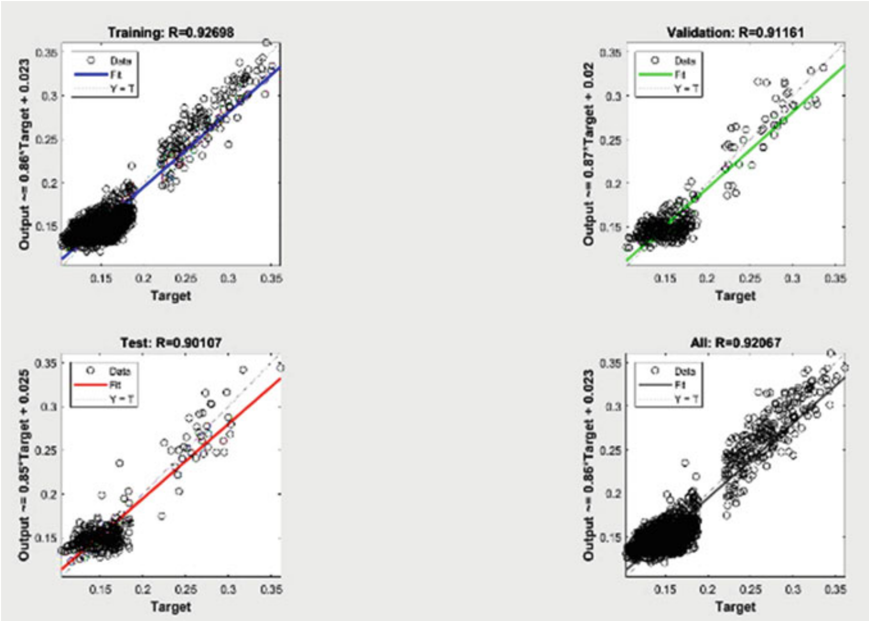

Fig. 3 Regression results obtained for the network structure SCI (9-10-1)

In general, the value of the coefficient of determination, which measures how well the input and output parameters are related, is found to be high (close to one), and the mean square error, which is the mean squared difference between the actual outputs and predicted output, is found to be equally low with reasonable accuracy. However, appropriate numbers would vary from situation to situation and depend on the availability of data. The average R^2 value of the ANN models developed in the study is 0.81 throughout training, validation and testing. The highest R^2 value was observed in predicting SCI, and the lowest value was observed in predicting shape factor F2 (Table 1). Similarly, the average R^2 value of the ANFIS models developed in the study throughout training, validation and testing is 0.64. Also, in this model, the highest R^2 value was observed in predicting SCI and the lowest value in predicting shape factor F2 (Table 1).

The higher R^2 values demonstrate the significance of the correlation generated for the seven models developed using ANN and ANFIS, respectively, in this study, which is further supported by the low MSE values.

Table 2 Test results of ANFIS model

Number of membership functions	Output parameters	Model performance	RMSE	R^2
2	SCI (mm)	Training	0.017	0.848
		Validation	0.026	0.733
		Testing	0.031	0.662
2	BCI (mm)	Training	0.007	0.858
		Validation	0.012	0.662
		Testing	0.017	0.675
2	BDI (mm)	Training	0.013	0.792
		Validation	0.020	0.671
		Testing	0.021	0.675
2	Deflection ratio	Training	0.022	0.721
		Validation	0.031	0.490
		Testing	0.037	0.467
2	AUPP (mm)	Training	0.076	0.726
		Validation	0.102	0.621
		Testing	0.138	0.612
2	Shape factor F1	Training	0.067	0.673
		Validation	0.089	0.582
		Testing	0.118	0.541
2	Shape factor F2	Training	0.284	0.586
		Validation	0.387	0.432
		Testing	0.416	0.446

5 Conclusions

The appropriateness of utilizing AI-powered models for the prediction of structural performance parameters in asphalt pavements is justified in this research. The DBPs, namely SCI, BCI, BDI, AUPP, DR, F1 and F2, have been modelled using ANN and ANFIS to indirectly measure it from the structural, functional, environmental and subgrade soil properties. The study developed seven models for each of the DBPs as outputs using both ANN and ANFIS approach. The dataset to train, validate and test the model was obtained from field studies conducted by performing a FWD and NSV study. A big dataset of 2001 records were gathered via field testing. Further laboratory tests were performed to assess the subgrade soil properties. Seven ANN structures as 9-10-1, 9-14-1, 9-11-1, 9-14-1, 9-13-1, 9-15-1 and 9-14-1 for each of the seven output parameters independently to achieve more accurate modelling perspectives. The ANN model predicted better with R^2 value as high as 0.93 and RMSE value as low as 0.014. Similarly, seven ANFIS models were generated for

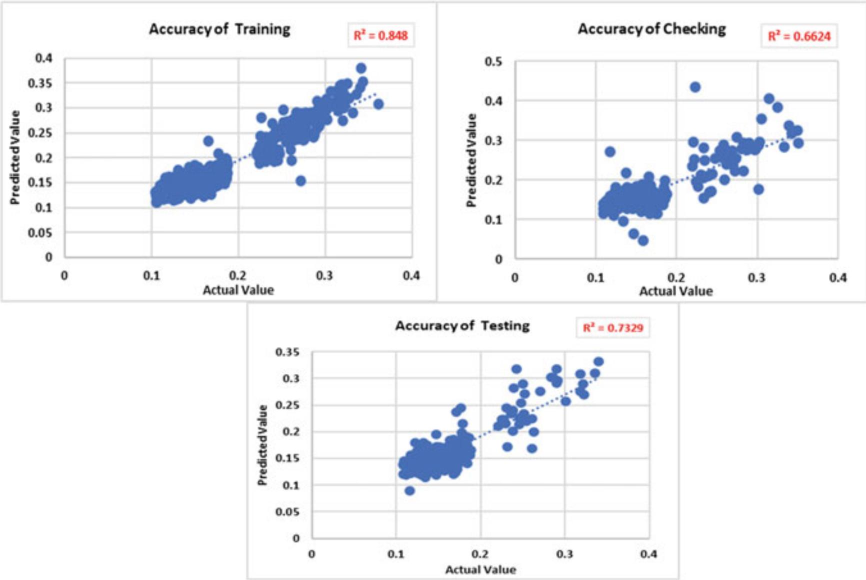

Fig. 4 Regression results for SCI using ANFIS

each of the seven output DBPs. The model could provide a prediction efficiency with a R^2 value as high as 0.086 and RMSE value as low as 0.007.

The higher R^2 values in predicting the Surface Curvature Index explain why the asphalt layer's characteristics have a greater overall influence on pavement quality. The results demonstrate the higher prediction efficacy of ANN model compared to ANFIS model. The study's preliminary methodology offers valid relationship between the adopted input parameters to the Deflection bowl parameters. This quick and straightforward method of data analysis reduces the requirement for the laborious backcalculation procedure.

In addition to providing a comprehensive knowledge of the pavement layers that contribute to the current state, the proposed models would aid in estimating Deflection bowl parameter values, which directly indicate of the structural condition of the pavement. This will substantially reduce the cumbersome data collection through FWD due to the simplicity of gathering input dataset. This will, however, reduce the dependency of Maintenance and Rehabilitation agency on the functional condition of the pavement.

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Comparative Studies on Gel-Incorporated Flexible Pavement

Delvin J. Joseph, Padmakumar Radhakrishnan, and Vignesh Dhurai

Abstract The climatic changes we are currently experiencing are so drastic that the existing pavements are not able to keep up with them; among these, the floods are more severe as they occur more frequently than other hazards. Aerogel is a substance that is highly resistant to water. This paper presents the study of application of aerogel in bituminous pavement construction. Since silica-gel is having similar features, for comparison silica-gel modified pavement is created, which is used to compare the results obtained with the aerogel-incorporated bitumen pavement. The FEM analysis results of both silica-gel and aerogel-modified bitumen are compared with the standard bitumen pavement analysis results. Similarly, aerogel-incorporated concrete pavement model is created, and results are compared with the standard concrete pavement. The aerogel-incorporated pavement performs well compared to silica gel while maintaining the required strength required for the standard pavement.

Keywords Aerogel · Finite element method · Water resistant pavement · Gel-incorporated pavement

1 Introduction

The primary objective of a pavement structure is to provide a surface that offers acceptable riding quality, skid resistance, light-reflecting characteristics, and low noise pollution. The aim is to ensure that the stresses transmitted due to wheel loads are reduced sufficiently, so they do not exceed the bearing capacity of the sub-grade.

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There are two main types of pavements that serve the purpose: flexible pavement and rigid pavement. An ideal pavement must meet certain requirements to perform its functions effectively. These requirements include having sufficient thickness to distribute wheel load stresses to a safe value on the subgrade soil, being structurally strong enough to withstand all types of stresses imposed upon it, having an adequate coefficient of friction to prevent skidding of vehicles, having a smooth surface to provide comfort to road users even at high speed, producing minimal noise from moving vehicles, having a dust-proof surface that does not reduce visibility and an impervious surface that protects the sub-grade soil. Furthermore, an ideal pavement should have a long design life and low maintenance cost.

Apart from these requirements, one of the most critical aspects that an ideal pavement must meet is the ability to withstand the detrimental effects of water over its lifetime. Thus, it can be summarized that a pavement structure should be designed to meet all these requirements to provide safe and comfortable driving conditions.

Silica aerogel particles are used to prepare lightweight concrete by replacing normal aggregates of concrete, the resulting mix was lightweight and thermal insulating concrete material, and the aerogel particles were stable during the hydration of cementitious materials [2]. The application of aerogel and their potential in heritage buildings along with technical properties of commercially available aerogel materials, super-insulating aerogel materials have an exceptional potential in the refurbishment of heritage buildings [3]. The aerogel can manufacture with locally available materials while large specific surface area, high extent of porosity and very low density followed by a microstructure in the form of interconnected pores and channels making these materials very fascinating for most of their high-performance applications [4].

Types, properties, industrial applications, manufacturing procedures and products of aerogel, modified products like oxide aerogels, polymeric aerogels, mixed aerogels, hybrids, composites with fibres etc. help to use them in different fields due to their wide varieties of properties [5]. Silica gel is a compound capable of absorbing water to their structures, asphalt mix at different combination of silica gels was compared with hot mix and warm mix asphalt with zeolites, and the results were like that with zeolites [6].

From the literature review, no study was reported on the evaluation of the potential of gels like aerogel in enhancing the moisture resistance and thereby the durability of pavements. The aim of this study is to compare the performance of gel-incorporated pavement with hot-mix asphalt pavement using the aid of finite element analysis by ABAQUS software.

2 Methodology

2.1 Aerogel

Aerogel, being a nanostructured, open porous solid made via sol–gel technology was selected for the pavement construction. There are several types of aerogels, some of them are:

- Oxides (Quarz, Titania, Zirkonia. Mixed Oxides)
- Polymers (Resorcin-, Melamin-Formaldehyde)
- Carbon (Pyrolyzed Polymere)
- Cellulose, starch etc. and almost everything that can be gelled (Table 1).

Among the above types of aerogels, silica-based aerogel is selected as they are available in abundance and easy to produce (Table 2).

2.2 Pavement Model Using ABAQUS™

A three-dimensional finite element model used for a typical flexible pavement section designed in ABAQUS™ software consists of bitumen surface layer, a base layer, a

Table 1 Main properties and typical values of aerogels depending on their composition [4]

Composition	Bulk density (gcm^{-3})	Specific surface area (m^2g^{-1})
Silica aerogel	0.003–0.5	600
Resorcinol formaldehyde aerogel	0.005–0.3	100–1500
Carbon aerogel	0.05	100–1000
Cellulose aerogel	0.1–0.35	200–400

Table 2 Main physical properties of SiO_2 aerogels [4]

Properties	Range	Typical value
Bulk density (kg/m^3)	3–150	100
Skeletal density (g/cm^3)	1.7–2.1	
Porosity (%)	90–99.8	
Mean pore diameter (nm)	20–150	
Inner surface area (m^2/g)	500–1500	1000
Refractive index	1.007–1.24	1.02
Thermal conductivity λ (in air 300 K) (Wm^2/g)	0.014–0.021	0.015
Young's modulus E (Mpa)	0.002–100	1
Sound velocity (m/s)	20–800	100

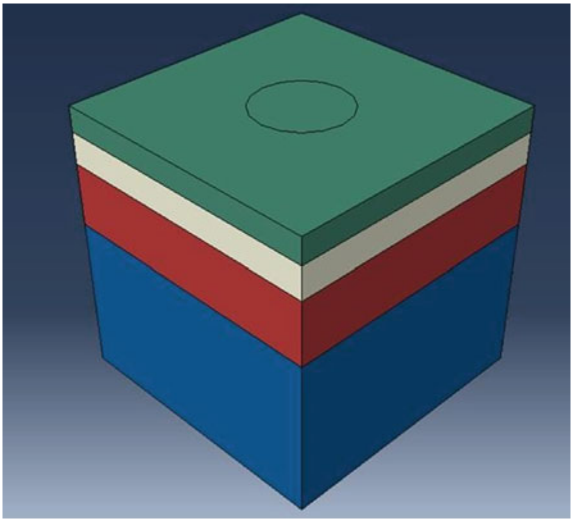

Fig. 1 Finite element model of standard pavement

subbase layer and a subgrade layer, which are all simulated by 3D deformable solid extrusion elements (solid homogeneous section type). The pavement surface was designed about 880 × 880 mm dimension and having a total thickness of 880 mm. The depth 880 mm is subdivided into four layers with thickness, respectively, as surface layer (80 mm), base layer (100 mm), subbase layer (200 mm) and subgrade (500 mm) (Fig. 1 and Table 3).

2.3 Load and Boundary Conditions

The design of pavement systems is heavily influenced by traffic load conditions, which include factors such as axial loads, axle configurations, tire contact areas, number of load repetitions and vehicle speed. Heavy vehicular traffic, particularly from trucks, is a significant cause of pavement distress and failure. In numerical and theoretical analyses, the most common method of applying wheel load is through uniformly distributed tire pressure loads on a circular equivalent contact area based on the wheel load and tire contact pressure. The wheel load is equal to half of the axle load and can be applied to the entire wheel path in the case of lading areas.

To simulate heavy traffic, an impulse-type load was used with an amplitude equal to half of an axle load ($P_{Axle} = 80$ kN, so $P = 40$ kN), resulting in a pavement stress of 0.56 MPa. This surface wheel load (P) was applied to a circular pressure contact area with a 150 mm radius at the centre of model domain. Boundary conditions were set for the model, with the bottom of the model constrained and displacements and rotations prevented on the sides parallel to the x and y axes. To accurately estimate the stress field in the pavement road section, the degree of mesh refinement is critical, with the

Table 3 Properties of each layer and substances [1, 3, 6]

Layer	Thickness (mm)	Density × 10^{-9} (tonne/mm^3)	Young's modulus (Mpa)	Poisson's ratio
Surface	80	2.332	2,000	0.3
Base	100	2.162	300	0.35
Subbase	200	1.922	600	0.35
Subgrade	500	1.762	76	0.45
Silica-gel bituminous pavement (SGBP)	80	2.859	1,000	0.35
Concrete pavement	80	2.4	40,000	0.15
Aerogel incorporated concrete pavement (AICP)	80	1	5,533	0.2
Aerogel incorporated bituminous pavement (AIBP)	80	0.2	659	0.2

densest mesh required for the superficial layer where the load is applied to capture permanent displacements. The pavement layers were meshed using eight-node linear brick elements (C3D8R).

Since traffic load conditions have a significant impact on the design of pavement systems, with heavy vehicular traffic being a primary cause of pavement distress and failure. The most common method of applying wheel load in numerical and theoretical analyses is through uniformly distributed tire pressure loads on a circular equivalent contact area. To accurately estimate the stress field in the pavement road section, the degree of mesh refinement is crucial, with a denser mesh required for the superficial layer to capture permanent displacements. Boundary conditions were set for the model to ensure accurate results, and eight-node liner brick elements were used to mesh all pavement layers.

3 Results and Discussion

In order to investigate the performance of pavement system, a series of 3-D-finite element dynamic simulations were carried out in order to evaluate the benefits offered against permanent deformation or rutting in function of the number of cycles of load.

3.1 Analysis of Pavement Section

Three combinations of pavement types are analysed, firstly standard bitumen versus silica-gel modified bitumen and aerogel-incorporated bituminous pavement followed by standard concrete surface pavement with aerogel-incorporated concrete pavement, both combinations show the same trend. Both strain and deformation increased in modified pavements, while stress and reaction force decrease. These, both, results are used to compare the behaviour of aerogel-incorporated bitumen pavement.

Different models of aerogel-incorporated bitumen pavement were created varying the aerogel percentages from 2.5 to 20%. Unlike the above models, the aerogel-incorporated bituminous pavement shows an increase in all the four results, namely stress, reaction force, strain and deformation. Increase in the values of strain and deformation was as expected showing that virgin aggregates show the better results, while the increase in values of stress and reaction force shows that aerogel can enhance the grain-to-grain transfer of load due to the proper filling of voids than standard pavement. From all the different percentages of aerogel used, 2.5% and 5% give the best result as the deformation was comparable to the standard bitumen pavement and even less than the 5% silica-gel modified bitumen pavement.

3.2 Aerogel-Incorporated Bituminous Pavement

Strain is increased compared to standard bitumen pavement by 13.44% and 21.59%, respectively, while strain in silica-gel pavement is increased by 62.26% compared to standard bitumen pavement. Thus, it can be inferred that the aerogel-incorporated pavements perform well compared to silica gel-incorporated pavement (Figs. 2 and 3).

Stress is increased compared to standard bitumen pavement by 9.1% and 14.86%, respectively. Stress in silica-gel pavement is increased by 24.42% compared to standard bitumen pavement. Thus, aerogel-incorporated pavements perform better compared to silica gel-incorporated pavement (Table 4).

4 Conclusions

This study is mainly focused on the use of silica aerogel in bituminous mix for a better road surface layer. The use of aerogel in road surfacing results in a more longer life span as the aerogel removes water from the pavement structure and provides thermal insulation, which results in improving the pavement properties.

From the FEM analysis, aerogel-incorporated bitumen pavement is found to give better performance compared to silica-gel-modified bitumen pavement. Even though the strength of AIBP is slightly reduced, it performs well like the standard flexible

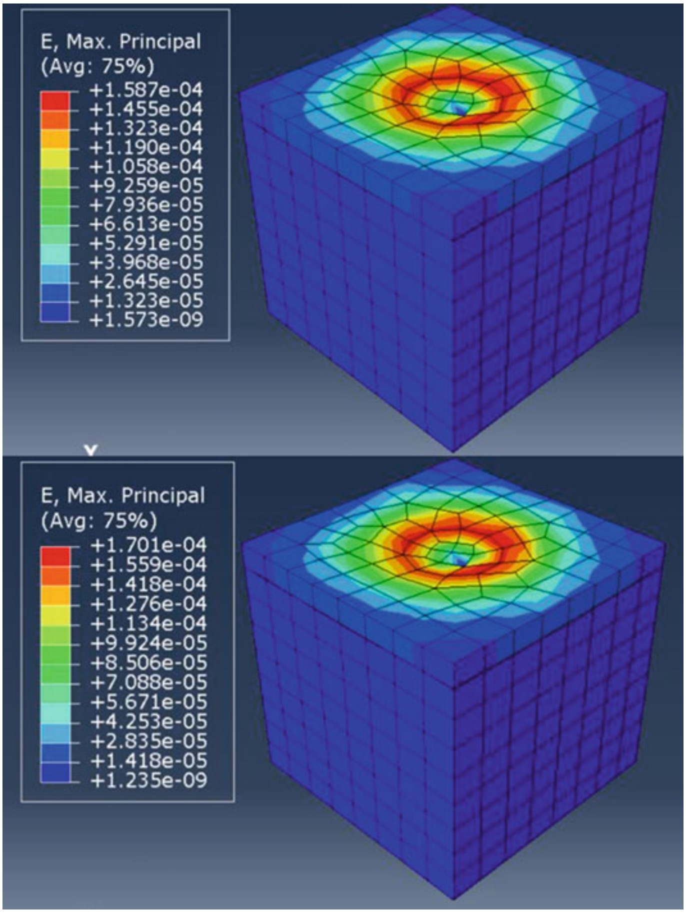

Fig. 2 Strain distribution between 2.5% (top) and 5% (bottom) aerogel pavement

pavement along with reduction of adverse effects of water and temperature changes due to the properties of aerogel. Further research must be performed to fully evaluate all the performance characteristics of aerogel-incorporated pavement.

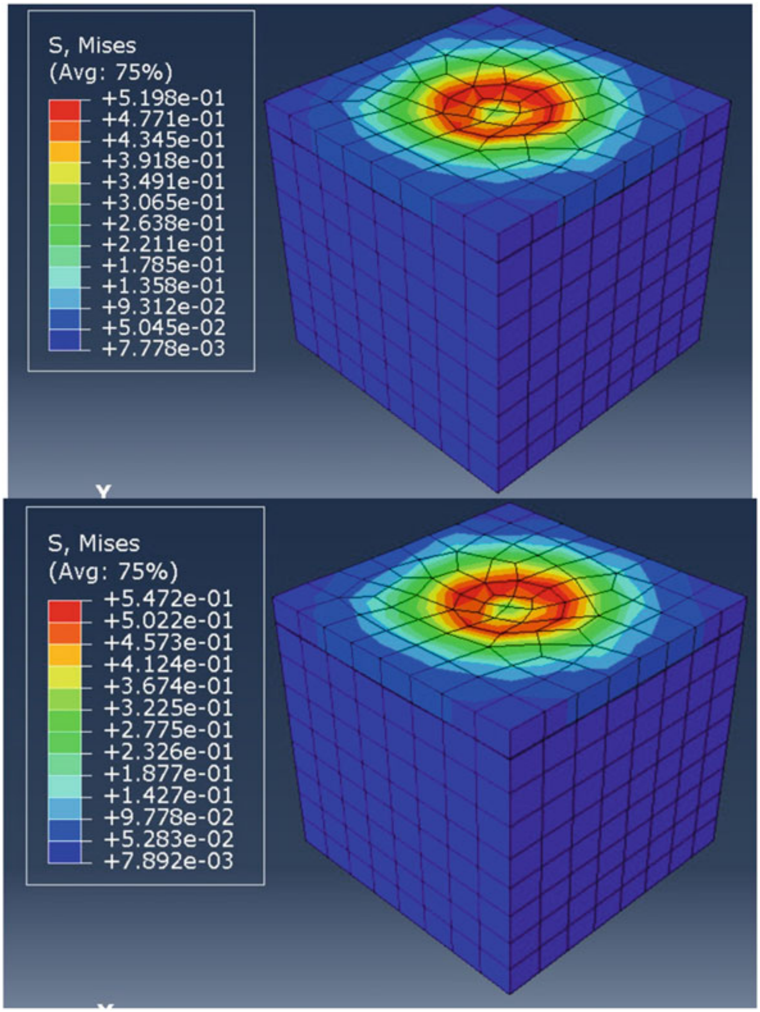

Fig. 3 Stress distribution between 2.5% (top) and 5% (bottom) aerogel pavement

Table 4 FEM analysis results (variation in characteristics in comparison with conventional pavements)

Materials	Strain variation (%)	Reaction force variation (%)	Stress variation (%)	Deformation variation (%)
SGBP	62.26	14.86	24.42	26.53
AICP	48.83	8.96	10.03	24.86
AIBP 2.5%	13.44	30.63	9.11	8.94
AIBP 5%	21.59	36.97	14.86	16.86
AIBP 7.5%	27.73	41.74	19.23	24.06
AIBP 10%	32.81	45.69	22.86	30.86

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Performance Assessment of Premix Carpet for Low-Volume Roads

Nishant Bhargava, Anjan Kumar Siddagangaiah, and Teiborlang Lyngdoh Ryntathiang

Abstract The paper presents the performance of premix carpet for low-volume roads in terms of structural capacity and functional properties. The structural capacity was evaluated using falling weight deflectometer (FWD). International roughness index (IRI) was used as a measure of the functional property. A total of 26 sections with a total road length of 110 km were selected for the study. At each test location, pavement composition was determined by excavating test pit along the edge of the shoulder. Subsequently, FWD test was conducted. Deflections were measured at 8 radial distances, ranging from 0 to 1500 mm. Deflection values ranged between 475 and 1129 µm below the loading plate and 30–224 µm at radial distance of 1500 mm. Using deflection readings, structural number (SN) and deflection bowl parameters including surface curvature index (SCI), base damage index (BDI), base curvature index (BCI) and AREA were calculated. Then, IRI was measured using Roughometer III device. IRI progression with time indicated that for a trigger value of 4.62 m/km, premix carpet roads would require maintenance after 41 months of service life. In addition, a good correlation between the deflection bowl parameters and IRI was observed. So, the limits of IRI were used to propose the recommended range of deflection bowl parameters. It was found that when the values of SCI, BDI and BCI increase to more than 313, 152 and 58, respectively, or AREA and SN reduce to less than 447 and 17, respectively, the pavement exhibits poor condition and would require rehabilitation.

Keywords Premix carpet · Low-volume road · Falling weight deflectometer · IRI

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1 Introduction

Premix carpet (PMC) is widely used in India for the construction of low-volume roads. It comprises of aggregates having nominal size 13.2 and 11.2 mm, which are pre-mined with binder and compacted to a thickness of 20 mm. Study on low-volume roads constructed with PMC in different states of India showed that its performance after 2–5 years of service life was satisfactory. In this study, the distresses that were targeted include bitumen-rich/dry surface, aggregate content less/excessive, surface texture, cracking, potholes, surface evenness, raveling and stripping. Minimal distresses were observed during the test period [1]. However, limited studies had addressed the issue related to development of maintenance plan for PMC.

In this regard, there is a need to assess the performance of low-volume roads. One of the most commonly adopted non-destructive test for structural evaluation of pavements is FWD. In this test, an impulse loading device in which a transient load simulating the actual traffic load is applied on the pavement surface. A fixed mass is dropped from a predetermined height on the system of springs placed over a circular loading plate. The corresponding peak load and peak deflection are measured at different radial distances. These deflections are then used to compute elastic moduli of pavement layers [2]. Different deflection bowl parameters including surface curvature index (SCI), base damage index (BDI), base curvature index (BCI) and AREA [3] have also been utilized to quantify the structural condition index of the pavement.

Another key parameter of low-volume road performance is its riding quality. International Roughness Index (IRI) is generally used to assess the riding quality. IRI is the measure of undulations along the pavement surface, i.e., total rise and fall with respect to true planar surface. Higher IRI often leads to poor riding quality, higher vehicle operation cost and increased risk of road accidents. One of the commonly used technique is accelerometer-based measurement. In such technique, the vertical acceleration is measured using sensor fitted at the rear axle of the test vehicle while the distance is measured using distance measure instrument.

However, studies on utilization of the deflection bowl parameters to determine the structural condition index rating of the pavement have been limited to roads constructed with surface layer of more than 40 mm thickness. In addition, limited studies have determined the IRI of low-volume roads. Hence, field investigations on structural capacity and functional properties could provide the much-needed information regarding the performance of low-volume roads.

1.1 *Objective*

The objective of the study is to assess the performance of premix carpet for low-volume roads in terms of structural capacity and functional properties. The following tasks were assigned for fulfilling the research objective.

- Identification of test sections and determination of pavement composition.
- Evaluation of structural capacity using FWD test and calculation of deflection bowl parameters from the deflection data.
- Assessment of pavement roughness in terms of IRI.
- Determination of association between structural condition index parameters and IRI.

2 Methodology

The research methodology adopted in the study was broadly divided into four phases:

- Phase 1: Selection of test sections

  The study was conducted in the two states of Northeast India including Assam and Meghalaya constructed under Pradhan Mantri Gram Sadak Yojana (PMGSY) initiative. Overall, 26 projects were selected in this study (project P1 to P26), out of which 17 projects were from Assam and 9 were from Meghalaya. The total length of all projects combined was 110 km, where 56 km length was from Assam and the remaining 54 km length was from Meghalaya. The terrain for Assam region was plain/rolling for all sections. For Meghalaya, the terrain was mountainous for all sections.
- Phase 2: Data collection

  For each section, at least one test pit having length and breadth of 0.6 m and depth upto subgrade were excavated, as shown in Fig. 1. The pavement composition was determined for each section.
- Phase 3: Structural strength assessment

  Falling weight deflectometer (FWD) was used to evaluate the structural integrity of the pavement. The test equipment is shown in Fig. 1. Given that the pavement condition for the test sections was good to fair, the FWD measurements were

(a) (b)

Fig. 1 Photos illustrating **a** Test pit and **b** FWD equipment conducted at the interval of less than 130 m. Deflection data were then used to calculate the deflection bowl parameters and structural number.

- Phase 4: Roughness measurement

  Pavement roughness was measured using Roughometer III. The results were computed in terms of International Roughness Index (IRI). Then, IRI was correlated with the structural number and deflection bowl parameters to benchmark the FWD deflection bowl parameter values for low-volume roads constructed with 20 mm thick PMC.

3 Experimental Protocols

3.1 Test Pit Excavation

In this study, one test pit was excavated for every 2 km of road section. Each test pit was excavated along the outer lane of the earthen shoulder such that the pavement layers were exposed. The dimension of the test pit was 0.6 m × 0.6 m with depth upto subgrade layer. The interface between two pavement layers was identified visually, and the data regarding number of layer and layer thickness were recorded [2].

3.2 Falling Weight Deflectometer

In this study, for the measurement of deflection using FWD, the plate load along with frame having displacement transducers was lowered to the test point on the pavement. The weight was raised to a predetermined height such that the load applied on the pavement was 40 kN. The load was then dropped, and the peak load and deformation were recorded. A total of eight displacement transducers placed at radial distances of 0, 200, 300, 450, 600, 900, 1200 and 1500 mm were used to measure deflection. At each test point, four readings were taken, out of which the first was the seating load and the next three readings were considered for further analysis [2].

3.3 International Roughness Index

Roughness of the pavement is a key performance indicator, especially for low-volume roads. It is a measure of surface irregularities, i.e., sum of rise and fall with respect to the selected datum. Higher roughness has an adverse effect on the riding quality along with increased vehicle operation cost. Pavement roughness is generally defined in terms of IRI. In this study, the ARRB Roughometer III, which is designed to provide roughness data for both sealed and unsealed roads, was used to assess the performance of the selected road sections. It uses a combination of wheel-mounted motion sensor and a distance input to measure the true longitudinal profile of the road. The longitudinal profile is used to calculate the IRI.

During field investigations, the various components of the Roughometer III were initially attached to the survey vehicle as per the equipment standards. After installation of all the components, the test vehicle was taken to the start of the project. The vehicle was allowed to accelerate and reach a constant speed at the start of the road section. The survey was carried out within the speed limit of 30–60 kmph. As soon as the vehicle was able to maintain a constant speed between the limit, the survey was started. All the data collected were stored in the hand-held device, which was processed using a desktop software supplied along with the equipment.

4 Results and Discussions

4.1 Pavement Composition

The pavement composition determined using the test pit excavation is shown in Fig. 2. It could be observed that the pavement composition for all section has premix carpet of 20 ± 2 mm thickness as wearing course. The base and sub-base layer comprised of water bound macadam (WBM) and granular sub-base (GSB), respectively. The thickness of WBM ranged from 75 to 150 mm whereas for GSB, the layer thickness varied from 100 to 200 mm.

4.2 Falling Weight Deflectometer

FWD was conducted on the road sections having good to fair conditions only. It was observed that some of the road sections were damaged while some sections were inaccessible to carry the FWD equipment. So, FWD measurements were conducted on 17 sections only. The range of deflection values observed in these 17 sections is shown in Table 1. It could be observed that the average deflection below the loading plate, i.e., D_0, was in the range of 475–1129 μm. Only two sections exhibited D_0 greater than 1000 μm whereas the remaining sections had D_0 less than 760 μm. This shows that the structural capacity of 15 out 17 sections is satisfactory.

Deflection bowl Parameters

The deflection readings from FWD test data were used to determine the deflection bowl parameters. In this study, four parameters including surface curvature index (SCI), base damage index (BDI), base curvature index (BCI) and AREA were used [3]. The formulas and the associated application are shown in Table 2. For all the formulas, D_i is the deflection at the radial distance i mm.

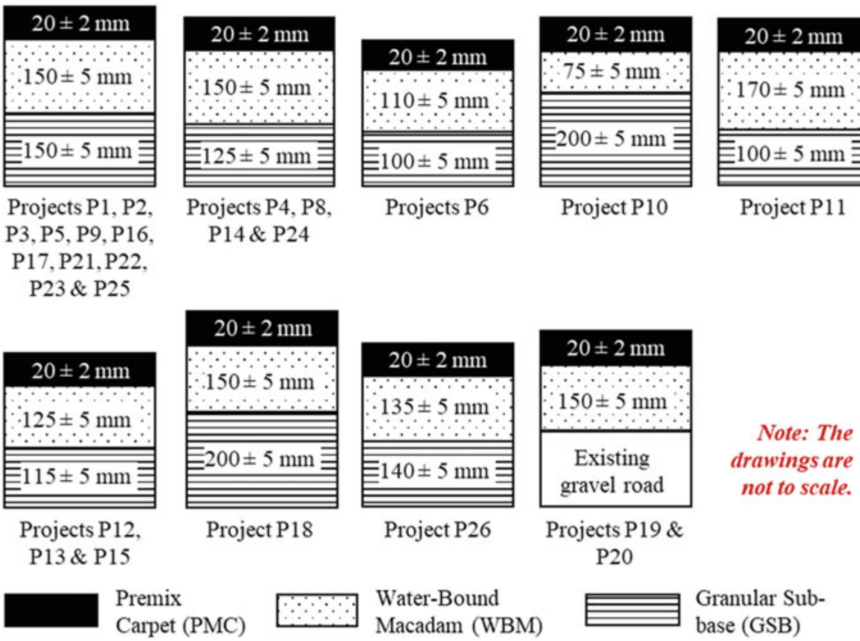

Fig. 2 Pavement composition determined after test pit excavation

Table 1 Deflection data from FWD test

Statistic	Deflections (µm)							
	D_0	D_{200}	D_{300}	D_{450}	D_{600}	D_{900}	D_{1200}	D_{1500}
Minimum	474.7	330.3	229.2	160.3	113.8	66.5	48.2	29.9
Maximum	1128.7	783.9	539.8	339.1	217.6	126.9	112.6	224.0

Table 2 Deflection bowl parameters [3]

Serial number	Parameter and relation	Indication
1	Surface curvature index SCI $= D_0 - D_{300}$	Indication of primarily the base layer structural condition
2	Base damage index BDI $= D_{300} - D_{600}$	Indication of the subbase and probably selected layer structural condition
3	Base curvature index BCI $= D_{600} - D_{900}$	Indication of the lower structural layers like the selected and the subgrade layers
4	AREA method $\mathrm{AREA}_{1200} = 4 + 6 \times \frac{D_{200}}{D_0} + 5 \times \frac{D_{300}}{D_0} + 6 \times \frac{D_{450}}{D_0} + 9 \times \frac{D_{600}}{D_0} + 18 \times \frac{D_{900}}{D_0} + 12 \times \frac{D_{1200}}{D_0}$	Normalized area under the deflection basin from centre of load to radial distance of 1200 mm from the test load

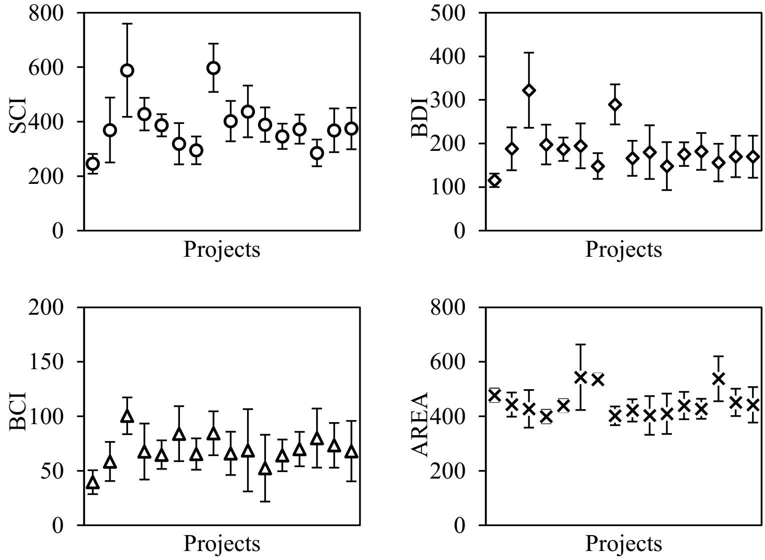

Fig. 3 Deflection bowl parameters for different projects

The results of the deflection bowl parameters are shown in Fig. 3. The range of values of deflection bowl parameters observed is as follows:

- SCI = 250–600 μm
- BDI = 115–320 μm
- BCI = 40–100 μm
- AREA = 400–550

Study on limits developed for Indian conditions suggested that the maximum permissible value for the warning condition for SCI, BDI and BCI is 600, 250 and 125, respectively [4]. The comparison of test results with the specification limits showed that almost all the test sections were having structural condition rating of good to warning. However, since the values were developed with pavement composition having 80–100 mm thick bituminous layer and 750 mm granular layer, it could be said that the limits should be relaxed even more for low-volume roads.

Structural Number

Structural Number (SN) is often used as an indicator for structural condition index. In this study, Eq. 1 was used to determine the SN for all test sections [5].

$$\mathrm{SN} = a_0 \mathrm{SIP}^{a_1} \mathrm{HP}^{a_2} \tag{1}$$

where SIP = Structural Index of Pavement = $D_0 - D_{1.5\mathrm{HP}} = D_0 - D_{450}$; HP = Total pavement thickness, mm; coefficients a_0, a_1, a_2 were assigned as 0.1165, −0.3248

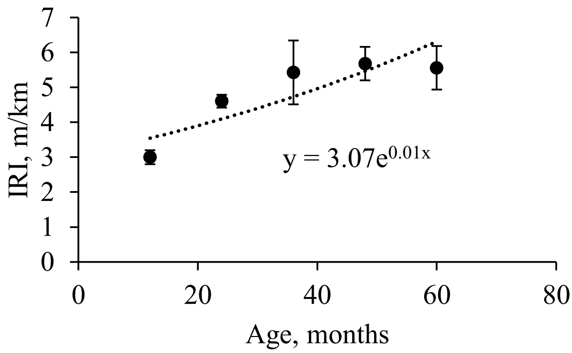

Fig. 4 IRI progression with time

and 0.8241, respectively [5]. For the 17 test sections, the SN varied within the range of 13.8–18.7, with the average value being 16.2.

4.3 International Roughness Index

In this study, the pavement roughness was evaluated using Roughometer III. The results were quantified in terms of IRI. Then, utilizing the data collected, the riding quality deterioration with time was determined. To develop the relation, the traffic conditions were considered similar on all project roads. The road sections were grouped into five categories 12 months old, 12–24 months, 24–36 months, 36–48 months and 48–60 months (or higher). Few outliers were removed to analyse the data considering traffic and soil variations and selection bias.

Results illustrated in Fig. 4 showed that there was an exponential increase in IRI with time. For a trigger level of 4.62 m/km of IRI (3500 mm/km Unevenness Index) [6], the model pointed out that low-volume roads constructed using premix carpet would require an improvement in riding quality at the end of 41 months (almost 3.5 years).

4.4 Relation Between Structural Condition Index and IRI

Parameters for structural condition index including SCI, BDI, BCI, AREA and SN were correlated with IRI. Results of the analysis are shown in Fig. 5. It could be observed that IRI increases exponentially with the increase in SCI, BDI and BCI. On the other hand, with the increase in AREA and SN, IRI decreases exponentially. This could be explained by the fact that lower values of SCI, BDI and BCI represent better structural condition of base, sub-base and subgrade layer. With the increase in the values of these parameters, the structural condition deteriorates leading to a

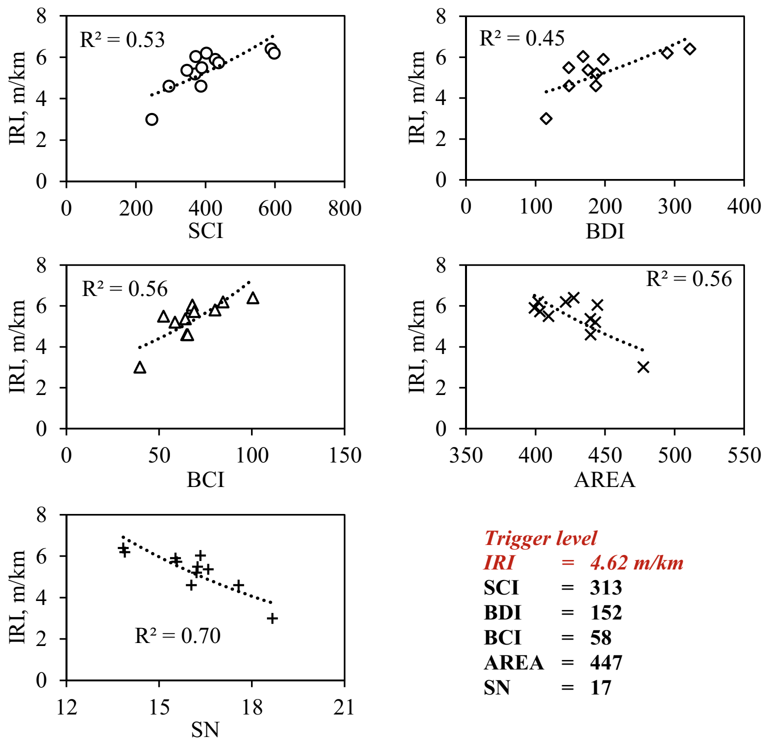

Fig. 5 Relation between IRI and structural condition index parameters

decline in the riding quality. Alternatively, higher SN represents a relatively better structural condition of the pavement. Hence, lower IRI was observed.

After establishing the relationship, the threshold limits of structural condition index parameters were determined. The trigger level of IRI was considered as 4.62 m/km, according to IRC recommendations [6]. Corresponding to IRI of 4.62 m/km, the threshold values of SCI, BDI, BCI, AREA and SN using the equation of the model. Result of the analysis is shown in Fig. 5. The proposed threshold limits could be utilized for effectively planning the maintenance and rehabilitation activities for low-volume roads.

It should be noted that these threshold limits were developed with the limited dataset. Different climatic conditions, traffic composition and material properties should be considered for generalizing the proposed model.

5 Conclusion

This study evaluated the performance of 20 mm thick premix carpet for low-volume roads. A total of 26 sections were selected, out of which 17 sections were tested for structural capacity and functional property. Structural condition index using deflection measurements from FWD test and roughness measurement using IRI were used to characterize the performance. The following conclusion could be drawn from this study.

- Structural condition index rating was within the range of sound to warning for most of the test sections. It shows that premix carpet performs well for low-volume roads.
- IRI increased exponentially with time. The model predictions pointed out that the low-volume roads would require a riding quality improvement after 3.5 years of service life, considering the trigger level of IRI as 4.62 m/km.
- The deflection bowl parameters and structural number showed a good correlation with IRI. Hence, the quality of subgrade, sub-base and base construction can influence the progression of riding quality with time in low-volume roads.
- Threshold limits of SCI, BDI, BCI, AREA and SN were proposed in the study, considering the trigger level of IRI as 4.62 m/km.

The outcomes of the study were established based on the limited dataset available. Further investigations could help in generalizing the established model and the threshold limits proposed in this study.

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Development of Resilient Modulus Model for the Bituminous Course

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Abstract As per IRC 37, the design of flexible pavement requires the accurate prediction of resilient modulus for bituminous concrete. This, determination of M_r requires specialized test equipment, which may not be available in many laboratories. Therefore, it would be rather reasonable to create a model that could estimate M_r from easy to estimate volumetric parameters and other parameters obtained from sample conventional tests. This study aims to investigate the effect of different binders (VG30, VG40, PMB and CRMB), binder contents, volumetric parameters (air voids) and temperature on Resilient Modulus (M_r) in Bituminous Concrete (BC) course and Dense Bituminous Macadam (DBM) course. The samples were compacted using the Marshall Compacter and subsequently the volumetric parameters were calculated. The Indirect Tensile Strength (ITS) test was conducted to calculate the ITS and Toughness. The Repeated Load Indirect Tension Test was conducted to find out the Resilient modulus (M_r) values for all mixes. The regression analysis was performed using the Microsoft Excel tool, and the relationship between resilient modulus, volumetric parameters and ITS was developed. Using the final model, Resilient Modulus of mixtures may be predicted from volumetric parameters and indirect tensile strength under comparable or different testing conditions.

Keywords Bituminous concrete (BC) course · Dense bituminous macadam (DBM) · Resilient modulus · Indirect tensile strength · Polymer modified bituminous (PMB) mixes · Crumb rubber modified bituminous (CRMB) mixes

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1 Introduction

The quality of a country's road transportation system has a significant effect on its economic and social growth. The majority of India's road network is made up of flexible pavements [1]. The design of flexible pavement requires accurate prediction of resilient modulus of all the layers of pavement. If resilient modulus predictions are inaccurate, pavement will fail before it is intended to, costing a lot of money to repair [2]. Under repeated loading tests, the resilient modulus is defined as the ratio of deviator stress to recoverable elastic strain [3]. The temperature environment to which flexible pavements are subjected has a significant impact on their functional and structural performance. Temperature has been demonstrated to influence the strength and deformation properties of asphalt concrete mixtures, affecting the resilience modulus of the asphalt layers [4]. The asphalt type (unmodified and modified asphalt) and aggregate gradation also affect the resilient modulus. Resilient modulus changed as a result of the asphalt binder and aggregate gradation changes [5].

Considering the issues connected with cracking, the tensile characteristics of asphalt mix are of excessive interest to pavement engineers. As a result, the tensile strength of asphalt concrete mix is critical for asphalt paving. The indirect tensile strength test is performed to evaluate the bituminous mix's tensile qualities [6, 7]. The comparative study of ITS test was conducted on modified asphalt mixes [8].

The harshness of the highway system's problems has been compounded by increased traffic loads, necessitating the improvement in the quality of existing bituminous materials. Polymer binder modification has been offered as a solution to improve the physical and rheological behaviours of bituminous binder [9]. The polymer modified asphalt concrete has a greater resilience modulus than the unmodified asphalt concrete. Polymer modified asphalt concrete gives higher strength and durability than the unmodified asphalt concrete [10]. Crumb Rubber Modified Bitumen (CRMB) is also one of the option other than the polymer modified bituminous mixes. Crumb rubber is a recycled rubber made mostly from natural, synthetic, and carbon black rubbers found in automobile trash tyres [11]. Crumb rubber enhances the viscosity, softness point, and temperature susceptibility of modified bitumen, as well as its resistance to deformation at increasing pavement temperatures [12].

Bituminous mix design is a difficult procedure that needs precise proportions of aggregate and asphalt binder in order to meet particular volumetric and mechanical criteria [13]. A good performance-volumetric relationship (PVR) can help mixes work more effectively and may be employed in future quality design mixes. Voids in mineral aggregate (VMA) are a critical design parameter in the current Superpave mix design technique that relates hot-mix-asphalt (HMA) mix qualities to field performance. The ability to achieve the proper VMA early in the mix-design process has a considerable impact on design time and effort, as well as the ability to generate a cost-effective and appropriate design with acceptable field performance. Air voids is also one of the volumetric parameters, which affects the performance of the pavement [14]. To prevent binder ageing, permeability, and resultant stripping difficulties, asphalt mixes should contain the least possible air spaces [15]. The addition

Table 1 Resilient modulus predictive models

Model name	Equation	
ANN model	$E_p = \frac{1}{2}\sum_j \left(d_{pj} - O_{pj}\right)$ $E = \sum_p E_p$	[17]
Resilient modulus for wearing course	$M_r = -1297074.90 - 165520594\,T - 52354.064\,P_S$ $-15094.311\,A_V + 301011.367\,\eta + 2306479.544\,SA$	[18]
Model for using waste PET plastic modified bituminous mixes	$M_r = 1.08102 \; * \; (ITS)^{1.124} - 465 * T$	[19]
Model for WMA	$M_r(psi) = 915411 + 19343\,(AC) - 7928.4\,(TT)$ $-282.24\,(LD) + 39540\,(\text{Filler Type}) - 89014\,(A_v)$	[20]

of polymer and crumb rubber in unmodified bitumen gives more accurate results for the percentage of voids in mineral aggregate and percentage voids field with the asphalt [16]. Various regression models that are already developed to find out resilient modulus value are listed in Table 1.

2 Objectives of the Study

The primary objective of the study was to a resilient modulus model that relates M_r to the volumetric parameters of bituminous mixtures.

3 Experimental Design and Procedure

3.1 Materials

In this study, the experimental design included the utilization of aggregate, four different types of binders (VG30, VG40, PMB and CRMB) and stone dust as filler material. For the preparation of Marshall samples, BC-1 and DBM-1 gradation were used as per MoRTH specification. The sample was prepared using the mid-point gradation. The aggregate's engineering properties were measured and compared to the respective specifications given in Table 2.

The engineering properties of the binders are measured and compared to the corresponding standards. The testing was performed on all four binders as per the procedures described in the relevant codes, and the findings are shown in Tables 3 and 4.

Table 2 Engineering properties of the aggregate

Characteristics	Obtained	Limit	Specification
Aggregate impact value (AIV)	11.83%	Max 24%	IS:2386 (Part IV)
Combined flakiness and elongation indices	30.29%	Max 35%	IS:2386 (Part I)
Specific gravity	2.77	–	ASTHO T84 & T85
Water absorption	0.2%	Max 2%	IS:2386 (Part III)

Table 3 Engineering properties of VG30 and VG40 grade bitumen

Characteristics	VG30		VG40		Specification
	Obtained	Limit	Obtained	Limit	
Specific gravity	1.01	–	1.03	–	IS:1202
Penetration at 25 °C, 100 g, 5 s, 0.1 mm, min	52	45	49	35	IS:1203
Softening point, °C, min	51	47	53.2	50	IS:1205
Kinematic viscosity, 135 °C, poise	445	350	525	400	IS:1206 (Part 1)
Failure temperature °C	79	–	88	–	AASHTO T315
G*/sinδ, kPa	1.00	≥1	1.15	≥1	AASHTO T315

Table 4 Engineering properties of PMB and CRMB binders

Characteristics	PMB		CRMB		Specification
	Obtained	Limit	Obtained	Limit	
Specific gravity	1.03	–	1.01	–	IS:1202
Penetration at 25 °C, 100 g, 5 s, 0.1 mm, min	33.3	30–50	58.5	50	IS:1203
Softening point, °C, min	81.3	60	67	60	IS:1205
Viscosity, 150 °C, poise	4	3–9	5.25	3–9	IS:1206 (Part 1)
Failure temperature °C	94	–	94	–	AASHTO T315
G*/sinδ, kPa	1.32	≥1	1.51	≥1	AASHTO T315

3.2 Experimental Program

This experimental study includes four types of binder (VG30, VG40, PMB and CRMB). The aggregates and stone dust were locally sourced. To verify that the materials conform to the minimal standards specified in the codal regulation, physical properties of all binders and aggregates were verified. HMA specimens were prepared using four different binders and were compacted using a Marshall compactor. BC-1 (at 4.5 and 5.5% binder contents) and DBM-1 (at 4 and 6% binder contents) gradations were utilized to prepare the test samples. Please note that the binder contents used in this study are under and above the anticipated OBC, this was done in order to look at bitumen content sensitivity to M_r so that wider differences in properties can be achieved. Following the preparation of the specimen, density-void analysis was performed to determine all volumetric characteristics. Following the volumetric investigations, an indirect tensile strength (ITS) test and a resilient modulus (M_r) test were performed at 25 and 35 °C. The main objective of this research is to develop a model to predict the resilient modulus using ITS results and the mixture volumetric properties.

4 Results and Discussions

4.1 Viscosity Test

Rotational viscosity test was performed using a Brookfield viscometer in accordance with ASTM D 4402. This test technique is used to assess the apparent viscosity of asphalt binders at handling, mixing or application temperatures. The results of the viscosity test are shown in Table 5.

4.2 Mixing and Compaction Temperatures

According to MS-2, equiviscous temperature ranges were employed in asphalt mix design procedures for laboratory mixing and compaction temperatures. Equiviscous mixing and compaction temperatures are utilized in laboratory mix design approaches to normalize the influence of asphalt binder stiffness on mixture volumetric characteristics. Using that viscosity value, the mixing and compaction temperatures for all four binder mixes were calculated using the equiviscous principle. Table 6 shows the mixing and compaction temperatures of all binder mixtures.

Table 5 Viscosity of binders

Binder	Viscosity (Pa.S)		
	135 °C	150 °C	170 °C
VG30	0.450	0.100	–
VG40	0.525	0.225	0.075
PMB	0.825	0.400	0.150
CRMB	1.050	0.525	0.225

Table 6 Mixing and compaction temperature of mix

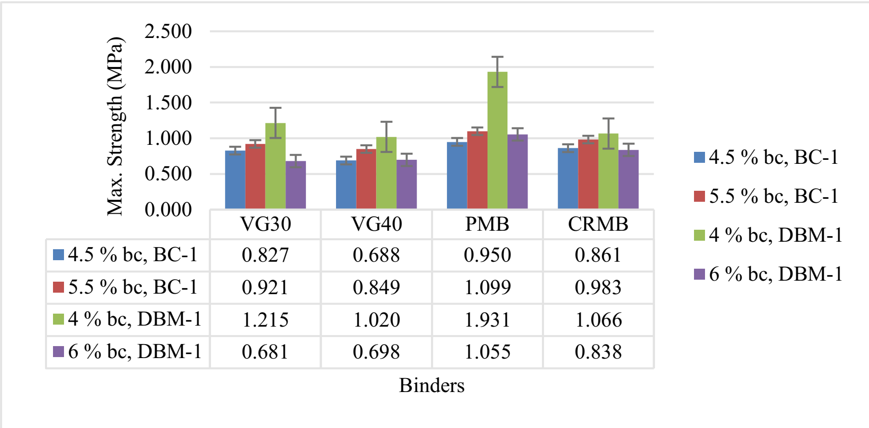
Binder	Mixing temperature (°C)	Compaction temperature (°C)
VG30	152–155	142–147
VG40	158–161	148–153
PMB	166–168	159–163
CRMB	169.5–171	164–167

4.3 Volumetric Study

Asphalt mixtures' volumetric characteristics are utilized to attain a satisfactory performance. The volumetric parameters were calculated using the MS-2 equations and the results are shown in Table 7.

Table 7 Volumetric properties of Mix

Gradation and binder content	Volumetric parameters	VG30	VG40	PMB	CRMB
BC-1 (5.5% binder content)	(P_a)	2.38	2.77	2.10	2.10
	VMA (%)	11.98	9.98	9.77	9.64
	VFA (%)	80.16	72.27	78.51	78.23
	G_{se}	2.91	2.92	2.90	2.91
	P_{ba} (%)	2.76	2.84	2.64	2.73
	P_{be} (%)	2.89	2.82	3.01	2.92
BC-1 (4.5% binder content)	(P_a)	3.07	3.33	2.56	2.08
	VMA (%)	11.29	11.14	10.54	10.82
	VFA (%)	72.82	70.17	75.78	80.92
	G_{se}	2.87	2.88	2.88	2.86
	P_{ba} (%)	1.30	1.44	1.40	1.14
	P_{be} (%)	3.26	3.12	3.17	3.41
DBM-1 (4% binder content)	(P_a)	3.71	3.64	3.50	3.20
	VMA (%)	11.63	12.33	11.20	11.71
	VFA (%)	68.21	70.69	73.36	72.70
	G_{se}	2.84	2.81	2.83	2.85
	P_{ba} (%)	1.82	1.44	1.70	2.02
	P_{be} (%)	2.25	2.62	2.36	2.07
DBM-1 (6% binder content)	(P_a)	1.44	1.88	1.27	1.09
	VMA (%)	11.10	11.47	11.29	10.67
	VFA (%)	87.29	83.61	83.92	90.05
	G_{se}	2.78	2.78	1.17	2.80
	P_{ba} (%)	1.12	1.10	1.17	1.35
	P_{be} (%)	2.93	2.95	2.88	2.70



	VG30	VG40	PMB	CRMB
4.5 % bc, BC-1	0.827	0.688	0.950	0.861
5.5 % bc, BC-1	0.921	0.849	1.099	0.983
4 % bc, DBM-1	1.215	1.020	1.931	1.066
6 % bc, DBM-1	0.681	0.698	1.055	0.838

Fig. 1 Indirect tensile strength of mix

4.4 Indirect Tensile Strength (ITS)

In conjunction with laboratory mix design testing, the Indirect Tensile Strength (ITS) test may be performed to assess the relative quality of bituminous mixes and predict the possibility of rutting or cracking. The ASTM D 6931-12 guideline was followed for performing the indirect tensile strength (ITS) test. The results of the indirect tensile strength test are shown below. The results demonstrate that PMB mixes have the highest indirect tensile strength when compared to the other binder mixes at both the binder content and both the gradation BC-1 and DBM-1, as shown in Fig. 1.

The peak load and corresponding deformation values were provided by ITS test. The stress and strain were determined using the load and deformation data. Toughness was then estimated from the stress–strain curves. Toughness is generally defined as the area under the stress–strain curve, and it also represents the energy absorption capacity. Figure 2 illustrates the toughness values of all binder mixtures using the BC-1 and DBM-1 gradations. The results reveal that PMB mixes have the highest toughness value when compared to other binder mixes at both binder contents and considering both BC-1 and DBM-1 gradations.

4.5 Resilient Modulus (M_r)

Resilient modulus is used in the evaluation of material quality and as input for pavement design, evaluation and analysis. Repeated load indirect tension test is used to find the resilient modulus value. Resilient modulus M_r test has been conducted as per the ASTM D 4123 guideline. At 25 and 35 °C, 100 cycles of load repetition

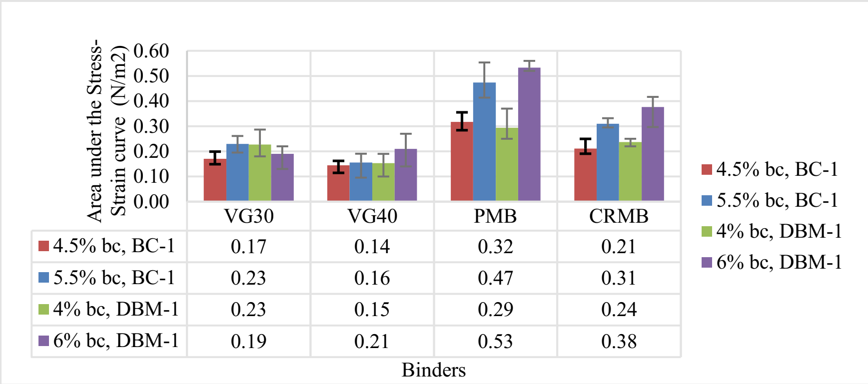


Fig. 2 Area under the stress–strain curve

are applied. In the resilient modulus test, 10% of the ITS pick load was used as cyclic load and 4% of cyclic load was used as contact load. Figures 3 and 4 show the resilient modulus results of the various mixtures.

The results of the resilient modulus test demonstrate that the PMB mixes had the highest resilient modulus at both temperatures. As the binder content falls, so does the resilient modulus value for BC-1 mixtures, however, opposite trend is seen for DBM mixtures. Both the BC-1 and DBM-1 gradations showed a similar tendency. Temperature is another key and significant component for the resilient modulus test. When the temperature increases, the value of the resilient modulus falls.

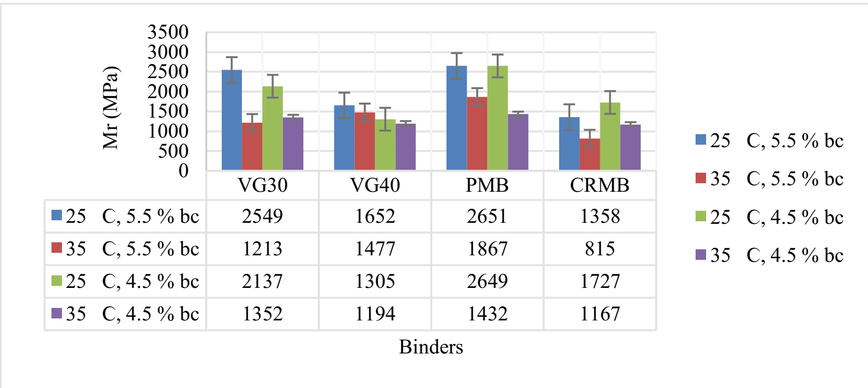


Fig. 3 M_r for BC-1 gradation

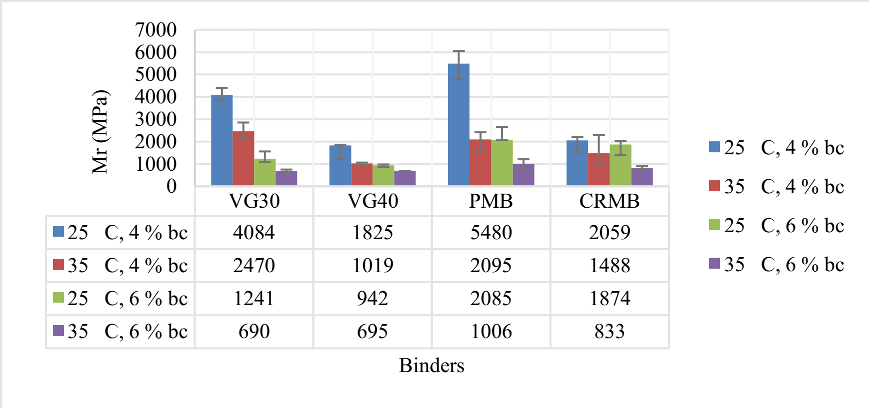


Fig. 4 M_r for DBM-1 gradation

4.6 Regression Analysis

The data for the regression analysis was acquired using volumetric investigations, Indirect Tensile Strength test, and Resilient Modulus test. Some of the parameters considered in the resilient modulus model are Toughness, Indirect Tensile Strength, Binder Content, Test Temperature, and Percentage Air Voids. Using the acquired data set, the Resilient Modulus Model for Bituminous Mixtures was developed by regression analysis using the Microsoft Excel tool, as shown below.

$$M_r = 2980.99 - 395.91\ (S) + 2124.14\ (ITS) \\ - 123.47\ (bc) - 76.43\ (T) - 155.985\ (A_v)$$

where

M_r = Resilient Modulus (MPa)

S = Toughness value (MPa)

ITS = Indirect Tensile Strength (MPa)

bc = Binder Content (%)

T = Temperature (°C)

A_v = Air Voids (%)

The resilient modulus model indicates that as the indirect tensile strength increases, so does the resilient modulus, however when the other factors such as toughness, binder content, temperature, and air voids increase, the resilient modulus decreases. The measured versus predicted resilient modulus, as well as the prediction accuracy of the analysed models for testing data sets, are shown in Fig. 5. The

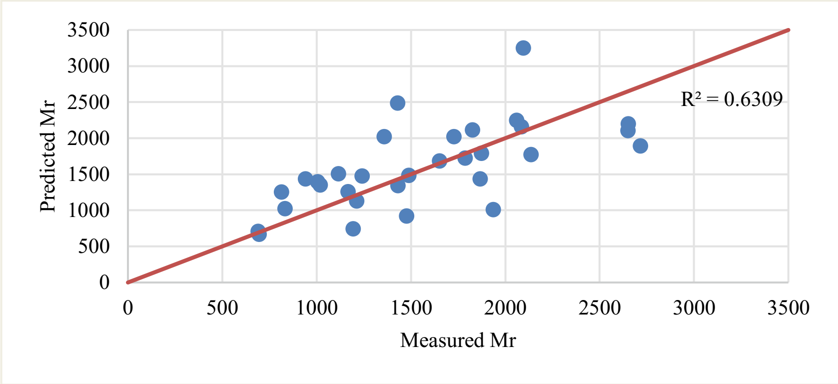


Fig. 5 Calculated versus predicted M_r

majority of the points are close to the line of equality indicates that the model produces appropriate results and may be used to calculate the resilient modulus value.

5 Conclusions

The resilient modulus is an important consideration when designing flexible pavements. The primary objective of this study was to develop a predictive resilient modulus model for the bituminous mixture. The laboratory experiments comprised of the ITS test and the M_r test, which were performed at different test temperatures (25 and 35 °C) considering the BC-1 (4.5 and 5.5% bc) and DBM-1 (4 and 6% bc) gradations. The following conclusions are derived based on the test data obtained in the laboratory and analysis performed.

- PMB mix has the highest ITS value.
- PMB mix has the highest toughness in both gradations and at both binder contents. In both the BC-1 and DBM-1 gradations, similar trend is observed in toughness as the binder content was reduced.
- The PMB mix of BC-1 gradation had the highest Resilient Modulus (M_r) values at 25 and 35 °C. Whereas PMB mix of DBM-1 gradation had the highest Resilient Modulus values at both temperatures 25 and 35 °C when binder concentration was 6%, when binder content was reduced to 4%, PMB mix had the highest Resilient Modulus values at 25 °C and VG30 mix had the highest Resilient Modulus values at 35 °C. The value of the resilient modulus falls as the temperature increases.
- The regression analysis revealed that temperature and indirect tensile strength had a significant influence on resilient modulus value.

The developed resilient modulus model can be used to predict resilient modulus values from ITS testing and volumetric data, which will be useful for laboratories without specialized equipment to predict reasonably the values of resilient modulus.

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Forensic Investigations for Failure of Flexible Pavements: A Case Study

Abhishek Mittal and Amit Kumar

Abstract Flexible pavements undergo some form of distress during the design life. So, it becomes necessary to carry out forensic investigations to understand the causes of distresses/failures and develop an optimal rehabilitation strategy. The present study describes an investigation undertaken to determine the probable causes of distress in flexible pavements in an industrial township.

The studies conducted include visual surface condition assessment, classified traffic volume counts, axle load studies, pavement deflection studies using Benkelman beam, bituminous core cutting, laboratory characterization of samples from all pavement layers, and full-depth pit cutting at some locations on the flexible pavements.

During the condition assessment, it was observed that the condition of the pavement was poor on the entire stretch with heavy ravelling and potholes. From field and laboratory studies, it was inferred that all the pavement layers (including subgrade) were poorly compacted and did not meet the specified requirement as per the Indian specifications. Overall, the quality of the materials and workmanship was of poor quality. The pavement was heavily overloaded by commercial vehicles and a high vehicle damage factor (VDF) value was obtained. The deflection studies indicated that the existing pavement thickness was inadequate and need to be strengthened. Accordingly, suitable thickness of overlay was recommended to strengthen the pavement.

Keeping in view the structural and surface condition of the existing pavement, traffic loads and damaging factors, the short-term and long-term corrective and rehabilitation measures were suggested.

Keywords Failure investigations · VDF · Axle load · Benkelman beam

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1 Introduction

Faridabad is a major industrial hub of Haryana. Haryana State Industrial and Infrastructure Development Corporation Ltd. (HSIIDC) is maintaining the roads in the industrial Model Township (IMT) area in Faridabad. The roads cater to heavy volumes of loaded trucks. The roads in the area were constructed during the period 2011 to 2016. The roads are in bad condition and need to be improved. Based on the request of HSIIDC, this study for the evaluation of the roads was taken up [1].

2 Objectives and Scope of the Study

The main objective of this project is to carry out the investigation to study the causes of failure of the study road. The study included the evaluation of general road condition and structural condition in order to assess the deficiencies and to suggest suitable remedial measures.

To achieve the above objectives, the scope of the study included the following activities:

a) Field investigations

- General condition assessment by visual survey
- Structural evaluation of pavement by Benkelman Beam deflection measurements
- Test pit observations and bituminous core sampling
- Extraction of material samples from various pavement layers
- Classified traffic volume studies and axle load studies

b) Laboratory investigations by studying engineering properties of extracted pavement materials/mixes
c) A comprehensive analysis, inferences and suggesting suitable remedial measures.

3 Field Studies

The field investigation work was undertaken with a view to assess the quality of pavement layers and to carryout structural evaluation of the pavement, so that reasons of pavement failure could be made out.

Table 1 General condition of roads at IMT Faridabad

Serial number	Road identification	Pavement surface condition w.r.t distress				
		General road condition	Cracking	Ravelling	Rutting	Potholes
1	CETP plant to Mohna road (90 wide road)	Poor	At isolated locations	At isolated locations	NIL	At isolated locations
2	Mohna distributary to entry at proposed bridge (90 m wide road)	Poor	At isolated locations	At isolated locations	NIL	At isolated locations
3	45 m wide road	Poor	At isolated locations	At isolated locations	NIL	At isolated locations

3.1 Visual Surface Condition Assessment

The visual condition data were collected by observing different forms of distress on the road sections. It was observed that the roads have developed severe cracking and many potholes were observed. The summary of road condition assessment is given in Table 1.

3.2 Benkelman Beam Deflection Measurements

To assess the structural condition of the road stretch under investigation, non-destructive method of Benkelman Beam rebound deflection has been used. Deflection measurement points covering the entire reach and representative of different conditions of study roads were selected for this study. The Benkelman Beam deflections were measured at points staggered at 50 m with a standard truck having rear axle load of 8.16 tonnes and tyre pressure of 5.6 kg/cm^2. The measurements were taken as per CGRA procedure laid down in IRC:81 [2].

3.3 Test Pit Observations

Based upon the condition survey of the study area, the locations of the representative test pits were decided. A total of 11 test pits were identified on the various study roads. The test pits, measuring about 1.0 m by 1.0 m, were dug open upto the subgrade level at these identified locations to study the condition of constituent layers. Samples from constituent layers were collected for further evaluation in the laboratory from all these test pits. The thickness of each constituent layer was measured in all the test pits and average value was taken. The measurements of in-situ densities of Wet Mix Macadam (WMM), Granular Sub-base (GSB) and subgrade soil were done by sand replacement and core cutter method.

3.4 Coring of Bituminous Layers (Surface and Binder Course)

The bituminous layers of wearing course and binder course were examined by coring from the pavement to get samples of these mixes for evaluating their properties in the laboratory. Cores of bituminous mixes, of 100 mm diameter, were taken out using a core cutting machine at many different locations on the study roads. The core locations were staggered so as to take the cores from all the lanes of the roads. The cores were further separated out in the laboratory by slicing using automatic slicer for further characterization.

3.5 Estimation of Design Traffic

For the purpose of design of strengthening/rehabilitation requirements, traffic, both in terms of both volume and axle loads, is an important parameter. Hence, present and the future projected traffic are considered for analysis in deriving the estimated load applications. Classified traffic counts and the 'Vehicle Damage Factor' are the requisites for traffic load estimation.

Classified Traffic Volume Counts

Classified traffic volume data were collected for 3 days (72 h) round the clock at the identified location on the study roads, for both the directions, covering all categories of vehicles. The traffic data were collected manually by trained enumerators under the continuous surveillance of CRRI team. The data on classified traffic counts for both directions of traffic are given in Table 2. The total number of commercial vehicles only, for each direction of travel, has been considered for overlay design.

Axle Load Studies (Using Static Weigh Pads)

The axle load studies were carried out near the main entry gate, for both the directions, for 24 h round-the-clock. The data obtained were processed to get the Vehicle Damage Factor (VDF), as given in subsequent section. VDF is a multiplier to convert the number of commercial vehicles of different axle loads and axle configurations to the number of standard axle load repetitions [3]. It is defined as the equivalent number

Table 2 Classified traffic volume data

Direction of traffic	Towards CETP plant		Towards main entry gate	
Vehicle type	Number	Percent composition of the total volume (%)	Number	Percent composition of the total volume (%)
Car/jeep	1844	41.9	2022	45.1
Light passenger vehicles/ mini-buses	4	0.1	7	0.2
Standard bus	14	0.3	25	0.6
Light commercial vehicles	**724**	16.5	**877**	19.5
Two axle trucks	**791**	18.0	**813**	18.1
Three axle trucks	**426**	9.7	**298**	6.6
Multi-axle trucks	**384**	8.7	**275**	6.1
Tractor/trailers	**214**	4.9	**169**	3.8
Total commercial vehicles for design (LCVs + trucks)	**2539**	**100**	**2432**	**100**

Note The percentage (rounded-off values) indicated is in terms of number of vehicles in a particular category to the total number of all motorized vehicles

Table 3 VDF obtained from axle load study

Direction of traffic		Towards CETP plant	Towards main entry gate
Serial number	Vehicle type	Individual VDF	Individual VDF
1	Light commercial vehicles (LCV)	0.71	0.59
2	Two axle trucks	2.52	2.15
3	Three axle trucks	3.80	2.91
4	Multi-axle trucks	17.8	12.7

of standard axles per commercial vehicle. The VDF varies with the vehicle axle configuration, axle loading, terrain, type of road and from region to region (Table 3).

4 Laboratory Characterization of the Collected Materials

The laboratory evaluation included testing for engineering properties of in-situ pavement materials. Laboratory evaluation of the materials collected from field aimed at evaluating them for their compliance with the required properties for a good performance. Various pavement materials viz., subgrade soil, Granular Sub-base (GSB), Wet Mix Macadam (WMM) base, Bituminous cores and chunk samples, collected from 11 pits, were subjected to detailed laboratory investigations.

4.1 Subgrade

The subgrade soil was found to be poorly compacted in all the test pits, except pit number 3. The relative compaction values were found to be in the range varying from 91.5 to 99.3%, as against the specified value of 97% as per MoRT&H [4] specifications.

4.2 Granular Materials (GSB and WMM)

The gradation of GSB was found to be meeting the specified gradation requirements, except for pit number 3, 4 and 8. However, the GSB layer was found to be poorly compacted as it was found to be not meeting the relative compaction requirements of 98% (except in pit no. 2) as per MoRT&H specifications.

The gradation of WMM used in granular base was found to be meeting the specified requirements, except for pit number 2. However, the WMM layer was found to be poorly compacted as it was found to be not meeting the relative compaction requirements of 98% as per MoRT&H specifications.

4.3 Characteristics of Bituminous Mixes

The samples of bituminous layers collected from the test pits and the core samples extracted were subjected to detailed laboratory testing. The combined cores extracted were sliced for top and lower bituminous layer and separated out in the laboratory, using a diamond cutter without disturbing their cylindrical shapes. The samples were tested for thickness, bulk density and percent bitumen. The extracted washed aggregates were also tested for various physical properties as per MoRT&H specifications.

The density acceptance criterion given by MoRT&H for bituminous mixes is given as below:

$$\text{Mean Density} \geq \text{Specified density} + \left[1.65 - \frac{1.65}{(\text{No. of Samples})^{0.5}}\right] \times \text{Standard Deviation}$$

Here, the specified density is taken as 92% of the theoretical maximum specific gravity (G_{mm}) (Tables 4 and 5).

The binder content was found to be less than the specified requirements for all the core and chunk samples taken.

Table 4 Results for acceptance of density values

Serial number	Road type identification	Mean density	Require density as per acceptance criterion	Remarks
1	Samples of BC (main carriageway 90 m roads)	2.196	2.291	**Not acceptable**
2	Samples of BC (main carriageway 45 m roads)	2.226	2.294	
3	Samples of SDBC (Service road)	2.215	2.273	

Table 5 Results for acceptance of thickness values

Serial number	Road type identification	Mean thickness	Required thickness as per design	Remarks
1	Samples of BC (main carriageway 90 m roads)	33.9	40	**Not acceptable**
2	Samples of DBM (main carriageway 90 m roads)	71.7	80	
3	Samples of BC (main carriageway 45 m roads)	32.6	40	
4	Samples of DBM (main carriageway 45 m roads)	63.8	80	
5	Samples of SDBC (Service road)	28.8	25	**Acceptable**
6	Samples of DBM (Service road)	53.2	70	**Not acceptable**

5 Probable Causes for Development of Distress

The probable causes for development of distress/defects derived based on the experiences gained at field, observations made during the test pitting and coring as well as from the laboratory and field data/ results obtained through the investigation done on the project road, can be summarized as follows:

(i) Overloading by commercial vehicles (trucks and multi-axles) as indicated by high VDF and number of commercial vehicles.
(ii) Materials used in the various component layers not meeting the specified requirements at many locations.

(iii) Inadequate compaction of the pavement layers as indicated by the layers not meeting the density requirements, which has rendered the pavement prone to permanent deformation under the action of loaded commercial vehicles.
(iv) Grossly inadequate structural capacity of the existing pavement and foundation for the traffic loads.

6 Proposed Short-Term and Long-Term Measures

The recommendations have been proposed for immediate action (short-term measures) and strengthening through overlay (long-term measures).

6.1 Short-Term Measures (Till the Time Major Strengthening is Carried Out)

- All existing surface defects like cracks, potholes, depressions/undulations/deformations etc. shall be properly treated/filled up.
- Proper sealing of cracks and filling up of undulations/depressions, wherever and whenever required, depending on the extent and severity of distress must be continued, as soon as they occur, on urgent/priority basis to minimise further progression of deterioration.
- Stipulated routine and periodic maintenance to be carried out at regular intervals.

6.2 Long-Term Measures (Strengthening Through Overlay)

The requirement of the flexible overlay to be provided for improving the structural adequacy of the existing roads has been worked out for 10 years design life, with the specification options as given below (Table 6).

It is also recommended that the road stretch is recommended to be re-evaluated for any further structural and functional improvement requirements at the end of 5 years. This is recommended considering the fact as the industrial activity expands in the study area, the traffic is also expected to increase, both in terms of number and loading. So, to avoid any excessive damage to the roads in the area, it would be good to re-evaluate the roads for structural and functional improvement requirements.

Table 6 Overlay thickness proposed for design life of 10 years

Serial number	Description of the road	Cumulative standard axles	Recommended overlay thickness
1	CETP plant to Mohna road (Direction: towards Mohna road)	18.7	**70 mm DBM + 50 mm SMA** or 70 mm DBM + 50 mm BC
2	CETP plant to Mohna road (Direction: towards CETP plant)	12.1	**65 mm DBM + 50 mm SMA** or 65 mm DBM + 50 mm BC
3	Mohna distributary to entry at proposed bridge (Direction: towards Mohna distributary)	18.7	**50 mm DBM + 40 mm SMA** or 50 mm DBM + 40 mm BC
4	Mohna distributary to entry at proposed bridge (Direction: towards entry at proposed bridge)	12.1	**50 mm DBM + 40 mm SMA** or 50 mm DBM + 40 mm BC
5	45 m wide roads	18.7	**75 mm DBM + 50 mm SMA** or 75 mm DBM + 50 mm BC

Note SMA has been recommended as a better option being a stone aggregate strength derived, bituminous rich specification with a fibre content and better rut resistance. The fresh rehabilitation layers need to be produced using modified bitumen/VG-40 to derive the enhanced life of the pavement

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Effect of Subgrade Stabilization on Pavement Design: Material Optimization and Economic Impacts

Sudeshna Purkayastha, Ritu Raj Patel, Veena Venudharan, and Ajitkumar Vadakkoot

Abstract The objective of this research was to evaluate the effect of subgrade stabilization on the flexible pavement design. The stabilizer used for the study was naturally derived mineral stabilizer, making it a sustainable alternative to the currently employed soil stabilizers. The scope of this study included the evaluation of improved properties of stabilized subgrade through Proctor compaction, UCS, and CBR tests; followed by pavement design as per IRC 37: 2018; and then economic analysis. The experimental results indicated that an addition of 4% mineral stabilizer increased the CBR by 15 times. Further, an overall thickness reduction of ~30 and 40% was observed with 2 and 4% stabilization, respectively. The associated material optimization was in order of 45% for low and medium traffic levels, and 58% for high traffic levels at 4% stabilization. Economic analysis based on the construction cost of materials showed 23 and 30% reduction when soil was stabilized with 2 and 4% stabilization, respectively, as compared to untreated soil. Overall, this study illustrated the effect of subgrade stabilization with mineral stabilizer on the pavement design and its associated economic impacts.

Keywords Subgrade stabilization · Pavement design · Economic analysis

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1 Introduction

The long-term performance of pavement structures is significantly impacted by the quality of pavement materials and stability of the different layers of the pavement structure. Among the various pavement materials, the in-situ subgrade soil plays a vital role in defining the pavement strength to withstand the traffic loads and environmental impacts. The subgrade's structural properties not only decide the suitability of soil on pavement construction but also influence the subsequent pavement superstructure over the subgrade [1].

However, in the recent past, it has been observed that the unavailability of good subgrade is one of the major concerns in most of the pavement construction locations. Though previously the in-situ material was replaced by good soil from borrow area, it was not always practical and economically viable. Thus, the most commonly used methodology in improving the physical and mechanical properties of subgrade soil to meet the engineering purpose is soil stabilization [2]. Though, there are numerous methods of soil stabilization; use of additives like lime, cement, and asphalt are popular stabilization methods because of the ease in application, efficient, and economical [3]. But over the period, it was observed that multitude concerns followed these common stabilization methods. It was found that soil stabilized by cement or lime showed high pH content. A lot of research has been done on this domain, and it was observed that alkaline migration from stabilized soil depended upon the permeability of the soil and the surrounding cover soil [4]. Moreover, a lot of CO_2 emissions due to cement production is seen, prevention of vegetation growth, groundwater contamination, and heat island creation, to name a few. Another drawback of calcium-based soil stabilizers includes the production of expansive products such as gypsum, ettringite, and thaumasite leading to cracking in soil. Such adverse influence from such stabilizers demands the need for green, sustainable, and effective nontraditional stabilizers, such as enzymes [5]. Substitution of conventional material (cement) and primary raw material (lime) with secondary raw material (waste and byproducts from industries) [6] corresponds to the Sustainable Development Goals set by the United Nations, preserves resources, saves energy, and reduces greenhouse gas emissions [7].

In this direction, mineral stabilization has received positive attention from the research community due to manifold benefits including, (i) naturally available, (ii) stabilization efficiency, (iii) recyclable, (iv) cost-effective, and (v) environmentally friendly. The mineral stabilizer also aids in [8]:

- Higher load-bearing capacity, tensile strength and improved modulus of elasticity
- Neutralization of pH levels and development of water-impermeable layers
- Excess heating or energy is not required
- Non-toxic and not harmful to health
- Recyclable up to 100%.

Thus, the main objective of the paper is to investigate the effect of a naturally developed mineral powder stabilizer on the structural properties of the soil subgrade and its effect on the pavement design. The scope of the study included:

- Literature review pertinent to subgrade stabilization, and associated changes in pavement design and construction
- Stabilization of subgrade soil with varying stabilizer dosages
- Structural characterization of stabilized soil using laboratory tests
- Design of pavements for various subgrade and traffic conditions
- Economic analysis of all designed pavements.

It is envisioned that this study will pave the way for the development of newer technologies on subgrade stabilization that are efficient, durable, and sustainable. The study will also aid in identifying the cost–benefit due to subgrade stabilization.

2 Materials and Experimental Program

2.1 Materials

Soil. The subgrade soil used for the study was collected from a pavement construction site in the Alleppey region of Kerala (as shown in Fig. 1). The soil was laterite in nature, which is rich in iron and aluminium that are formed in tropical areas. It is important to note that lateritic soils in their natural state generally have a low bearing capacity and low strength due to high clay content, strength and stability in the presence of moisture cannot be guaranteed. The pavement construction at this site was impossible due to the poor strength properties of the soil subgrade and necessitated stabilization. An X-ray diffraction analysis (XRD) on the soil sample indicated that the soil mainly consisted of C, H, Na, O, S, Fe, and Al. The particle size analysis done as per IS 2720 (Part V) [9] indicated that the amount of particles passing 75 μm sieve is more than 55%. Further, the Atterberg's limits determined in accordance to IS 2720 (Part V) [9] presented that the liquid limit, plastic limit, and plasticity index were, respectively, 63, 61, and 2%, representing a non-expansible highly plastic soil.

Mineral Stabilizer. The mineral stabilizer is a commercially available soil stabilizing agent that is developed from 100% mineral components (as shown in Fig. 2). The stabilizer is a sustainable alternative to the currently utilized soil stabilizers. The XRD analysis on the mineral stabilizer indicated the stabilizer is a polycrystalline material with the presence of C, H, Cl, Na, and O.

Fig. 1 Soil sample

Fig. 2 Mineral stabilizer

2.2 Experimental Program

For the purpose of determining the stabilizing capability of the mineral stabilizer, the following laboratory tests were carried out on stabilized subgrade at 0, 2, 4, and 6% of stabilizer dosages:

- Proctor Compaction (in accordance with IS 2720: Part VII)
- Unconfined Compressive Strength (in accordance with IS 2720: Part X)
- California Bearing Ratio (in accordance with IS: 2720: Part XVI).

Based on the seed values obtained from the laboratory evaluation, pavement design for various levels of stabilization was determined using IITPAVE software in accordance with IRC 37: 2018 [10] for three traffic levels, i.e., 10, 75, and 150 MSA. Further, a comparative analysis was prepared to identify the possible material savings with subgrade stabilization and consecutively, the potential economic benefits. The results and analyses of the laboratory experiments, and pavement design comparisons are discussed next.

3 Results and Discussions

3.1 Laboratory Test Results

Proctor Compaction. In order to determine the optimum moisture content (OMC) and maximum dry density (MDD), Standard Proctor test as per IS: 2720 (Part VII) [11] was conducted on the stabilized soil at various stabilizer dosages. Figure 3 presents the MDD results with the change in stabilizer dosage. As observed, the MDD increased with increase in dosage indicative of improved strength. The MDD of the natural soil subgrade was 1.415 g/cc at an OMC of 21%. With addition of the mineral stabilizer, a gradual increase in the MDD and reduction in OMC was observed. At 6% of dosage, the stabilized soil displayed MDD of 1.612 g/cc at an OMC of 17%.

Unconfined Compressive Strength. UCS test is one of the fastest methods of measuring shear strength of the soil. UCS test in accordance to IS: 2720 (Part X) [12] was conducted in order to determine UCS of the soil subgrade and to identify the impact of stabilization on compressive strength. As shown in Fig. 4, with increase in stabilizer dosage, a significant increase in the UCS magnitude was observed. Note that the shear failures observed in the specimens were found to be symmetrical. While the untreated soil subgrade exhibited an UCS of 7.547 kg/cm^2, i.e., 740.104 kPa, with

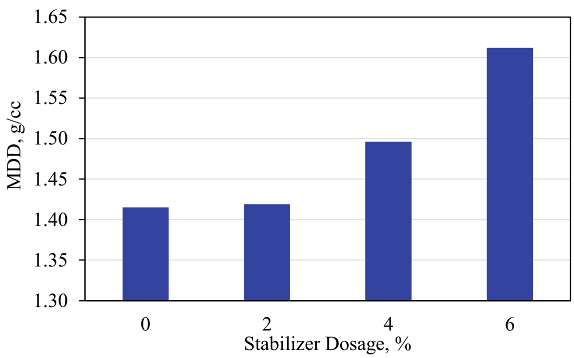

Fig. 3 MDD results of stabilized soil

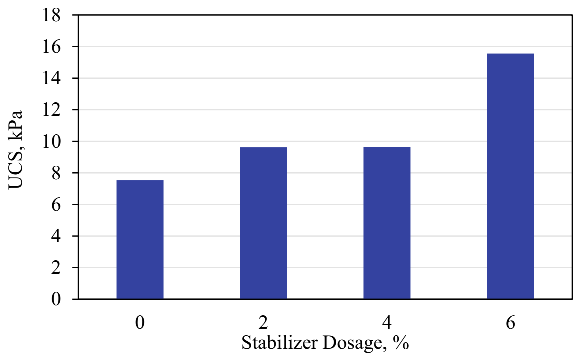

Fig. 4 UCS results of stabilized soil

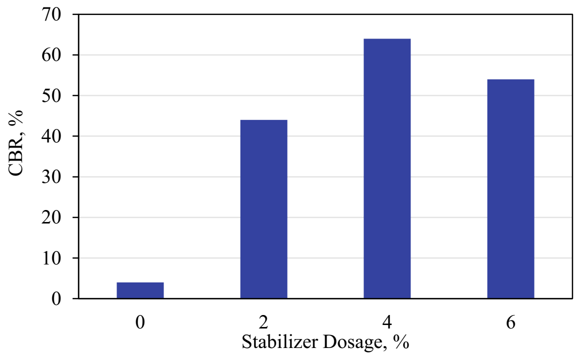

Fig. 5 CBR results of stabilized soil

mineral stabilization at 6% dosage, the UCS was found to be 1531.79 kPa. The UCS results indicated the improvement in compressive strength with soil stabilization.

California Bearing Ratio. CBR is a measure of the strength of the pavement subgrade and is a vital input for the pavement design. CBR test was conducted in accordance to IS: 2720 (Part XVI) [13]. Figure 5 displays the soaked CBR results of the stabilized subgrade. The CBR of the natural subgrade was found to be 4% and with the inclusion of mineral stabilizer, the CBR of the soil was significantly enhanced. Note that the CBR was maximum with a magnitude of 64% at 4% of stabilization indicative of improved bearing strength. It is also important to note that the CBR value decreased with further increase in the stabilizer dosage. Therefore, for further analysis, stabilization up to 4% dosage is examined.

3.2 Pavement Design

As an effort to identify the effect of subgrade stabilization on the pavement structure and its associated performance, a pavement design exercise was carried out in accordance with IRC 37: 2018. For the pavement design, the vital subgrade input property, CBR was adopted from the laboratory results as mentioned in the previous section. Further to evaluate the interaction effect of both subgrade stabilization and traffic levels on the pavement design, three traffic levels, 10, 75 and 150 MSA were considered in the study to represent the extreme and also the intermediate traffic conditions. The pavement design was accomplished using IITPAVE software available with IRC 37: 2018.

Figure 6a and b represents the pavement cross-sections considered for various traffic levels. As shown in Fig. 6a, the cross-section for 10 MSA traffic level includes, granular subbase (GSB), wet mix macadam (WMM), surface course with VG40 binder. In case of traffic levels, 75 and 150 MSA, in addition to GSB, WMM and surface course; a layer of dense bituminous macadam (DMB) with VG40 binder is considered as presented in Fig. 6b. Note that all layers were in accordance with MoRTH specifications (MoRTH 2013) [14].

Table 1 presents the pavement cross-sections for various traffic levels. Note that the pavement sections were designed by comparing the vertical strain at the top of subgrade (ε_v) and tensile strain at the bottom of bituminous layer (ε_t) from the traffic and pavement structure. A safe pavement cross-section is the one with lower ε_v and ε_t than the derived critical strains from the traffic conditions. Table 1 provides the cross-sectional details of pavements for different traffic and stabilization levels.

From Table 1, it was found the pavement composition altered significantly with the inclusion of mineral stabilizer in subgrade, with a reduction in total pavement thickness with increase in the stabilizer dosage. It resulted in satisfactory values for ε_v and ε_t at lower pavement layer thicknesses. Note that with 4% dosage of stabilization,

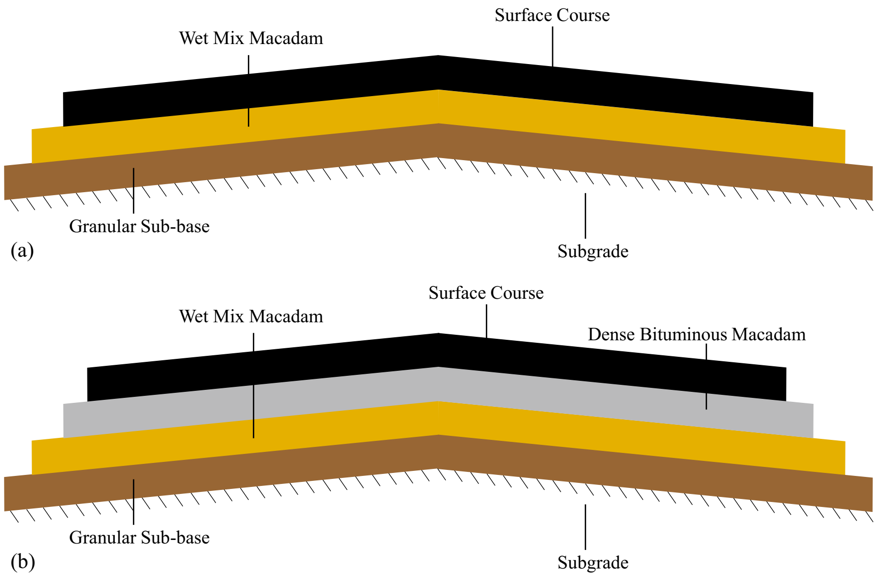

Fig. 6 Pavement cross-sections for traffic of **a** 10 MSA and **b** 75 and 150 MSA

Table 1 Pavement cross-sections for various traffic levels

Traffic level, MSA	Stabilizer dosage (%)	Pavement cross-section, mm			
		Surface course	DBM	WMM	GSB
10	0	40	–	250	300
	2	40	–	150	200
	4	40	–	100	200
75	0	40	80	250	300
	2	40	80	150	200
	4	40	80	100	200
150	0	40	100	350	400
	2	40	80	120	200
	4	40	80	110	200

the overall pavement thickness reduced approximately by 45% for low and medium traffic levels and the reduction was more than 50% for high traffic level pavements.

Similarly, it was also observed that an increase in traffic level necessitated a thicker pavement cross-section. Higher the pavement layer thicknesses, lower the ε_v and ε_t, and therefore, safe pavement design for higher traffic levels. With an increase in traffic level from 10 to 150 MSA, the overall pavement thickness increased approximately by 1.5 times.

3.3 *Economic Analysis*

This sub-section discusses the reduction in construction material required and associated savings in construction cost. In order to carry out the aforementioned tasks, a pavement of length 1 km and 3.5 m width was studied, Table 2 presents the material requirement and construction cost for all the nine designed pavement cross-sections. As observed, the construction material requirement drastically reduced with the inclusion of mineral stabilizer in the subgrade. For a traffic level of 10 MSA, with the addition of 2% stabilizer to the subgrade, the required construction material reduced by 36%, wherein the reduction in WMM was approximately 40% and in GSB was nearly 33%. Similar observations were made in case of higher traffic levels as well. With 2% stabilizer dosage, ~36% material reduction was observed in case of 75 MSA traffic and ~57% material reduction were observed in case of 150 MSA traffic.

Furthermore, with 4% dosage of subgrade stabilization, nearly 45% reduction in construction materials was noticed for traffic levels, 10 and 75 MSA; and 58% reduction was observed in case of 150 MSA traffic. It is concluded that subgrade stabilization not only improves the subgrade strength but also significantly lowers the material need for the pavement construction.

Table 2 Material requirement and economic analysis of pavement cross-sections

Traffic level, MSA	Stabilizer dosage (%)	Construction material (m^3)				Construction cost (₹)	Savings in construction cost (%)
		Surface course	DBM	WMM	GSB		
10	0	140	0	875	1050	3,945,641	–
	2	140	0	525	700	2,932,391	25.7
	4	140	0	350	700	2,672,866	32.3
75	0	140	280	875	1050	4,729,641	–
	2	140	280	525	700	3,716,391	21.4
	4	140	280	350	700	3,456,866	26.9
150	0	140	350	1225	1400	5,938,891	–
	2	140	280	420	700	3,560,676	40.0
	4	140	280	385	700	3,508,771	40.9

Additionally, the construction cost analysis carried out on all nine pavement cross-sections displayed the cost savings due to subgrade stabilization. Subgrade stabilization averted the need of thicker pavement cross-section and in turn resulted in significant decrement of pavement construction cost. As presented in Table 2, with a stabilizer dosage of 2% in subgrade, around 25% savings in construction cost was observed for lower traffic level of 10 MSA, and around 21% savings was observed for medium traffic level of 75 MSA. For high traffic level of 150 MSA, a rise in overall savings was observed with nearly 40% reduction in construction cost.

Similar observations were made for higher level of subgrade stabilization. With increase stabilization dosage by 2%, an increment in total savings was observed for all traffic levels. The increments were in orders of approximately 7, 5, and 1% for 10, 75 and 150 MSA traffic. Overall, it was concluded that the subgrade stabilization resulted in thinner pavement cross-sections and lesser construction cost in comparison with the un-stabilized subgrade.

4 Conclusion and Future Scope

This research work evaluated the performance and economic benefits of subgrade stabilization using a newly available mineral stabilizer. The properties of the stabilized soil were evaluated through essential tests, including Proctor Compaction, Unconfined Compressive Strength, and California Bearing Ratio; and the results were considered for flexible pavement design as per IRC37:2018. The major findings were summarized as follows:

- ***Proctor Compaction***: MDD was found to be increasing with an increase in the percentage of mineral stabilizer. An increment of 15% in MDD was observed when the subgrade was stabilized with 6% mineral stabilizer. Further, it was also observed that with the inclusion of stabilizer the amount of water required was reduced significantly.

- ***UCS***: was also found to be increasing with increase in the stabilizer dosage. With 6% stabilizer dosage, the UCS value increased by nearly 100%. The UCS results signified a definite improvement in the soil's compressive strength.
- ***CBR***: showed substantial improvement with an increase in mineral stabilizer content. With the addition of 4% stabilizer, the CBR value increased by 15 times. Higher CBR subgrade means a comparatively thinner subbase layer is required to satisfy the traffic load requirement of the pavement. However, it was also observed that with further increase in the stabilizer dosage, the CBR decreased indicative of an optimum stabilizer dosage.
- ***Pavement Design***: was done for different dosages of mineral stabilizer to determine the thickness of various layers for three traffic levels. 30 and 40% of overall thickness reduction were observed with the addition of 2 and 4% mineral stabilizer, respectively.
- ***Economic Analysis***: was done for different dosages of mineral stabilizer and for low and medium traffic levels, a general decrease in thickness resulted in nearly 23 and 30% cost saving when the soil was stabilized with 2 and 4% stabilization, respectively. The cost savings were the highest for 150 MSA traffic level.

The future scope of the study includes more research towards exploring the influence of the stabilizer on other soil types and its associated variations in stabilization, micro-characterization, and optimization. In this direction, it is also important to understand the permeability properties of the soil and the impact of stabilization on subgrade permeability and drainage characteristics. Further, it is also planned to conduct a comparative analysis on effect of subgrade stabilization on both rigid pavement and flexible pavement performance and its life cycle analysis. Overall, it is envisioned that this study will help understand the performance and economic benefits of stabilized subgrade using a sustainable alternative approach.

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An Effective Bitumen-Friendly Polymer for Superior Roadway Performance and Durability

Krishna Srinivasan, Sachin Raje, and Deepak Madan

Abstract Polymer Modified Bitumens (PMB) are utilized in Asphalt Mixes to improve high temperature grade, stiffness, rutting resistance and damage tolerance, resulting in more durable roadways which are able to withstand increased traffic loads.

PGXpand, a Bitumen-Friendly Polymer (BF Polymer), developed by Sripath Technologies, is unlike any traditional elastomeric or plastomeric polymers. The BF Polymer interacts with the binder in a very unique fashion, imparting key performance benefits to PMB Mixes, while mitigating the processing difficulties and shortcomings typically associated with traditional polymers.

Increasingly, experts from around the globe believe that performance and durability of roadways is more important than measuring elastic recovery of the modified binder, which only confirms the presence of a polymer [1, 2]. This is where the BF Polymer shines. It is designed to interact with bitumen in a very innovative and unique manner.

The BF Polymer is designed to be highly dosage efficient, storage stable, and easy to mix into bitumen. It is designed to improve the workability of PMB Mixes and mimic the advantages of warm mix additives. It is designed to lower the paving temperatures and make the mix easier to pave and compact.

The BF Polymer is designed to deliver the desired roadway performance and durability for targeted weather and traffic conditions. Roadways paved using BF Polymer PMBs deliver outstanding rutting resistance, excellent fatigue properties, and long-term durability.

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Keywords Bitumen-Friendly Polymer · PGXpand polymer · Polymer modified bitumen (PMB)

1 Introduction

PGXpand, a Bitumen-Friendly Polymer (BF Polymer), developed by Sripath Technologies, is a novel, medium molecular weight polymer, specifically engineered with a tailored architecture and chemistry [1]. It is a new family of polymer additives, unlike any traditional elastomeric or plastomeric polymers. The BF Polymer interacts with the binder in a unique fashion. It delivers certain key performance benefits to PMB Mixes, while making it easier to process the PMB by mitigating the difficulties and shortcomings of traditional polymers.

1.1 Key Characteristics of Bitumen-Friendly Polymer Additive

Figure 1 summarizes key characteristics of the BF Polymer presented in this paper. BF Polymer delivers excellent performance, it is easy to incorporate into a plant operation, and it helps reduce the overall cost of the mix.

When compared to a PMB made using traditional polymers, a PMB produced with the BF Polymer is much easier to incorporate into a Mix plant. It melts rapidly in Bitumen at around 150 °C. It can be easily incorporated into the binder using low energy mixers and short mixing times. It exhibits excellent storage stability.

The BF Polymer is highly dosage efficient. At low dosage levels, it lowers the viscosity of bitumen and enhances internal lubricity properties in Mixes. It improves the workability of PMB Mixes and mimics the advantages of a warm mix additive. It lowers the paving temperature and makes the mix much easier to compact.

Fig. 1 Key characteristics of BF polymer

The BF Polymer eliminates or reduces the need for raw materials such as, cross-linking additives, warm mix additives, and traditional polymers. It contributes to lower energy consumption and overall lower mix and paving costs.

This polymer boosts the high temperature performance of bitumen without any impact on the low temperature properties. Roadways paved with PMB mixes made using BF Polymer deliver outstanding rutting resistance, excellent fatigue properties, and long-term durability.

1.2 Key Materials Used and Test Standards

Binders, aggregates, and other asphalt additives from around the globe were used when generating the data presented in this paper. The BF Polymer used for the data in this report was PGXpand, a Bitumen-Friendly Polymer developed and commercialized by Sripath Technologies [1, 2]. BF Polymer has been compared against commercially available traditional plastomeric and elastomeric polymers. Test data presented was conducted using materials and as per specifications applicable to the geographic region where testing was conducted.

2 Results and Discussions

The BF Polymer has been tested, evaluated, and vetted by leading experts in academia, industry and transportation agencies from around the world. It has been trusted and effectively used on roadways across the globe since 2015 [2].

2.1 Improved High Temperature Properties

The impact of varying the BF Polymer content on the properties of a commercial grade of bitumen grade PG 64-22, sourced in the USA was evaluated. Table 1 outlines the impact of the addition of 1.5 to 3% of the BF Polymer to this bitumen. The BF Polymer raises the softening point of the base binder from 48 °C to over 120 °C for a 2% polymer addition. Penetration values are reduced from 67 dmm to around 40 dmm and the viscosity reduces from 450 cps to around 380 cps with the addition of the BF Polymer.

The viscosity profiles shown in Fig. 2 compare the viscosity versus temperature of the base bitumen (USA Grade 64-22) and the base bitumen modified with 1.5% of the BF Polymer. The addition of the polymer lowers the viscosity of the binder by 5–10% over the entire production-compaction temperature range, while boosting the high temperature grade.

Table 1 Hot mix properties of PMB mix USA bitumen PG 64-22 + Bitumen-Friendly Polymer; as per applicable ASTM standards

With PGXpand®	Softening point (°C)	Penetration (dmm)	True grade (°C)	Viscosity @ 163 °C (cps)
Base binder	48	67	66.8–26.1	450
1.5%	98	45	85.3–25.1	402
2%	122	40	95.0–25.0	384
3%	123	39	95.0–25.0	370

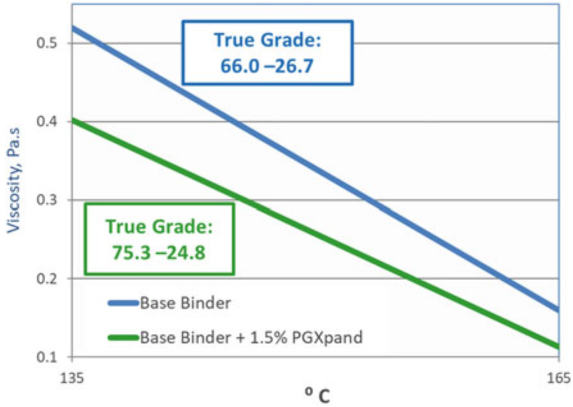


Fig. 2 Viscosity profile for USA base bitumen versus binder + BF polymer

The Royal Melbourne Institute of Technology in Melbourne, Australia (RMIT), evaluated the impact of the BF Polymer on the properties of an Australian bitumen C170 [3, 4]. Binder and mix level properties were evaluated on an AC14 dense grade mix. Both results are summarized in Table 2.

Table 2 Properties of Australian bitumen C170 + Bitumen-Friendly Polymer AC14 dense graded asphalt mix, evaluated at RMIT, Australia

Properties	Test method	Units	% PGXpand addition				
			1.2	1.5	2.0	2.5	3.0
Viscosity @ 165 °C	AS/NZS2341.4	Pa.s	0.11	0.11	0.12	0.11	0.12
Softening point	AGPT/T131	°C	74	107	111	115	117
Stress ratio @ 10 °C	AGPT/T125	–	0.99	1.02	1.05	1.02	1.03
Stiffness @ 25 °C	AGPT/T121	kPa	35	40	48	56	64
PMB separation	ASTM D7173	°C	1.1	0.8	0.2	1.1	1.5
Elastic Rec. @ 25 °C	ASTM D6084	%	22.0	19.0	13.5	13.0	11.0
Hamburg wet, rut depth @ 50 °C, 20 K	AASHTO T324	mm	4.88	4.62	3.57	2.70	2.18

Table 3 Properties of Indian bitumen VG30 + Bitumen-Friendly Polymer 5.4% binder, 3.84% voids, 12 mm, evaluated at IIT Bombay

Properties	Test method	Units	1.5% PGXpand
Viscosity @ 150 °C	ASTM D4402	mPa.s	244.4
Softening point	IS 1205	°C	89.7
Penetration	IS 1203	dmm	20
PMB separation	IRC SP 53:2010-A3	°C	0.90
Elastic Rec. @ 25 °C	IRC SP 53:2010-A2	%	20.67
Hamburg wet, rut depth @ 50 °C, 20 K	AASTHOT 324	mm	5.69
Ideal-CT @ 25 °C, 50 mm/min	ASTM D8225-2019	–	74
Resilient modulus @ 1 Hz, 35 °C	ASTM D4123	MPa	2181

A typical dosage level of 1.5 to 2.5% of the polymer delivered excellent properties. The softening point increased to around 110 °C, stiffness to around 50 kPa, while maintaining low levels of viscosity of around 0.1 Pa.s when tested at 165 °C. The rut depth as measured by Hamburg Wet Wheel Tracking tested at 50 °C was around 4 mm for 20,000 cycles.

A PMB based on Indian bitumen grade VG30 with 1.5% BF Polymer was evaluated at the Indian Institute of Technology Bombay, Mumbai, India (IIT Bombay) [5]. Mix level properties were evaluated on a mix with 5.4% binder, 3.84% voids and based on a nominal 12 mm aggregate. The properties are summarized in Table 3.

The PMB with 1.5% of the BF Polymer delivered excellent properties—a viscosity of 244 mPa.s at 150 °C, a softening point of 90 °C, penetration at 20 dmm, and PMB separation of 0.9 °C. The rut dept as per Hamburg Wet Wheel Tracking tested at 50 °C was 5.7 mm and the Ideal CT Index tested at 25 °C was 74.

2.2 *Superior Stability and Separation Properties*

PMB based on the BF Polymer exhibited excellent storage stability. Bitumen samples with 1.25% of the polymer were held in a cigar tube at 165 °C for 72 h. $G^*/\sin\delta$ of the top and bottom portions of the cigar tube were measured at 3 different temperatures. The variation between the top and bottom measurements was less than 5%. Other properties, such as softening point, penetration, and phase angle, showed similar small variations, demonstrating the outstanding storage stability of the BF Polymer.

Also, as seen in Table 2, an Australian Bitumen C170 mixed with BF Polymer at levels ranging from 1.2 to 3.0% showed excellent PMB separation properties.

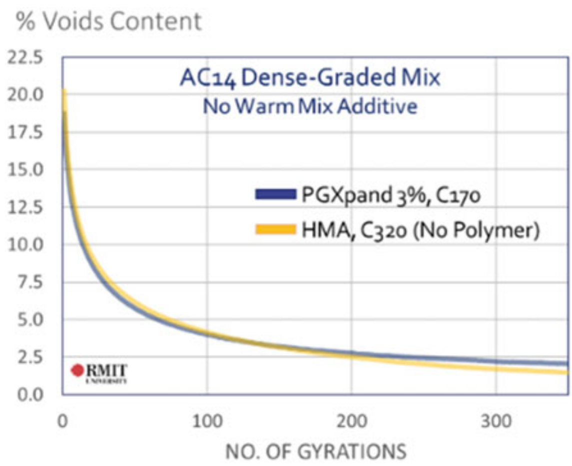


Fig. 3 Comparing compactability curve of BF polymer PMB versus no polymer hot mix Australian bitumen C170 + BF polymer versus hot mix based on C320 bitumen

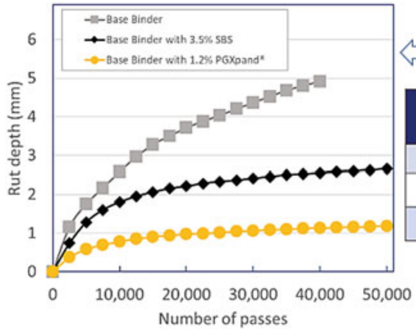
2.3 Unique Mix Compaction and Workability

Even at low dosage levels, the BF Polymer lowers the viscosity of bitumen and enhances its internal lubricity properties, resulting in improved workability of such PMB Mixes. The polymer mimics the advantages of a warm mix additive, lowers the paving temperature and makes the mix much easier to pave and compact.

Figure 3 compares the compactability curves of an Australian AC14 dense-graded Mix based on a C170 Australian binder and 3% BF Polymer to the curve for a standard Hot Mix Asphalt (HMA), made using a C320 Australian bitumen with no polymer added. The two curves are almost identical, demonstrating the unique ability of the BF Polymer to improve Mix workability and aid compaction. It should be noted that the softening point of the bitumen in the BF Polymer based mix is over 110 °C compared to the softening point of around 50 °C for the standard HMA.

2.4 Excellent Rutting Resistance

An in-depth study on the rutting performance of the BF Polymer was conducted by the Indian Institute of Technology Madras, Chennai, India (IIT Madras) [6]. Figure 4 compares the rutting resistance, as measured by a Hamburg Wheel Tracking Test at 60 °C, of an Indian bitumen grade VG30 to two PMB mixes, one made with 3.5% traditional elastomeric polymer and the other with 1.2% BF Polymer [6]. The bitumen in both PMB mixes had a softening point of around 75 °C. The PMB Mix made with BF Polymer has far superior rutting resistance. By any measure of rutting resistance, the BF Polymer consistently showed superior rutting resistance compared to the traditional elastomeric polymer.



Mixture	TR (µm/cycle)	WTR (µm/cycle)	Pi at 20,000th Load Cycle (%)	WTS$_{air}$ (mm/10^3 cycle)
Base Binder (VG30)	0.6	0.4	0.7	0.18
Base Binder + 3.5 % SBS	0.4	0.3	0.3	0.09
Base Binder + 1.2 % PGXpand	0.2	0.1	0.1	0.03

Fig. 4 Rutting resistance of Indian bitumen VG30 mixes base binder versus 3.5% SBS PMB versus 1.25% BF polymer PMB Hamburg Wheel Tracking Test @ 60 °C, Indian bitumen VG30

2.5 Fatigue Performance

In addition to providing superior high temperature performance and rutting resistance, PMB mixes made using the BF Polymer, in spite of low elastic recovery values, demonstrated fatigue performance comparable to traditional elastomeric PMB mixes.

The 4-point beam bending fatigue performance of two PMB mixes is compared in Fig. 5. The curve for the PMB mix made using 1.75% BF Polymer is almost identical to the curve for the PMB mix made using 4% traditional elastomeric polymer. The flexural fatigue test was conducted at 20 °C, 10 Hz loading, using a PG 64-22 binder grade for a high-performance bridge deck mix in New Jersey, USA. Both the BF Polymer and the traditional elastomeric modified bitumens had a high temperature true grade of 75 °C.

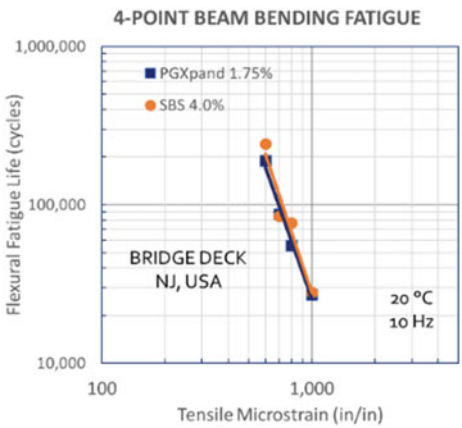


Fig. 5 Fatigue 4-point beam bending @ 20 °C, 10 Hz USA bitumen PG 64-22, 4% SBS PBM versus 1.75% BF polymer

Fig. 6 Bitumen-Friendly Polymer on expressway in India

3 Key Applications of Bitumen-Friendly Polymers

Bitumen modified with BF Polymer is used for a variety of paving and road repair and maintenance applications, including: hot mix paving, hybrid polymer modified bitumen, high-stiffness asphalt mixes (HiSAM), hot spray sealing, BF Polymer modified emulsions, and cold patch mixes for pothole repair.

3.1 Hot Mix Paving Applications

Polymer Modified Bitumen (PMB) manufactured using BF Polymer has been used to improve rutting resistance and deliver desired roadway performance on high-traffic roadways around the world since 2015. Such PMBs are easy to produce, help reduce cost and are easier to pave and compact.

As an example, the BF Polymer was successfully used to pave a wear layer surface on a 2 km stretch of a high traffic state highway SH-2 in Andhra Pradesh, India [7]. About 1.5% of the BF Polymer mixed with an Indian bitumen VG-30 yielded properties similar to a mix based on 3.5% of a traditional elastomeric polymer (Fig. 6).

As another example, the BF Polymer was successfully used to improve the rutting performance on a 4-lane km stretch of Agra-Gwalior Highway in India [8]. About 1.5 wt.% of BF Polymer was added to an Indian bitumen grade VG30. The Mix based on this PMB was used to pave a 30 mm wear layer on a 4 lane-km stretch of Agra-Gwalior highway about 40 km from Gwalior.

3.2 Hybrid Polymer Modified Bitumen

As seen in Table 4, the BF Polymer can be used in combination with traditional elastomeric polymers to create Hybrid PMB formulations. Table 4 shows the effect of adding BF Polymer to a PMB created by adding 2.5% traditional elastomeric polymer to the base binder. An addition of 1.5 to 3% of BF Polymer to the PMB,

Table 4 Hybrid polymer modified bitumen properties of USA bitumen PG 64-22 + 2.5% SBS + BF polymer

With PGXpand	Softening point (°C)	Penetration (dmm)	True grade (°C)	Viscosity @ 163 °C (cps)
Base binder + 2.5% SBS	66	74	73.1–29.0	450
1.5%	105	47	85.6–28.4	402
2%	125	42	95.0–28.1	384
3%	124	38	95.0–28.0	370

increased the softening point, lowered the penetration, and reduced the viscosity by about 7–10%.

Such a Hybrid PMB allows the incorporation of select benefits of BF Polymer into a traditional elastomeric PMB, making them easier to produce and use, deliver improved roadway performance.

3.3 High Stiffness Asphalt Mixes (HiSAM)

Currently available high-stiffness mixes are quite expensive, need to be paved at high temperatures, require multiple lifts to lay down the road, are difficult to manufacture and are not easy to use in the field.

High-Stiffness Asphalt Mixes based on BF Polymer (HiSAM) are a viable alternate solution. HiSAM mixes are easy to manufacture and use, allow paving at lower temperature, and are environmentally more sustainable. The technology allows for the reduction in void content, reduction in base course thickness, improvement of moisture resistance, and consolidation of one or more lifts.

3.4 Hot Spray Seal Applications

BF Bitumen is also used in hot spray chip seal applications. The BF Bitumen can be easily incorporated into bitumen, in some cases along with crumb rubber or traditional elastomeric polymers. Such PMBs have a lower viscosity, are easy to spray, result in better roadway performance, and potentially help reduce cost.

3.5 PGXpand Modified Emulsion Applications

The BF Polymer can be easily incorporated into bitumen to create a PGXpand Modified Emulsion. Such emulsions are easy to emulsify using traditional emulsification technologies and applied on a road surface using normal application techniques, equipment and practices. BF Polymer based emulsions are used for slurry seal, chip seal, scrub seal, fog seal, and microsurfacing applications.

The BF Polymer promotes rapid skin-over, resulting in a tack-free trackless road surface that can be rapidly opened to traffic. Key advantages include, less greenhouse gas emissions, improved emulsion stability for transport, outstanding rutting resistance, and durable roadways that require less frequent maintenance.

4 Ease of Use in a Plant Operation

The BF Polymer is easy to incorporate into bitumen in a PMB plant, a mix plant. Table 5 outlines the recommended guidelines for incorporating BG Polymer into a Mix plant operation [9]. The addition of the BF Polymer into bitumen is an endothermic process, thus, the mixture needs to be heated during the entire blending process to maintain the desired temperature. It is important to note that when BF Polymer is added to bitumen, the viscosity drops by about 5 to 15%. At suggested blending temperatures, the BF Polymer melts easily into the bitumen, due to liquid-on-liquid mixing. At a blending temperature of about 150 to 160 °C, the BF Polymer can easily be mixed into bitumen within 1 to 2 h using low shear mixers, and minimal recirculation.

Table 5 Ease of using Bitumen-Friendly Polymer

Key parameters	Guidelines
Blending temperature	Endothermic reaction 150 to 163 °C (300 to 325 °F)
Mixing equipment	Low shear mixer
Blending speed	Maintain a small minimal vortex
Blending time	1 to 3 h
Paddle speed and temperature	Reasonable speed above 163 °C (325 °F)
Viscosity	Drops with addition of PGXpand

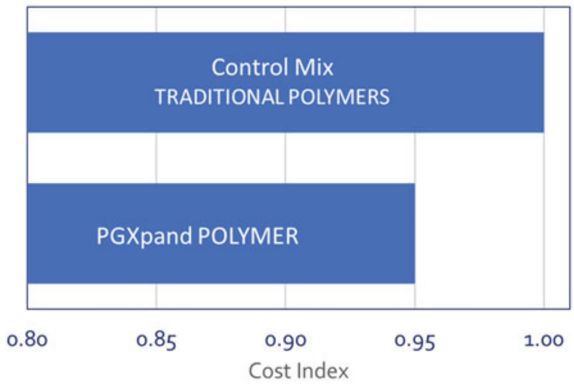


Fig. 7 Bitumen-Friendly Polymer reduces overall mix cost

5 Reducing Overall Mix Cost with Bitumen-Friendly Polymer

The use of BF Polymer yields important cost benefits. Paving mixes typically require less than half the amount of a traditional polymer to yield similar properties. It is typically possible to replace traditional elastomers in a 2:1 or better ratio to yield similar roadway performance. This reduction in raw materials requirement results in overall savings in mix cost, as demonstrated in Fig. 7.

6 Conclusions

The Bitumen-Friendly Polymer has been specially engineered to interact with bitumen in a very unique and innovative manner, allowing product to deliver superior roadway performance and durability. The polymer improves key performance qualities of bitumen, such as, softening point, penetration, and true high grade. It lowers viscosity of PMB mixes, improves workability and makes mixes easier to compact. This unique polymer delivers roadways with outstanding rutting resistance, fatigue properties and long-term durability. The BF Polymer has been tested, evaluated, and vetted by leading experts in academia, industry and transportation agencies from around the world. It has been trusted and effectively used on roadways across the globe since 2015. In the future, the authors plan to continue evaluating and reporting on the performance of adoption of BF Polymer in PMB mixes on roadways around the world.

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Rheological Investigation of Soft Grade Asphalt Binder Modified With Crumb Rubber-Nanosilica Composite

Tabish Mehraj, Mohammad Shafi Mir, and Bijayananda Mohanty

Abstract This study aims at investigating the effect of using nanosilica (NS) as a modifier for Crumb rubber (CR) modified asphalt binder. In this study, the concentration of CR was kept constant as 12% (wt of base binder) and the nanosilica concentration was varied from 1% to 6%. The effect of varying concentrations (1, 2, 3, 4, 4.5, 5 and 6%) of nanosilica (by weight of binder) on CR modified binder were evaluated by utilizing various physical tests like penetration, softening point, and ductility. The rotational viscosity (RV) and dynamic shear rheometer (DSR) tests were used to analyze rheological properties of base binder and nanosilica polymer modified asphalt binder. In addition, the performance of modified asphalt after thin film oven (TFO) (short-term aging) and Pressure aging Vessel (PAV) test (long term aging) were assessed as well. Furthermore, the storage stability of modified asphalt binder was evaluated. Results showed that the addition of nanosilica has a positive effect on rutting performance of CR modified asphalt binders. Storage stability of the CR modified asphalt binders improved significantly after the addition of nanosilica. Using softening point and rheological parameters (complex modulus (G*) and phase angle (δ), the best values were possessed by 12% CR-4% NS modified binder. During rheological characterization, it was found that complex modulus increases, phase angle decreases, superpave rutting parameter increases and failure temperature increases with increasing nanosilica content. It was also found that Brookfield viscosity increases with increasing nanosilica concentration as the binder becomes stiffer. All the test results confirmed the fact that the crumb rubber-nanosilica modifier is effective in enhancing the high temperature properties (rutting resistance) of the soft grade binders and at the same time, it increases the elasticity of the binders.

Keywords Crumb rubber · Nanosilica · Superpave rutting parameter · Superpave fatigue parameter · Viscosity · Ageing · Storage stability

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1 Introduction

Today is a world of sustainable technology and researchers are interested in finding sustainable, eco-friendly and cost-effective materials. Use of crumb rubber, also called black pollutant [34] in asphalt production is such cost-effective, ecofriendly [4, 30] and sustainable material which transforms an unwanted residue into a new bituminous mixture which is highly resistant to rutting and fatigue. The use of crumb tire rubber (CTR) is an interesting alternative from both economical and environmental perspectives [13]. CTR is dangerous due to its potential environmental threat and fire hazards [17] and its usage in pavements solve its disposal problem [37] and such threats. Crumb rubber modified binder (CRMB) provides improved mechanical properties, increases pavement durability and reduces reflective cracking and fatigue resistance [23]. Compared to conventional road surfaces, those made from CRMB have a longer service life, with 50–70% decrease in noise level raised cold- and heat-resistance properties and improved slip resistance, resulting in shorter braking distance and a higher safety coefficient [34].

Crumb rubber can be incorporated into asphalt mixtures either by wet or dry process. In the dry process, CR is used as partial substitution for fine aggregate. However in wet process, the CR powder is first blended with hot base asphalt and then swelled in the matrix to prepare modified asphalt. The wet process can generate two totally different modified asphalts, called as asphalt rubber (AR or wet-process-high-viscosity) and terminal blend (TB or wet-process-no-agitation). AR binders can be produced by using more coarser rubber (minus 30 mesh, greater than 15% wt of virgin asphalt), and are resistant to rutting and reflective cracking but cause issues due to higher viscosity such as difficult to handle while paving, inability to store over long periods and modification of mixing equipment by contractors [15]. Thus, a good alternative is the TB binder that overcomes the viscosity issue by using less and finer CR (plus 30 mesh, less than 10%wt of virgin asphalt) and by applying high temperature shear in the modification. TB has better storage stability than AR [24].

TB binder holds many advantages such as low viscosity, good workability and applicability to dense graded mixture [25] but degradation of CR might impair the elasticity of binder and high temperature performance [22, 24]. Although digestion of rubber in TB improves its thermal storage stability it still has some separation problems due to partially undissolved rubber particles [14]. To remove such anomaly, modification with a second modifier can be done.

In recent years, nanomaterials have been widely used to enhance the physical, rheological and mechanical properties of asphalt binder [29]. Addition of nano-sized additives improves the performance of asphalt binder and overcomes the drawbacks of polymers. Thus, Polymer nanocomposites are considered to be more powerful modifiers [28]. Nanosilica is an inorganic nanomaterial that has been being widely used to enhance the performance of polymer modified asphalt [1]. Nanosilica can be produced by agricultural waste materials such as rice husk ash, sorghum vulgare seed heads and bagasse ash by precipitation method, bio-digestion process and sol–gel process (Bhat and Mir 2020). Addition of Nanosilica improves the storage modulus, elasticity and ageing resistance of the asphalt binder [27] because of its high specific surface area, high functional density and high strain resistance.

Various researches have been done to improve the physical, rheological, and mechanical properties of CRMA in recent years. For example, Attia has added 2% SBS to improve the binder performance [6]. Khasawneh added MSW and NS to CRMB and found no enhancement by MSW of high temperature performance but a negative effect on low-temperature performance. Addition of 12% CTR was recommended because it produced the greatest enhancement for high temperature performance [5]. Abedali added bentonite (khawa clay) and crumb rubber to asphalt. The addition of bentonite and crumb rubber enhanced asphalt properties such as the viscosity, ductility and softening point, decreased penetration [2]. Liu added NS to pre-oxidized CR. H_2O_2 and NaClO were used to oxidize the CR. Overall, Storage stability and high temperature performance was enhanced [18].

The purpose of the current study is to use nanosilica to improve the performance of CRMB at high and intermediate temperatures as well as its storage stability. The goal of this study is to determine whether it is feasible to use 12% CRMB with NS particles while utilising seven different NS contents (NS contents of 1, 2, 3, 4, 4.5, 5, and 6%). To learn more about the properties of CR-NS binders, several experiments were carried out, including the standard tests, brookfield viscosity (BV), dynamic shear rheometer (DSR), storage stability, short term ageing, and long term ageing.

2 Goals of the Study

(1) To determine Optimum Mixing Time for preparation of Crumb rubber-Nanosilica modified bitumen.
(2) To evaluate the Optimum Nanosilica content for the Crumb rubber modified binder (CRMB) using softening point method and rheological parameters (G^* and δ).
(3) To investigate the influence of Crumb rubber-Nanosilica nanocomposite on viscosity of the asphalt binder.
(4) To study the effect of Crumb rubber-Nanosilica content on rheological behavior of the asphalt binder based on rutting and fatigue.
(5) To evaluate the effect of Crumb rubber-Nanosilica on Ageing and high temperature storage stability of the asphalt binder.

Table 1 Base binder physical properties [8]

Test	Standard code	Values	Specification limit (minimum)
Softening point (°C)	IS: 1205	46	40
Ductility (cm)	IS: 1208	100+	75
Penetration 0.1 mm at 25 °C	IS: 1203	88	80
Dynamic viscosity at 60 °C	1S: 1206(Part II)	1064	800
Kinematic viscosity at 135 °C	1S: 1206(Part III)	278	250

Table 2 Crumb rubber physical properties

Specification	Value
Particle size	<0.9 mm
Relative density	1.18
Natural rubber	35–58%
Rubber hydrocarbon	45–54%
Ash	4–6.5%
Carbon black	29–35%

3 Programme and Technique for Experiments

3.1 Compositional Description of Material

The base binder for this investigation is 80/100 penetration grade bitumen (VG-10), which was bought from a nearby distributor. Table 1 displays the mentioned base binder's numerous characteristics. For the purpose of altering the asphalt binder, nanosilica (NS) and crumb rubber (CR) were both taken into consideration. Platonic Nanotech Private Limited provided the nanosilica, while a regional distributor provided the CR. Tables 2, 3 and 4 (which were provided by the supplier), respectively, present the fundamental characteristics of CR and Nanosilica. Figures 1 and 2 show the SEM images of CR and nanosilica, respectively. The optimal concentration of CR was determined in this study to be 12% [5], while the concentration of nanosilica was changed from 1 to 6% [1]. Additionally, the mixing temperature (180 °C) was maintained [19].

3.2 Preparation of Samples

Crumb rubber and nanosilica were added to the asphalt binder using a high-shear mixer. To start, a consistent liquid was created by heating the asphalt binder on its own. CR was added to the asphalt binder at 12% [5] by weight of the asphalt binder. Finally, nanosilica was slowly added to the mixture at different concentrations (0, 1,

Table 3 Nanosilica physical properties [7]

Specification	Value
Purity	99.5%
Average size of the particle	30–50 nm
Specific surface area	200–250 m^2/g
Bulk density	0.10 g/cm^3
True density	2.5 g/cm^3
Morphology	Porous

Table 4 Nanosilica particles' elemental make-up [7]

SiO_2	Al	Fe	Mg	Ca
99.5%	0.02%	0.05%	0.1%	0.08%

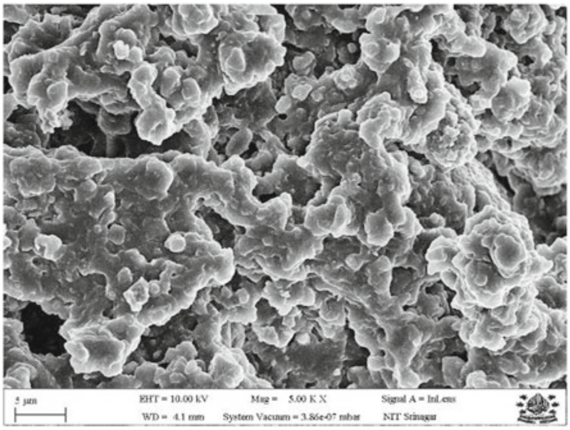


Fig. 1 Crumb rubber in a SEM image

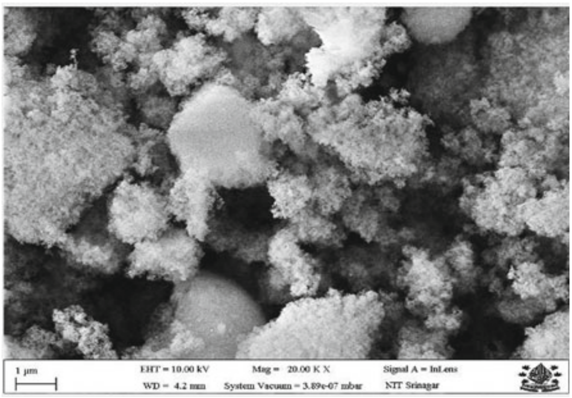


Fig. 2 Nanosilica in a SEM image

2, 3, 4, 5.5, and 6%) and mixed with the asphalt binder at 3000 rpm at a temperature of 180 °C [19]. The ideal mixing time was discovered to be 120 min using the softening point approach. Two stages make up the entire experimental strategy and are shown in Fig. 3.

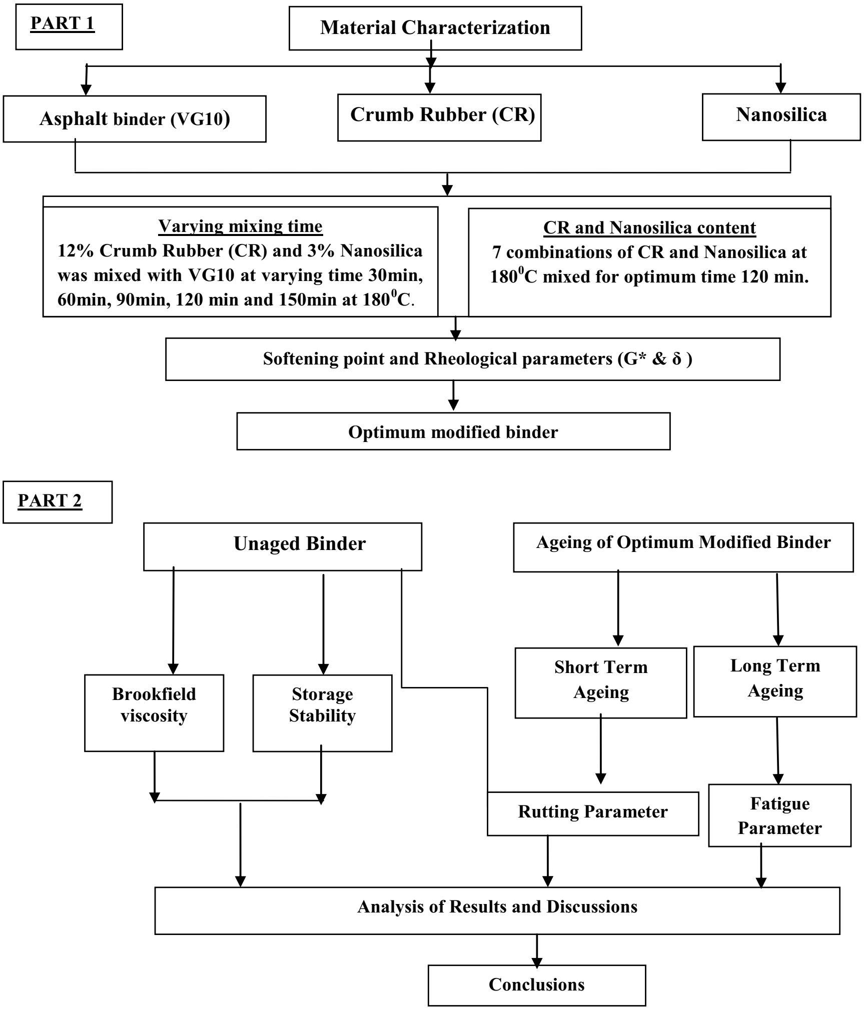


Fig. 3 Experimental flow chart

4 Test Procedures

4.1 Scanning Electronic Microscope (SEM)

The nanosilica powder and crumb rubber powder morphologies were examined using the SEM analysis.

4.2 Physical Characterization of the Binders

Utilising penetration (IS: 1203), softening point (IS: 1205), viscosity (AASTHO T316-13), and ductility (IS: 1208), the fundamental parameters of the base binder and crumb rubber-nanosilica modified binder were assessed.

4.3 Viscosity Test

In the Brookfield apparatus @ AASTHO T316-13, rotational viscosity was utilised to calculate viscosity at the temperatures for mixing and compacting. By measuring the torque needed to rotate the vertical shaft, the viscosity of the binder was ascertained. A chamber maintained at 135 °C with a spindle moving at a speed of 20 rpm was used to test 8–11 g of base and modified binders [31]. Assuming that 135 °C is the laying temperature, viscosity is computed at this temperature.

4.4 Rheological Characterisation of Binders

Using a dynamic shear rheometer (DSR) i.e., an Anton Paar MCR 102, the linear viscoelastic characteristics of unaged and aged bitumen were evaluated over a range of frequencies and temperatures in compliance with D7175 - 15. Complex modulus (G*), phase angle (δ), rutting parameter (G*/Sinδ), fatigue parameter (G*.Sinδ), T_{SHRP}, and ageing index were the parameters collected. A temperature range of 46, 52, 58, 64, 70, 76, 82, 88, and 94 °C was used, while an angular frequency of 10 rad/s was maintained (Bhat and Mir 2020). For fatigue characterization, the samples needed for testing were created in a silicon mould that was 2 mm thick, 8 mm in diameter, and had a gap of 2 mm between parallel plates. For rutting, however, a mould that was 1 mm thick, 25 mm in diameter, and had a gap of 1 mm was utilised. According to SHRP recommendations, aged samples are tested at a strain rate of 10%, whereas unaged samples are tested at a strain rate of 12%. For unaged binders and aged binders, the value of G*/sinδ is restricted to 1 kPa and 2.2 kPa, respectively.

Rutting resistance using Superpave rutting parameter: It is provided by G*/Sinδ and used to evaluate the asphalt binder's resistance to rutting. Rutting resistance will be stronger for an asphalt binder with a higher G*/Sinδ ratio because less energy will be lost during each cycle of loading.

Fatigue resistance using Superpave fatigue parameter: It is stated by G*.Sinδ and means that there will be less stress accumulation if there is less energy dissipated per cycle. As a result, materials with lower G*.Sin have superior fatigue resistance. The highest value of G*.Sinδ is 5000 kPa, according to SHRP. A 25 °C test temperature was used.

4.5 Ageing Process

Thin Film Oven was used to age the base and modified binders over a short period of time in accordance with ASTM: D1754. A moving film of the asphalt binder was heated in this oven for 5 h at 163 °C. This technique's primary objective is to quantify the impact of heat and air on a moving film of semi-solid asphaltic materials. Certain qualities are investigated before and after this test to ascertain the impact of ageing on the binders. "The resistance to hardening under the impact of air and heat is determined by short-term ageing. This apparatus simulates the circumstances that exist during the mixing and laying out of asphalt mixes" [21]. Pressure ageing vessels (PAVs) were used for the long-term ageing of base and modified binders in line with ASTM: D6521. 20 h at 100 °C and 2.1 MPa of air pressure were used for the Long Term Ageing experiment. It mimics the conditions in the field during a pavement's first 5–7 years of use.

4.6 Effect of Crumb Rubber and Nanosilica on Ageing Resistance

During the mixing and laying process as well as throughout their service life, binders oxidise. In addition to some of the lighter and more volatile components of the binder evaporating, unsaturated bonds present mostly in the aromatics and resin fractions undergo oxidation during the ageing process, rendering the binder brittle and perhaps contributing to pavement degradation. Consequently, binders with a reduced level of oxidation are desired. The ageing resistance of unmodified and modified binders is assessed in the current study utilising two parameters: ageing index and incremental softening point.

Softening point incremental (SPI): It is a difference between the age-related softening point of aged and unaged binders. For binders with strong ageing resistance, a lower value of the softening point incremental is necessary. Equation 1 serves as a representation.

$$SPI = SP_{aged} - SP_{unaged} \tag{1}$$

where SPI is softening point increment, SP_{aged} is the softening point of the short term aged binder and SP_{unaged} is the softening point of unaged binder [26].

Ageing index: The ageing resistance is calculated using the superpave rutting parameter, which also determines the ageing index. As the value of AI rises, so does the sensitivity to ageing. The temperature was maintained at 60 °C with a 10 rad/sec frequency. Equation 2 serves as a representation.

$$AI = \left|(G*/Sin\delta)_{aged}/(G*/Sin\delta)_{unaged}\right| \tag{2}$$

where $(G^*/Sin\delta)_{aged}$ and $(G^*/Sin\delta)_{unaged}$ are rutting factor parameter of the short-term aged and unaged asphalt binder respectively [21].

4.7 Storage Stability Test

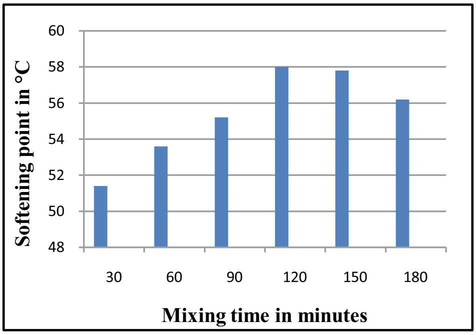
Because the modifier has a propensity to separate from the asphalt binder at higher temperatures, the storage stability test is performed to assess the stability of the modified asphalt binder. This test is used to determine whether the modifier and base binder have been blended evenly. The test's aluminium tube has a height of 140 mm and a diameter of 25 mm. In line with ASTM D7173-14, it was filled with the modified sample and then kept vertically at 163 °C for 48 h [11]. Two representative samples were obtained from the tube's top and bottom. According to [12], the modified asphalt sample is regarded to be high-temperature storage stable if the difference between the asphalt sample's softening point between the top and bottom portion is less than 2.5 °C.

5 Results and Discussion

5.1 Determination of Optimum Mixing Time

Using the Softening point, the optimum mixing time was discovered. At various mixing times (30, 60, 90, 120, 150, and 180 min), the softening point of the 12% CR-3% NS modified binder was discovered. Figure 4 illustrates the fluctuation in the softening point with varying mixing times. It is evident that as mixing time rises, the softening point rises till 120 min. Nevertheless, it falls when the mixing time is increased from 120 to 180 min. This is consistent with the results of Al-Mansob et al. [3], Cong et al. [10]. This is because prolonged mixing degrades the

Fig. 4 Variation of softening point with mixing time for 12% CR-3% NS modified binder



qualities of the binders, which may be linked to changes in the binders' physical and chemical properties. At 120 min, or 58 °C, the most softening point improvement was discovered. So, based on the softening point, we can say that 120 min is the optimum mixing duration.

5.2 Determination of Optimum Mixing Concentration

Figures 5, 6, and 7 display the results of the softening point test, complex modulus (G*), and phase angle (). Figure 5 illustrates how all modified binders have higher softening points than basic binders. They are therefore stronger and less prone to long-term deformation [36]. As demonstrated in Fig. 5, the softening point increases at constant CR concentrations up to 4% of nanosilica, while it decreases at lower concentrations. This is as a result of NS's ability to stiffen the binder and increase its temperature susceptibility [7]. According to other studies where it was found that an excessive amount of nanomaterials in nanocomposite modified asphalt binders may cause the elastic nature of modified asphalt binders to be destroyed, at a constant CR concentration, the softening point initially increases slightly and later remains unaffected [26]. The most flexible binder, with a softening point of 62 °C, is 12% CR-4% NS modified. Therefore, it might be said to be the ideal concentration.

Figures 6 and 7 illustrate the complex modulus and phase angle obtained from the DSR test for aged and unaged samples, respectively, at varied concentrations of NS and 12% CR. Figures 6 and 7 show that the G* value increases as the concentration of NS in the polymer modified binder increases, however it drops after 4% NS concentration. Thus, it demonstrates that the bituminous binder becomes stiffer as the NS content in PMB increases. Additionally, it is clear that as binders age, they stiffen and exhibit higher values for complex modulus and lower values for δ.

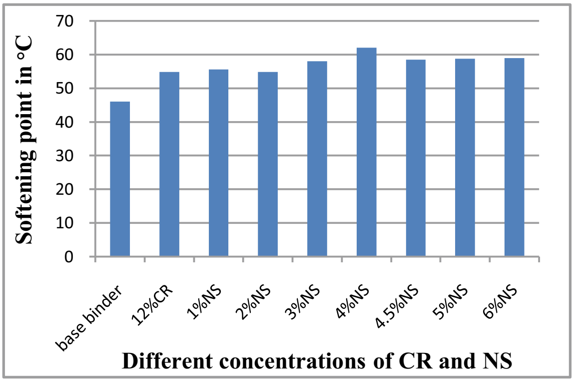


Fig. 5 Base and modified binders softening points

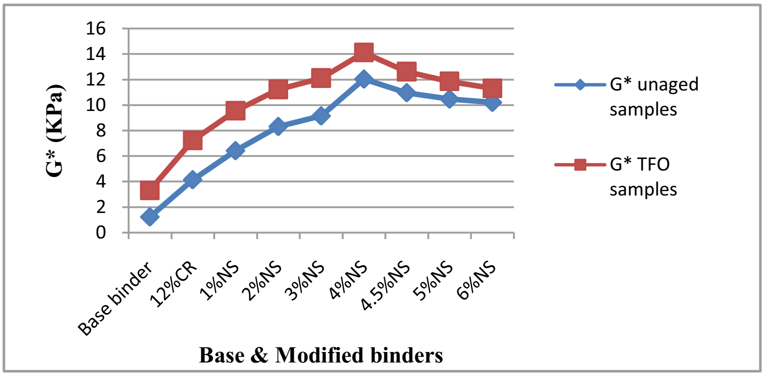


Fig. 6 Complex modulus of unaged and TFO samples at test temperature of 60 °C

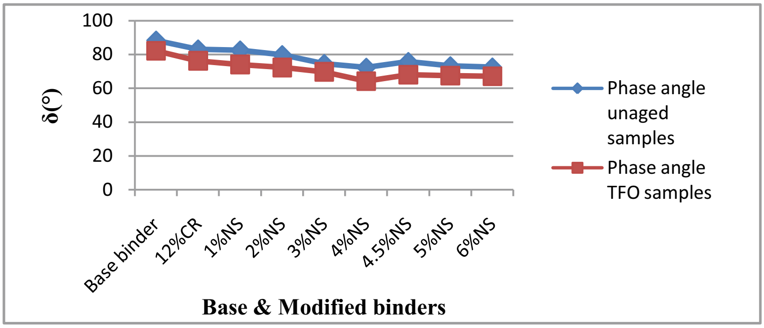


Fig. 7 Phase angle of unaged and TFO samples at test temperature of 60 °C

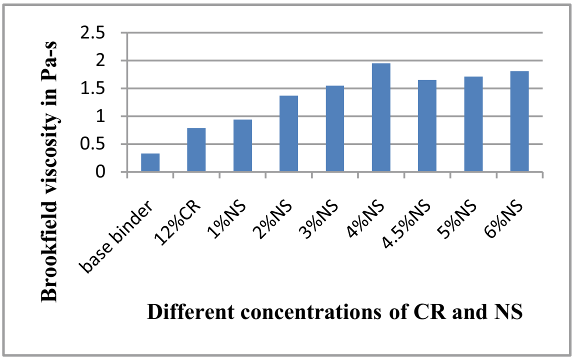


Fig. 8 Base and modified binders' Brookfield viscosities

5.3 *Determination of Brookfield Viscosity*

According to AASHTO T316-13, the Brookfield viscosity test is used to assess the viscosity of the modified and unmodified asphalt binder at 135 °C. Figure 8 shows that viscosity increases as NS concentration increases, which may be caused by the hardening impact of nanoparticles. The improved dispersion of the additional nanomaterial layers in the base binder may also be responsible for the modified asphalt binder's enhanced viscosity, which boosts the bonding strength by limiting the flow of asphalt [16]. Due to the modified asphalt binders' enhanced hardness, their increased viscosity suggests an improvement in their physical characteristics and an improvement in their high temperature qualities. Binders treated with nano-silica to have 12% CR-4% have the highest viscosity. At 135 °C, modified asphalt has a viscosity that is significantly greater than the base binder, this difference may be caused by the influence of polymers from the composition analysis of the modifier. As the extremely viscous binders are combined with the aggregates and create thicker films surrounding them, the resistance to water, environmental deterioration, and cohesive forces between the components rise [35]. Additionally, because the viscosity is not excessively high, there are no construction issues because binders with extremely high viscosities become hard and challenging to compact.

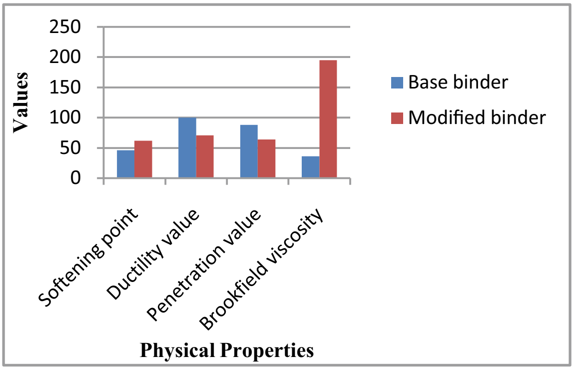
5.4 *Physical Properties of Optimum Modified Binder*

The physical characteristics of the 12% CR-4% NS modified binder are given in Table 5 and Fig. 9. As it can be seen from table, the softening point increases, ductility decreases and penetration value decreases as compared to the base binder, suggesting that the binder becomes stiffer. As a result, high temperature properties are enhanced due to addition of modifiers.

Table 5 Physical characteristics of 12% CR-4% NS modified binder

S.no	Test	Unit	Code	Value
1	Softening point	°C	IS:1205	62
2	Ductility value	Cm	IS:1208	71
3	Penetration value	0.1 mm	IS:1203	64
4	Brookfield viscosity	Pa-s	AASTHO T316-13	1.95

Fig. 9 Physical characteristics of optimum modified binder



5.5 *Rheological Characterization*

Complex shear modulus (G*): It provides a measurement of the binder's overall resistance to deformation when repeatedly sheared. The graph of all modified and unmodified samples (Fig. 10) demonstrates an exponentially declining trend in G* with rising test temperature. The G* value increased as the percentage of NS in CRMB grew from 0 to 4%, indicating that the modified asphalt binder is more stiff than the unmodified one. This indicates that NS-modified binders are slightly stiffer, more durable, and more resistant to deformation than unmodified binders. These results are consistent with findings of [1, 15] for NS. The nanocomposite has a greater complex modulus value, which indicates that a polymer network has formed in the changed binder. Furthermore, the complex modulus is falling as temperature rises, suggesting that this could drastically reduce the deformation resistance of both the base and modified binders. The 12% CR-4% NS modified binder has the highest value of G*, demonstrating the greatest improvement in rutting resistance. Furthermore, as seen from the charts, the complex modulus does not significantly change above 4% NS concentration.

Phase angle: Figure 11 depicts how the phase angle changes with temperature for base and modified binders. For the alteration of asphalt, phase angle measurement is typically thought to be more sensitive to chemical and physical structure than complex modulus [38]. Phase angle, according to Zheng et al. represents the proportional

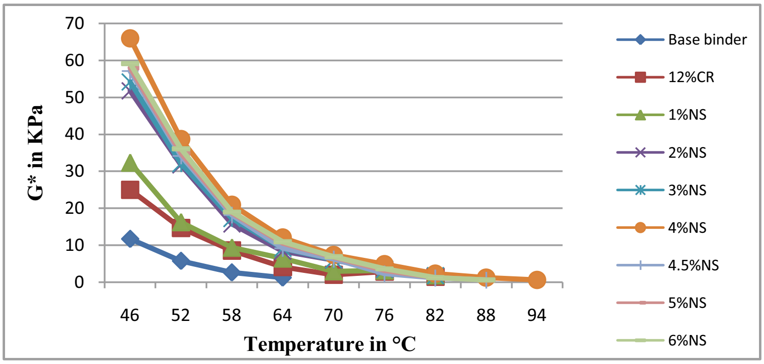


Fig. 10 Variation of G* with temperature for base and modified binders

distribution of the overall response between an in-phase (elastic) component and an out-of-phase (viscous) component in the shear dynamic loading mode. Figure 11 makes it obvious that the inclusion of CR-NS reduces the phase angle and, as a result, improves the elastic behaviour of the asphalt. Base binder begins to become viscous at higher temperatures and transforms into a Newtonian fluid when the phase angle approaches 90° [20]. Phase angle falls as NS content rises, indicating increased elasticity and raising the possibility for elastic recovery of pavement deformation. The modified binder with 12% CR and 4% NS has the smallest value for phase angle.

Superpave rutting parameter: Rutting parameter is thought to be a useful indicator of how well the improved binders prevent rutting. Figure 12 illustrates how different binders exhibit various rutting parameter values at a certain temperature. At all temperatures, 12% CR-4% NS showed the highest improved rutting metrics. It follows that adding NS can increase the asphalt binders' tolerance to high temperatures.

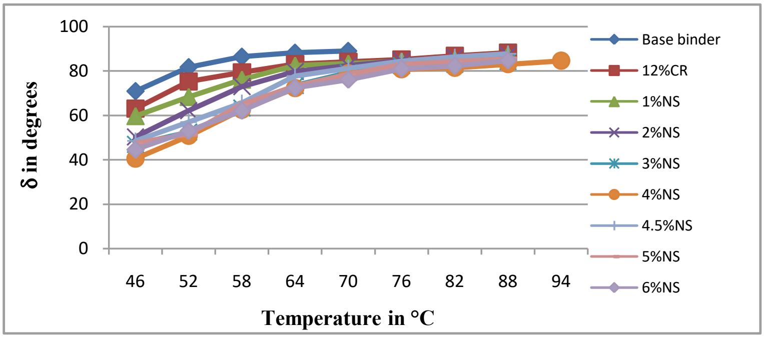


Fig. 11 Variation of phase angle with temperature for base and modified binders

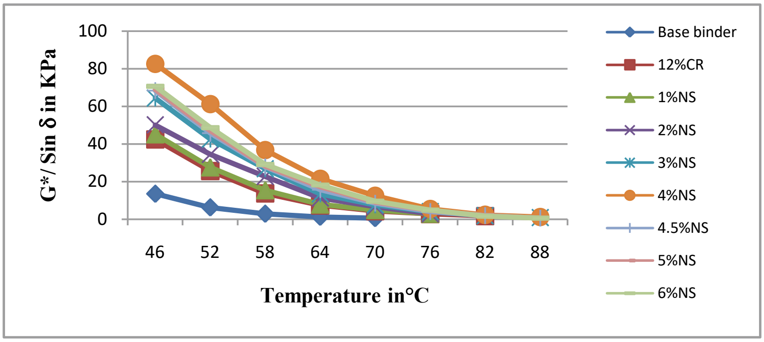
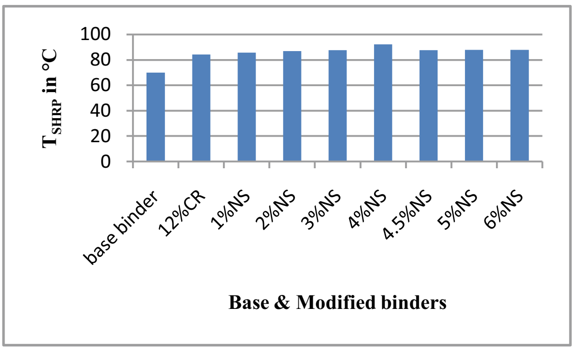


Fig. 12 Variation of superpave rutting parameters with temperature for base and modified binders

Failure temperature T_{SHRP}: According to Superpave binder grade requirements, it is referred to as the point at which G*/sin(δ) drops below 1.0 kPa. Road Materials and Pavement Design typically make use of this feature 13. Failure temperatures of modified and unmodified binders are used to determine the performance grade (PG) of asphalt binders [33]. Additionally, the base binder's failure temperature is 70 °C, which is the lowest. In Fig. 13, the T_{SHRP} values are displayed. The outcomes show that the NS-modified asphalt binders have better high temperature rheological characteristics. The asphalt binder had the maximum failure temperature at 4% NS in 12% CRMB. As can be observed from the graph, the failure temperature rises as NS concentration does, and this is due to NS functioning as the nanofiller in the CR matrix. The degree of NS dissipation in the polymer matrix of CR determines how much the mechanical characteristics will improve.

Fatigue Performance: Figure 14 illustrates the fluctuation of the fatigue parameter (G*.Sinδ) for modified and unmodified asphalt binders at 25 °C with various concentrations of CR and NS. According to [1], pavement microdamage is a result of fatigue damage brought on by repetitive bending stresses. Such distress causes stiffness and reduces the pavement's ability to withstand more pain. Additionally, an increase in

Fig. 13 Modified binders T_{SHRP} values



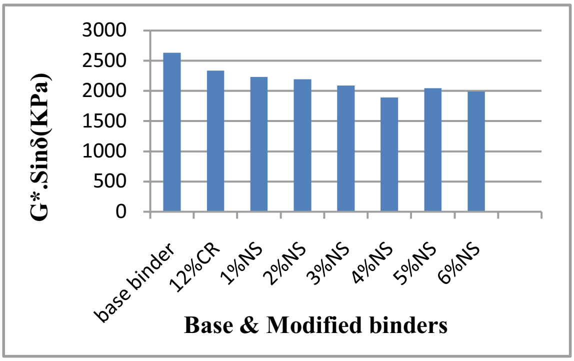


Fig. 14 Modified binders G*.Sinδ values

G*.sinδ was seen when NS was added at varied concentrations (with the exception of 4%), demonstrating the detrimental effect on fatigue failure. As a result, the addition of 12% CR-4% NS can improve the fatigue resistance performance at intermediate temperatures.

5.6 *Determination of Ageing Effect*

The impact of modifiers on the ageing resistance of the binders has been investigated using two parameters.

Softening point incremental: As binders age, they get tougher, hence the softening point of aged binders is higher than that of unaged binders. Figure 15 displays the values of the softening point incremental following short-term ageing. The resistance to ageing rises as the values of SPI fall with rising CR-NS concentration. The modified binder with 12% CR-4% NS shows the lowest SPI value.

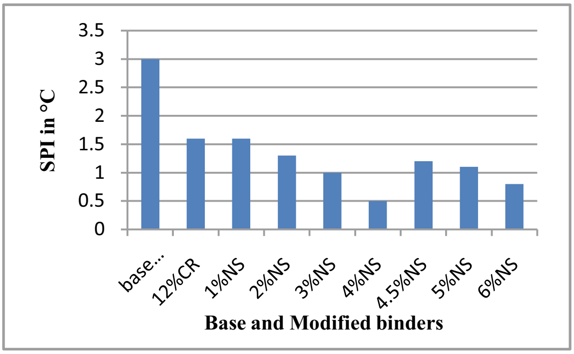


Fig. 15 SPI values for base and modified binders

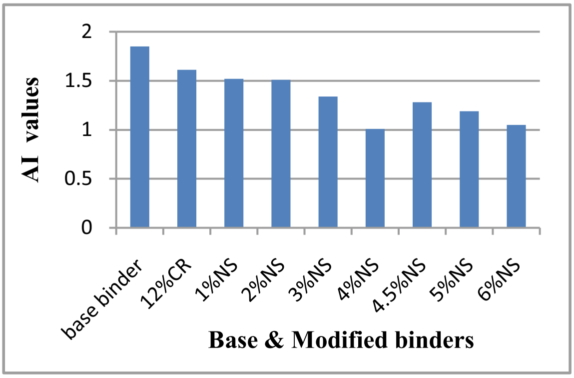


Fig. 16 AI values for base and modified binders

Ageing index: Figure 16 displays the results of the calculation of the rutting ageing index values at 60 °C and 10 rad/s frequency. The CRMB exhibits increased ageing resistance as CR percentage increases due to decreased carbonyl and sulphoxide indices, but as CR percentage increases, the likelihood of agglomeration also increases; in order to avoid this situation, as well as to increase ageing resistance, NS can be added to the CRMB [32]. Increased NS concentration has been observed to reduce the value of AI, indicating an improvement in oxidative stress resistance and ageing resistance. In terms of ageing resistance, NS is a good modifier (Bhat and Mir 2020). The 12% CR-4% NS modified binder once more demonstrates the lowest value of AI.

5.7 Storage Stability

Because the CR particles operate as a defect and the amount of stress that can be transferred across the interface is constrained, the modified asphalt is potentially unstable and requires a strong interface between the CR and the base asphalt. Here, NS serves as that interface because it can increase the bitumen's storage stability [7]. The internal characterization of this interaction is exceedingly challenging due to the complexity of the interaction between base asphalt and CR. In order to study the interaction, storage stability is utilised, and the result is established by measuring the softening point difference [9]. The sample is considered to have adequate storage stability when the softening point difference (SPD) between the top and bottom regions of the tube is less than 2.5 °C [12].

NS affects whether CR and asphaltenes are compatible. The copolymer blocks and inorganic filler interact when NS is added to CRMB, changing the microstructure. Table 6 shows that when the NS content increases, the difference between the softening points of the top and bottom regions of CR modified binder's diminishes. For 12% CR-4% NS, there is the smallest difference.

Table 6 Storage stability results of different combinations of CR-NS modified binders

S.no	Binders	Softening point difference between top and bottom of tube
1	12% CR modified binder	9
2	12% CR-1% NS modified binder	2.1
3	12% CR-2% NS modified binder	1.9
4	12% CR-3% NS modified binder	1.3
5	12% CR-4% NS modified binder	0.84
6	12% CR-4.5% NS modified binder	1.5
7	12% CR-5% NS modified binder	1.2
8	12% CR-6% NS modified binder	1.01

6 Conclusions

This research was done to create a modified asphalt binder containing CR-Nanosilica. This study assesses the impact of CR-NS on the modified binder's rutting, fatigue, and storage stability behaviour in Superpave. From the findings and conversations, the following conclusions can be drawn:

(1) The optimum mixing time was obtained as 120 min on the basis of the softening point method. The NS content was varied from 1% to 6% by weight of bitumen and CR concentration was taken as 12% by weight of bitumen. Thus, overall seven combinations of CR-NS were prepared. The best rheological performance was shown by 12% CR-4% NS.
(2) The Softening point increased by 34.7%, penetration and ductility decreased by 37.5% and 40.84% respectively as compared to neat asphalt. This demonstrates the increased stiffness and decreased temperature susceptibility of the CR-NS modified bitumen.
(3) High temperature rutting resistance improved as reflected by an increase in the value of complex modulus and Superpave rutting parameter. Complex modulus increased from 11.72 kPa to 65.98 kPa and Superpave rutting parameter increased from 13.57 kPa to 82.54 kPa at 46 °C (of neat and optimum modified asphalt respectively). It was also observed that as temperature increased from 46 °C to 88 °C, Complex modulus and Superpave rutting parameter decreased.
(4) The failure temperature of neat asphalt was obtained as 70 °C and that of optimum modified was 92.3 °C. Thus, CR-NS addition could improve the high failure temperature in the unaged binder state.
(5) CR-NS improves the elasticity of the binders as the value of phase angle decreases. The value of phase angle for neat asphalt was 71° and that of optimum modified was 40.55° indicating a decrease of 42.88% and thus an increase in elasticity.

(6) NS plays an important role in increasing the ageing resistance of CRMB which can be derived from the fact that the softening point incremental and ageing index decreases due to the addition of CR-NS. The SPI of neat asphalt was obtained as 3 °C and minimum value was obtained for optimum modified bitumen i.e., 0.5 °C. Also, ageing index of neat asphalt was 1.85 and minimum value was obtained for optimum modified bitumen i.e., 1.01.
(7) The viscosity of CR-NS modified bitumen increases as NS content increases and maximum value is shown by 4% NS concentration, added to 12% CRMB which is 1.95 Pas.
(8) The results of Superpave fatigue parameter showed that there is an improvement in the fatigue resistance as G*.Sinδ value decreases as NS concentration increases upto 4%. Hence, the intermediate temperature performance is enhanced and maximum enhancement is shown by 12% CR-4% NS for which the value was obtained as 1890 kPa.

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Utilization of Waste Polyethylene in Open Graded Friction Course

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Abstract Apart from resolving the environmental issues, waste polyethylene has found its utility in the pavement construction industry, thereby achieving economy and sustainability. This paper has attempted to explore the use of waste polyethylene derived from milk packaging pouches as an additive in bituminous paving mixes applicable for the surface courses. The open graded friction course (OGFC), especially recommended for rut resistance and friction development, has been considered in this study. Unlike National Centre of Asphalt Technology (NCAT) guidelines for OGFC, IRC-37 (2018) recommends higher grade bitumen for bituminous mix used in surface course. Accordingly, this laboratory study is based on mix design as per NCAT procedure for the mix with viscosity graded VG 40 bitumen, with its concentration varying from 4 to 6% by weight of the mix. The waste polyethylene in the form of small fibers was introduced at a rate of 0, 4, 6, and 8, by total weight of bitumen and mixed with the aggregate as a dry process before the bitumen was added and all ingredients mixed almost homogeneously. The optimum bitumen content (OBC) and optimum polyethylene content (OPC) of OGFC mixes were determined based on the drain down test, Cantabro loss test, and moisture susceptibility test. Based on the above test results, the mixture prepared with 6% waste polyethylene satisfied the important criteria set for the mix design. Hence, it is concluded that waste polyethylene can be effectively utilized for improving the performance of OGFC.

Keywords Waste polyethylene · Open graded friction course · VG-40 bitumen · Optimum bitumen content · And Optimum polyethylene content

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1 Introduction

Bituminous mixtures are commonly used in the construction of flexible pavements all over the world. As most of the conventional bituminous paving mixtures fail due to rapid increase in traffic volume with consequent increase in traffic load on the pavement, and due to extreme climatic conditions, it becomes imperative to go for improvement in the properties of bituminous mixtures by using additives or modifiers.

Out of various additives, the polymers including polyethylene not only easily mix with bitumen but also significantly improve the mechanical properties of the bituminous mixture [1]. The effects of use of waste plastic in bituminous mixtures is almost similar to that with the polymer modified bitumen. Bituminous mixtures prepared with the polymer modified bitumen as a binder, have better stability, fatigue life, resistance to moisture, resistance to thermal cracking and resistance to rutting, as compared to the mixture prepared with conventional bitumen [2, 3]. Addition of this plastic waste not only gives solution to the waste disposal problem, but also increases the durability of the paving mixture, as a result the service life of the bituminous pavement is extended. From various literatures, it was found that the polymer modified bitumen is produced by the wet process in which waste plastic by weight of bitumen is mixed with conventional bitumen at a higher temperature and the mix is stirred vigorously for a certain time. Thus, it improves the property of conventional bitumen [4]. This process has many disadvantages as it requires a specialized mixing plant. But now-a-days researchers try to avoid this method as it requires huge fund and also emits harmful gases. Unlike the wet process, the dry process also involves mixing of the aggregate and binder along with the additive mixed at a specified temperature, which is safe and economical.

In India, every year million tons of Low density polyethylene (LDPE) waste plastic are generated and very few are recycled. In the dry process, LDPE or plastic waste is mixed at a required temperature which forms a film around the aggregate surface resulting in interlocking bonds between aggregates which improve the performance of the bituminous mixture. Many researchers have already investigated on this dry process in the bituminous mixtures like the dense graded mixture and dense bituminous macadam [5, 6].

Though use of LDPE in bituminous mixtures is not new, still more research is needed to improve the properties of OGFC bituminous mixtures with LDPE waste polyethylene as an additive. OGFC is a special bituminous mixture having 18–20% air voids which is produced by lowering the fine aggregate and filler. Hence, for proper interlocking between aggregates it is necessary to have a high viscous binder like polymer modified binder or polymer stabilizing additive along with conventional bitumen [7, 8]. OGFC mixtures prepared by using modified binders and reclaimed polyethylene fibers (0.3% by weight of total mixture) resulted in significant improvement in engineering properties when compared with the conventional bituminous mixture [9]. On the other hand, for surface course of the bituminous mix used in high volume road like expressway and national highway, the Indian Road Congress

(IRC:37-2018) has recommended a higher viscosity grade bitumen (such as VG 40) for preparation of the mix [10]. Similarly, the National Centre of Asphalt Technology (NCAT) (2000) has suggested hard grade or modified bitumen for preparation of the OGFC mix for better performance in terms of drain down and aggregate interlocking.

The previous research studies mostly focused on using the wet mixing process for the preparation of OGFC mixture that used lower grade bitumen such as viscosity grade, VG 10 and VG 30 bitumen. However, no study has been reported on OGFC mixes involving the dry mixing process using a higher grade paving bitumen. Thus, this study basically investigates the compatibility of waste polyethylene in the form of used milk pouches (by weight of bitumen) as an additive in OGFC bituminous mixtures using VG 40 following the dry mixing process.

1.1 Objective

The objective of this study is to evaluate the OGFC bituminous mixtures using VG 40 as the binder and waste polyethylene as an additive following the dry mixing process.

2 Materials and Methods

2.1 Materials Used

Aggregate

For preparing the OGFC mixture, National Centre of Asphalt Technology (NCAT) (2000) gradation was followed where aggregates with nominal maximum aggregate size (NMAS) of 12.5 mm were considered as presented in Table 1 [7, 8].

Table 1 Adopted aggregate gradation for OGFC Mixture (NCAT-2000)

Sieve size (mm)	Percentage passing	
	Specified	Adopted
19	100	100
12.5	85–100	92
9.5	55–75	65
4.75	10–25	19
2.36	5–10	8
0.075	2–4	3

The physical properties of the coarse aggregates used are given in Table 2. Fine aggregates comprising of the stone crusher dusts had specific gravity of 2.65. Ordinary Portland cement (OPC) of 43 grade was used as the filler material (passing 0.075 mm IS sieve). Its specific gravity was found to be 3.01.

Bitumen

In this experimental study, VG 40 bitumen is used as the binder to form hot bituminous mixtures (OGFC). The important physical properties of the bitumen used are summarized in Table 3.

Stabilizing Additives

Low density polyethylene (LDPE) (Fig. 1) material is required in huge quantity for milk packaging. After use, these packaging bags become solid waste and cause disposal problems. Hence, for eco-friendly environment and to enhance the engineering properties of the OGFC mixture, reclaimed LDPE in the form of the above packaging bags collected from tea stall, and then the process was followed as per IRC: SP: 98-2013 recommended guidelines so that it can be used as the additive in

Table 2 Physical properties of coarse aggregate

Properties	Test methods	Test results	Requirement as per MoRTH (2013)
Aggregate impact value (%)	IS: 2386 Part IV [11]	14.3	<30%
Aggregate crushing value (%)	IS: 2386 Part IV [11]	13.02	<30%
Los Angeles abrasion value (%)	IS: 2386 Part IV [11]	18	<40%
Flakiness index (%)	IS 2386 Part I [12]	19	<30%
Elongation index (%)	IS 2386 Part I[12]	21	<30%
Specific gravity of coarse aggregate	IS: 2386 Part III [13]	2.7	–
Water absorption (%)	IS: 2386 Part III [13]	0.1	<2%

Table 3 Physical properties of VG 40 bitumen

Properties	Test methods	Test results	Requirement as per IS: 73-2013
Penetration at 25 °C (0.1 mm)	IS:1203 [14]	50	Min 35
Specific gravity		1.03	Min 0.99
Softening point (°C)	IS:1205 [15]	50	Min 50
Absolute viscosity at 60 °C (Poise)	IS:73 [16]	3308	3200–4800
Kinematic viscosity at 135 °C (cSt)		506	Min 400

Fig. 1 Waste plastic from milk packaging

the OGFC mixture [17]. Low density polyethylene which is passing through 4.75 mm IS sieve is taken as the stabilizing additive whose specific gravity is 0.90 and melting point 130 °C.

2.2 Preparation of Samples

Mixture design procedure specified in ASTM D 7064 has been used for the preparation of OGFC mixture samples. The coarse aggregate, fine aggregate and cement in appropriate quantities as per adopted aggregate gradations presented in Table 1 were heated up to the required temperature [18]. The heated aggregate is then mixed with a specified quantity of hot bitumen (at a specified temperature). Thereafter, the required amount of waste polyethylene is added uniformly and gradually followed by thorough mixing of the ingredients. Mixing was subsequently continued with the addition of hot bitumen and the ingredients were mixed uniformly and thoroughly to achieve the required mixture consistency. Compaction was done using a compactive effort of 50 gyrations in a Superpave gyratory compactor (SGC) [8]. The bitumen contents in the mixtures were varied (4–6% by weight of total mixture) to determine the optimum bitumen content (OBC). According to IRC SP: 98 (2013), the dosage for waste plastics is 6–8% by weight of bitumen in case of the dry process. Considering the said facts, dosages of waste polyethylene were considered as 0, 4, 6 and 8% by weight of bitumen for possible improvement in the properties of the OGFC bituminous mixture to determine the optimum polyethylene content (OPC) for the resulting best possible mixture.

2.3 Tests Performed

The following tests were performed for determining the optimum binder content (OBC) and optimum polyethylene content (OPC) of OGFC mixtures.

Cantabro Loss Test

This test is suggested by ASTM D 7064 [8] that helps to determine the abrasion loss of compacted porous asphalt specimen by using Eq. (1). Aged compacted samples were subjected to the Cantabro abrasion test to evaluate the effect of accelerated laboratory aging on resistance to abrasion. For this, the specimen has to be kept in a forced draft oven at 60 °C for 7 days and then cooled to a temperature of 25 °C for 4 h, and then the test is performed.

$$CL = \frac{A - B}{A} \times 100 \tag{1}$$

where, A = Initial weight of specimen, B = Final weight of specimen, CL = Cantabro Loss (%).

Drain Down Test

The drain down test procedure was followed as per AASHTO T 305 (1997) [19]. This test helps to assess the extent of the drain down from a OGFC mixture likely to happen in the field. From the drain down test, the binder drainage was calculated by following Eq. (2).

$$\mathrm{D} = \frac{\mathrm{W2} - \mathrm{W1}}{1200 + \mathrm{X}} \tag{2}$$

where, W1 = Initial mass of the plate, W2 = Final mass of the plate and drained binder, X = mass of waste polyethylene in the mixture.

Here, the value of X was taken as 0 gm as the total weight of the mix is 1200 gm including weight of polyethylene.

Moisture Susceptibility Test

It is very much essential to study the resistance to moisture resistance characteristics of bituminous mixtures as moisture is a critical factor leading to the failure of bituminous pavements. The moisture susceptibility or resistance to moisture damage of the OGFC mixtures was evaluated by the wet abrasion loss (WAL) test. Based on the mode of deterioration in the OGFC mixtures, it is appropriate to evaluate the moisture susceptibility of the above mixtures by WAL approach. The reason is, in general, the tensile strength tests are performed to assess the cracking potential of the dense asphalt mixture, while the Cantabro abrasion tests are performed to assess the abrasion resistance (or resistance to particle loss) of compacted open graded asphalt mixtures. The procedure of moisture conditioning of the compacted OGFC bituminous mixture specimens was according to AASHTO T 283 [20]. However, based on the Indian Roads Congress recommendations only one freeze thaw cycle is adequate for the moisture conditioning of OGFC bituminous mixtures. Researchers found that wet abrasion loss on specimens should not exceed 30% [7, 8].

3 Test Results and Discussion

Results of various tests of OGFC mixes with varying bitumen contents and waste plastic contents are presented below.

3.1 *Volumetric Properties*

Figure 2 shows variations in air voids with respect to bitumen contents. It can be observed that with increase in bitumen contents, air voids percentage decreases. Also, it is found that increase in waste polyethylene contents caused a decrease in air voids. This may be explained as the waste polyethylene fills the gaps between aggregates and makes the sample stronger. Figure 3 shows variations in voids in mineral aggregate (VMA) with respect to bitumen contents.

3.2 *Cantabro Loss Test*

Figure 4 shows variations in unaged abrasion loss with respect to bitumen contents. As the binder contents and waste polyethylene contents increase, the air voids decrease, and the interlocking between aggregates increases. Addition of waste polyethylene increases the viscosity of the bitumen and therefore also increases the stiffness of the mixture. As a result, the abrasion loss decreases. Similarly, with increase in waste polyethylene contents, the aging of bitumen also reduced. This implies the aged abrasion loss also reduces as demonstrated in Fig. 5.

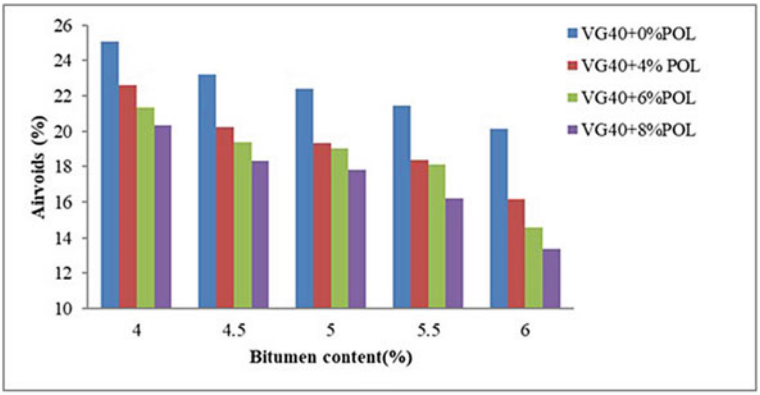

Fig. 2 Air voids versus bitumen contents

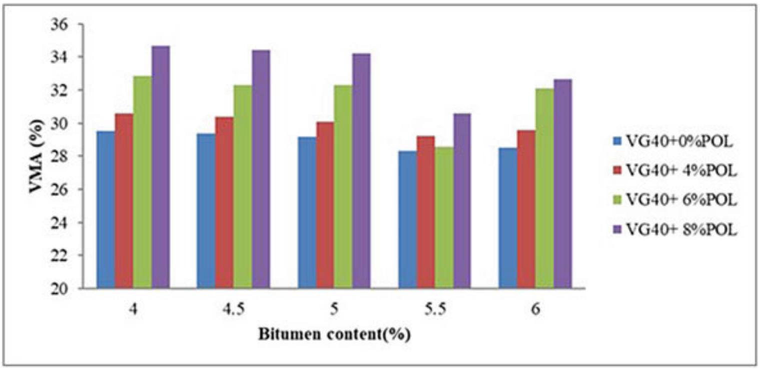

Fig. 3 VMA versus bitumen contents

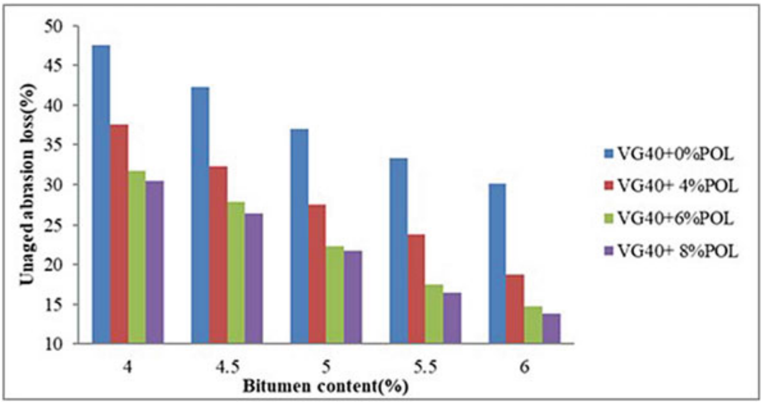

Fig. 4 Un-aged abrasion loss versus bitumen contents

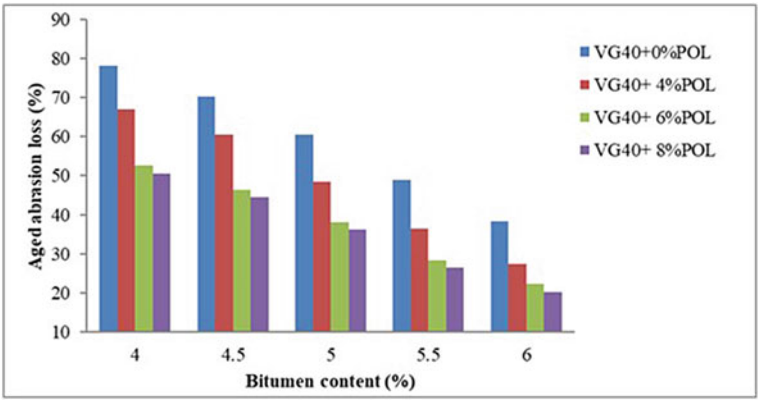

Fig. 5 Aged abrasion loss versus bitumen contents

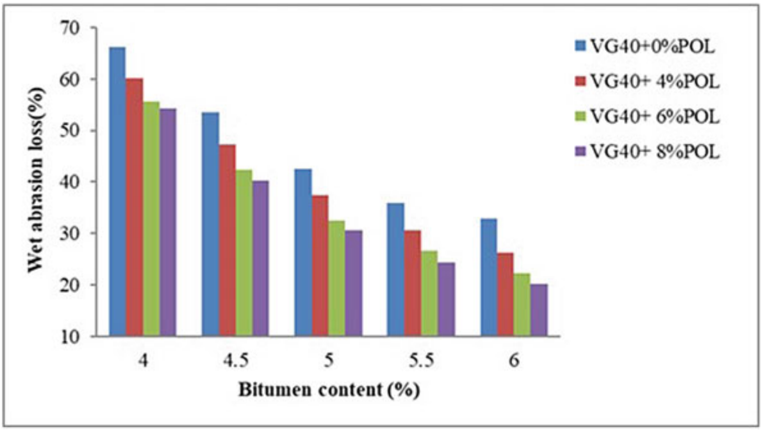

Fig. 6 Wet abrasion loss versus bitumen contents

3.3 *Moisture Susceptibility Test Results*

Figure 6 presents the moisture susceptibility characteristics of bituminous mixtures with respect to various bitumen contents. It can be observed that the wet abrasion loss percentage decreases with increase in the bitumen content. It can be attributed to the increased quantity of waste polyethylene and bitumen content in the mixture. The bitumen and polyethylene combinedly make a film around the aggregate which increases the moisture resistance of the mixture.

3.4 *Drain Down Test Results*

Figure 7 shows variations in the drain down of bitumen from the loose mixture with respect to bitumen contents. By adding waste polyethylene the drain down decreases as the mix becomes stiffer.

3.5 *Determination of OBC and OPC*

The OBC of the OGFC mix should be satisfied bt the following criteria [7, 8]: a minimum of 18% Air void, abrasion loss on unaged specimens should not exceed 20%, abrasion loss on aged specimens should not exceed 30%, abrasion loss on moisture conditioning specimens should not exceed 30%, maximum drain down of 0.3% by the total mixture mass. Here the OGFC mix prepared with 5.5% VG-40 bitumen containing 6% of LDPE satisfied all of the above criteria. However, the

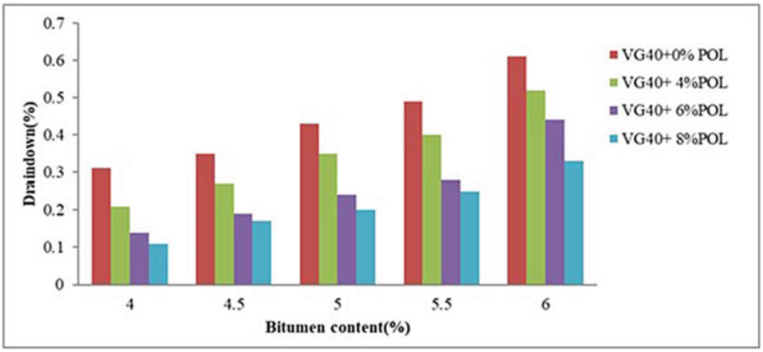

Fig. 7 Draindown value versus bitumen content

Table 4 Characteristics of OGFC bituminous mixtures at OBC and OPC

Properties	Values at 6% OPC	Recommended values (%)
OBC (%)	5.5	–
Air void (%)	18.2	18–20
UAL (%)	17.54	20
AAL (%)	28.49	30
WAL (%)	26.56	30
Draindown (%)	0.28	0.3

mix prepared with a higher bitumen content and higher percentage of LDPE as the stabilizing agent though satisfy criteria such as abrasion loss in aged, unaged and wet condition but failed in air voids containing the compacted sample (which is less than 18%). AS a result these samples fail to satisfy the permeability criteria. It is recommended that the laboratory permeability value should greater than 100 m/day. Hence OBC and OPC of OGFC mix is 5.5 and 6%. As per ASTM D 7064 guidelines, OBC of mixtures prepared with VG 40 is found to be 5.5% and OPC is 6% by weight of the mixture and bitumen, respectively. The detailed properties of the mixture at its OBC and OPC are given in Table 4.

4 Summary

The important conclusions of the study are outlined below.

- With increase in the binder content, air voids of the mixture decrease. Similarly, with increase in the polyethylene content, air voids decrease. As a result, the abrasion loss also decreases.

- The optimum bitumen content (OBC) and the optimum waste polyethylene (OPC) content in the mixture prepared with VG 40 bitumen are found to be 5.5% and 6% by weight of the mixture and bitumen, respectively.
- With addition of 6% waste polyethylene by weight of bitumen in the mixture, the drain down value decreases and all other results satisfy the recommended value as per ASTM D 7064. Hence OPC was considered to be 6% by weight of the bitumen.
- Besides, the addition of 6% waste polyethylene (by weight of bitumen) improves the moisture susceptibility characteristics.
- Hence, it is concluded that the waste polyethylene which otherwise remains as a waste after its single use, can be effectively utilised in a bituminous surfacing of a pavement.

5 Future Scope

Other performance tests such as permeability and rutting test of the mixture need to be conducted for further evaluation.

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Rheological Characteristics of Waste Engine Oil-Modified Bituminous Binder

T. Srikanth, R. Amal, and M. Sivakumar

Abstract Bituminous binders encounter different distress during the life cycle of flexible pavement construction. Fatigue cracking and rut resistance are the most important characteristics of the binder. Addition of environmental-friendly modifiers/additives into the binder improves the rheological properties of bituminous binders. In this study, the effect of waste engine oil (WEO) on rheological characteristics of plain bitumen (VG30) as an additive was assessed through laboratory investigations. Also rutting and fatigue performance characteristics of the modified bitumen were investigated using Dynamic Shear Rheometer. From the Multiple Stress Creep Recovery test, the percentage recovery and Non-recovery creep compliance at different dosages of additive were evaluated. Linear Amplitude Sweep to determine $G*/\sin\delta$ and $G*\sin\delta$ indices for the bio-additive modified bitumen containing different percentages of WEO. The chemical properties of waste engine oil were identified by conducting Fourier Transform Infra-Red Spectroscopy. Overall it was found that WEO can be used as a softening agent to the aged bituminous binder.

Keywords Waste engine oil · Fatiguelife · Rutting · DSR

1 Introduction

Due to its first-rate driving comfort. Flexible pavement has won recognition and presently 90% of the world's highways are flexible pavements. Currently, there is an increase in concern about re-utilising waste materials and minimising the use of natural resources in the construction of flexible pavements. A turning point has been reached in pavement construction due to the growing need for environmentally friendly development. The hot mix bitumen has been widely used across the world. The life of the pavement depends on the loads which are passing through it and the environmental conditions. But, today the production of heavy vehicles with more axle loads and high urban and rural traffic has accelerated pavement failure. Along

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with these reasons, changes in the characteristics of bitumen due to environmental effects also lead to the deterioration of pavements. Bitumen should not be hard at low-temperature conditions so that cracks will not develop, also it should not be soft enough at high temperatures so that it can resist the rutting due to wheel loads. These parameters are mainly depending upon the rheological properties of bitumen.

Bitumen should not be hard at low-temperature conditions so that cracks will not develop, also it should not be soft enough at high temperatures so that it can resist the rutting due to wheel loads. These parameters are mainly depending upon the rheological properties of bitumen. Adding softening agents/additives can reduce bitumen binder viscosity, thereby lowering mixing temperature and resulting in lower energy consumption and greenhouse gas emissions. Overall, compared to plain bitumen, modified bitumen has better financial, social and environmental advantages. The present study discusses the effect of waste engine oil as an additive on rheological characteristics and performance assessment of the bituminous binder.

2 Literature Review

Liu et al. [1] investigated the effect of Waste Engine Oil (WEO) on the rheological characteristics of the bituminous binder. The findings showed that WEO-modified bitumen had increased temperature sensitivity and fatigue resistance but decreased elastic portion, heat sensitivity, zero shear viscosity, and anti-rutting ability. Jia et al. [2] demonstrated that addition of waste engine oil up to 5.0% by weight of the bitumen binder improved the rheological characteristics of aged bitumen. Hesp et al. [3] discovered that rejuvenated bitumen with a WEO level above 15% might perform poorly in low temperatures. A comprehensive study on the effect of waste engine oil on temperature susceptibility characteristics of the bituminous binder becomes inevitable. The effect of WEO on the rutting and fatigue characteristics of the bituminous binders also needs to be thoroughly investigated. In the present study, plain bitumen of VG30 grade is used as a binder, since they are most widely used in PWD roads of Kerala State and waste engine oil collected from the motor vehicle workshop as an additive to the bitumen by the weight binder. The present study considers only a single source WEO additive in order to avoid the variation in WEO constituents and its properties.

2.1 Objectives of the Study

This study aims to enhance the rheological characteristics of the bituminous binder by adding waste engine oil as a softening additive. The objectives of the study are assessing the physical and mechanical properties of the bitumen at different dosages of WEO; obtaining the optimum dosage of WEO content to improve its rheological characteristics of the bituminous binder which results in high rut-resistance and more

Table 1 Physical properties bituminous binder and WEO

Property	Plain bitumen (VG 30)	WEO content (%)			
		1	2	3	4
Penetration value, 0.1 mm, at 25 °C	73.3	92.0	107.0	156.3	162.3
Softening point, °C	42.7	42.1	41.6	39.1	38.0
Ductility, cm	>100	>100	88.7	82.7	76.0
Rotational viscosity, cPs at 135 °C	353.33	256.7	220.8	198.3	167.5
Properties of WEO	Test results				
Specific gravity	0.95				
Viscosity (cPs)	132				

fatigue life. These properties were investigated in the laboratory using Dynamic Shear Rheometer (DSR). The chemical property of the additive was determined using Fourier Transform Infra-red Microscopy.

3 Materials and Methods

3.1 Materials

Plain bitumen of VG30 grade was used as the binder. Waste engine oil which was collected from the local motor vehicle workshop was used as an additive to the binder. WEO is kept in the glass beaker for the duration of 48 h to allow the solid particles settle down. After 48 h, WEO was filtered with filter paper in order to separate the unwanted coarse particles in the oil. The physical properties of plain bitumen and WEO were tested by conventional laboratory tests and the results obtained are reported in Table 1. Since the specific gravity of WEO ranges from 0.9 to 0.96 [4] which is less than one, it floats on the water. The viscosity of WEO ranges from 120 to 150 cPs [5]. The main goal of this study is to evaluate the effect of WEO on the physical, mechanical, chemical and rheological properties of the bituminous binder. In order to achieve these objectives the following methodology was adopted in this study.

3.2 Preparation of WEO Modified Bitumen

The Waste engine oil was mixed with plain bitumen using mechanical shear mixer at the rate of 4000 rpm. WEO was mixed with bitumen at different dosages such as

1, 2, 3 and 4% by weight of bitumen at 135 °C [6] for the duration of 10 min. The prepared sample is kept 24 h for aging. Based on the physical properties of WEO modified bitumen (WEOMB), the various tests were conducted and compared with the plain bitumen (VG30). By comparing the test results, optimum dosage of WEO was obtained and recommended for further investigations on mix properties and for the construction of flexible pavement.

3.3 Physical Properties

The physical properties of the bituminous binder with and without WEO additive were assessed by conducting laboratory test such as, penetration test at 25 °C (IS:1203) [1], softening point test (Ring and Ball test, IS:1205) [1], ductility test (IS1208), specific gravity test at 27 °C (IS:1201) [1] and rotational viscosity test at 135 °C (ASTM D4420) [7].

3.4 Rheological Characteristics

The rheological characteristics like rutting resistance and fatigue cracking were determined by using Dynamic Shear Rheometer (DSR) as per AASTHO T315. In this, Multiple Stress Creep Recovery (MSCR) as per AASTHO M 350 and Linear Amplitude Sweep (LAS) tests was conducted as per AASHTO T101-14 [8]. From MSCR, percent recovery (%R) and Non-recoverable creep compliance (J_{nr}) values were determined. At the same stress level, less the Jnr value and more % R means that the rut resistance of modified bitumen is high. As per AASHTO TP70 and AASHTO M322 codes, MSCR test and results calculation at a default temperature was carried out by using 25 mm diameter parallel plate with 1 mm gap between plates. In LAS test, frequency sweep and linear amplitude sweep are simultaneously performed on the modified bitumen. From the frequency sweep test, complex shear modulus (G*), phase angle (δ) had been evaluated and registered at every frequency. Then the rheological characteristics, fatigue parameter ($G^*\mathrm{Sin}\delta$) and rutting parameter ($G^*/\mathrm{Sin}\delta$) were calculated. Test was performed at three specified temperatures 40, 50 and 60 °C by using the 8 mm diameter parallel plate with 2 mm gap between plates. $G^*/\mathrm{Sin}\delta$ is the rutting parameter; more the $G^*/\mathrm{Sin}\delta$ value indicates higher rutting resistance. $G^*\mathrm{Sin}\delta$ is the fatigue parameter, the lesser $G^*\mathrm{Sin}\delta$ value indicates more fatigue life of the bituminous binder.

3.5 Storage Stability

The storage stability of the WEO modified bitumen was determined by conducting a separation test. The ring and ball (softening point) test on samples was taken from the top and bottom portion of the conditioned WEO modified bitumen, in enclosed aluminium tubes in a vertical position, in an oven for a period of 48 h at 163 ± 5 °C.

3.6 Fourier Transform Infrared Spectroscopy (FTIR)

FTIR is a technique used to obtain the infrared spectrum of absorption or emission of a solid, liquid or gas. Each molecule or chemical structure will produce a unique spectral fingerprint, which can be used as a tool for the identification of chemical bonds or functional groups present in a molecule. Source emits radiation that goes through the interferometer and into the detector, passing through the sample. Amplifier and analog-to-digital converter then amplify and convert the signal to a digital signal, respectively. The signal is then sent to a computer, where the Fourier transform is performed.

3.7 Scanning Electron Microscopy (SEM) Analysis

Scanning Electron Microscope (SEM) scans a concentrated electron beam across a surface to generate a high magnified image. The electrons in the beam inter-react with the sample material and creating different signals which can be used to acquire surface topography and composite data of the sample. It provides the particle's shape, size, and texture. First, a small amount of sample which was for analysis was taken and placed on the sample stub. The SEM chamber was vented which allows the chamber to reach nominal pressure, and the sample was placed into the chamber and closed. After sample insertion, it will be auto-focused and acquire a focused picture of the sample given.

4 Results and Discussions

4.1 Physical Properties

The penetration value for WEOMB increases with increase in WEO content than plain bitumen (VG30). Maximum penetration value was obtained at 1% WEO content. It is continuously increasing with increasing additive percentage. However, the binder becomes soft with the addition of waste engine oil due to liquid form.

In the case of the softening point for WEOMB, it decreases with increase in waste engine oil content. Since the WEO act as a softener to the bituminous binder as well as the low viscosity value gives a low softening value. Maximum softening point at 1% WEOMB is compared with VG30.Ductility of WEOMB decreases with increase in WEO content but comparing with VG30 they all give better ductility. Specific gravity for WEOMB is less than bitumen, while testing WEOMB is floating on the surface of the water. Rotational viscosity of WEOMB decreases with WEO content increase, at 1% WEOMB content which gives maximum value of viscosity as compared with VG30.

4.2 Multiple Stress Creep Recovery (MSCR)

The MSCR results shows that for WEOMB, percent recovery (%R) and non-recoverable creep compliance (Jnr) had decline of rut resistance at 0.1, 3.2 kPa loadings. For WEOMB, percent recovery is decreased and Jnr value is increased with increase in WEO content. Hence, it is not showing rut resistance. The %R and Jnr values graphs for WEOMB are shown in Fig. 1.

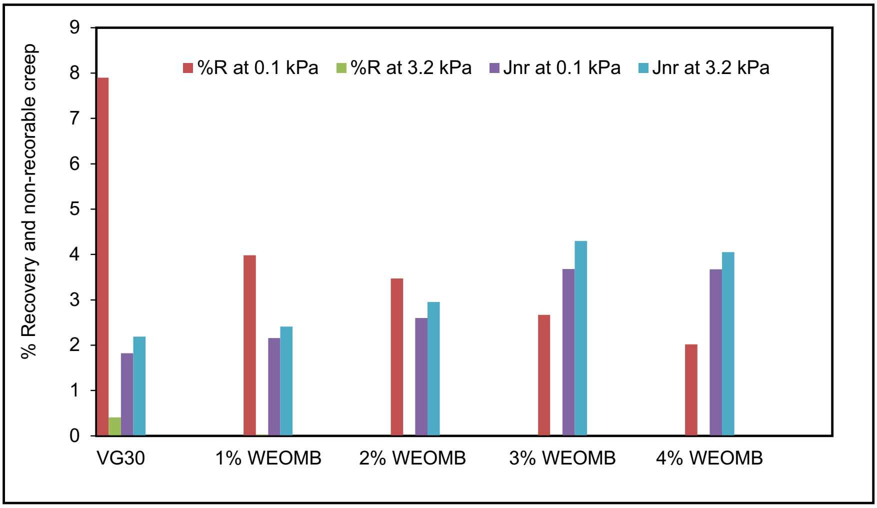

Fig. 1 Percent recovery (%R) and non-recovery creep (J_{nr}) values for WEOMB

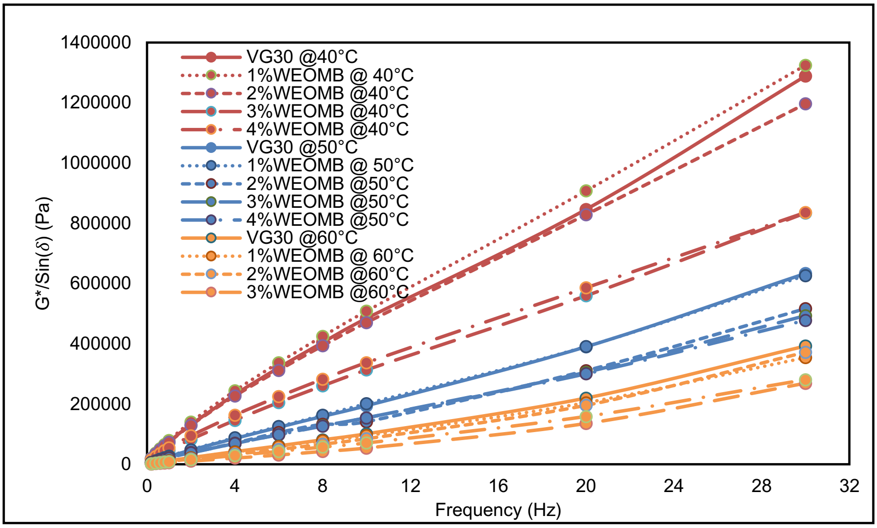

Fig. 2 Rutting parameter (G*/Sin(δ)) versus % WEOMB with varying temperature at different frequencies

4.3 Rutting and Fatigue Resistance of WEOMB from Frequency Sweep

The rutting parameter ($G^*/\sin\delta$) for WEOMB at a high temperature, 1% WEOMB enhanced more rut resistance competing with other percent of WEOMB. Fatigue parameter ($G^*\sin\delta$) for WEOMB at all temperatures 3% WEOMB is improved fatigue resistance. For WEOMB, due to an increase in temperature, rutting parameter $G^*/\mathrm{Sin}(\delta)$ is reduced to 70% at 50 °C and 80% at 60 °C with 40 °C as the reference temperature (Fig. 2). For WEOMB, due to an increase in temperature, the fatigue parameter $G^*\mathrm{Sin}(\delta)$ value is reduced from 65 to 50% at 50 °C and 90% to 70% at 60 °C with 40 °C as the reference temperature, depending upon the frequencies (Fig. 3).

4.4 Fatigue Life of WEOMB from Linear Amplitude Sweep (LAS)

The fatigue life of modified bitumen evaluated from the shear stress applied on the binder for various temperatures at constant shear strain rate figures are shown below for WEOMB; the lesser the shear stress at constant shear strain, the better the fatigue life.

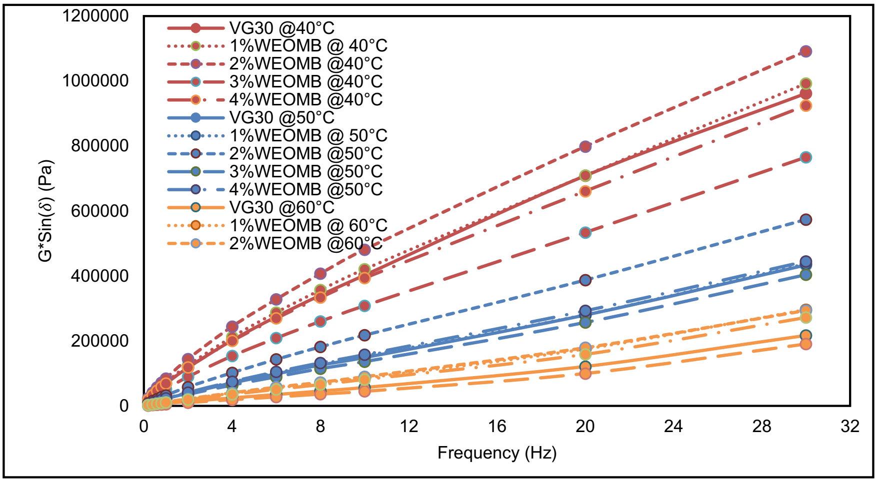

Fig. 3 Fatigue parameter (G*Sin(δ)) versus % WEOMB with varying temperature

Based on Fig. 4, the shear stress has a lower peak at constant shear strain rate which is a better sign for evaluating the positive effect of modified binders on fatigue life. It is assessed that increase in waste engine oil content the shear stress is decreased at a constant shear strain rate compared to the VG30 plain bitumen. Due to decrease in shear stress, a better fatigue life is observed at 3% of WEOMB.

4.5 Storage Stability of WEOMB

Separation test results show that WEOMB separated more at the top portion, due to less specific gravity of WEO. Table 2 lists the outcomes of the storage stability test. The variation in the softening point values of the down to top portions of the sample is exceeding 2.2 °C, the modified bitumen is not advisable to preserve and usage will have several issues [9]. From the results, it is observed that the separation tendency values are less than 2.2 °C as mentioned in Table 2. This shows a positive trend in storage stability of WEO modified bitumen since it works as a softener to the bituminous binder. This is also supported by the specific gravity of WEO i.e., Specific gravity of WEO is less than the specific gravity of water.

4.6 Fourier Transform Infrared Spectroscopy (FTIR)

FTIR spectrum indicates (Fig. 5) that WEO consists of hydrocarbon compounds, C–OH plane band and functional groups like C–O–C and metals. IR spectrum clearly

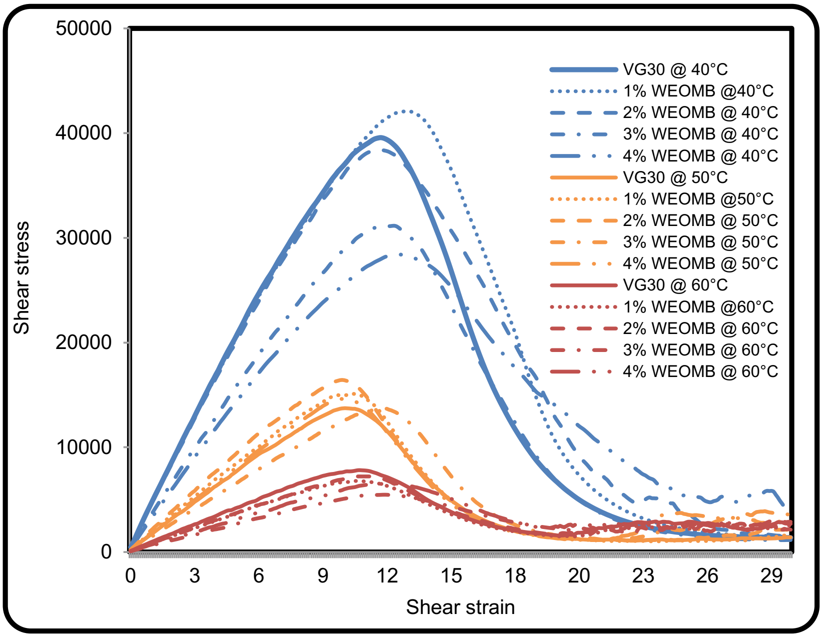

Fig. 4 Shear stress variation with temperature for WEOMB

Table 2 The storage stability results for WEOMB

WEO content (%)	Softening point on bottom (°C)	Softening point on top (°C)	Softening point differences (°C)
1	42.25	41.85	0.40
2	40.40	39.70	0.70
3	38.75	38.45	0.30
4	38.30	38.05	0.25

confirms the presence of functional groups in waste engine oils were C–H stretching of hydrocarbons in the region of 2842 and 2748 cm^{-1}. The absorbance of peak at 1104 cm^{-1} and 448 cm^{-1} signified the presence of functional groups like C–O–C and metals respectively [10].

FTIR analysis of waste engine oil is shown in Fig. 5, and the spectrum presence of functional groups based on peak wave no (cm^{-1}) and bonds are as follows. The common organic compounds present in the WEO were noticed at 2842 and 2748 cm^{-1} due to the sp3 C–H stretching bonds. A small peak obtained at 2311 cm^{-1} showed the occurrence of CO_2 in the WEO which may be due to the absorption of CO_2 from the atmosphere. The absorption peak of the hydroxylic C–O single bond noticed at 1064 cm-1 was due to the stretching of the C–O–C group. The weak peak noticed at 3543 cm^{-1} indicated the existence of Al–OH, while two C–H stretchings

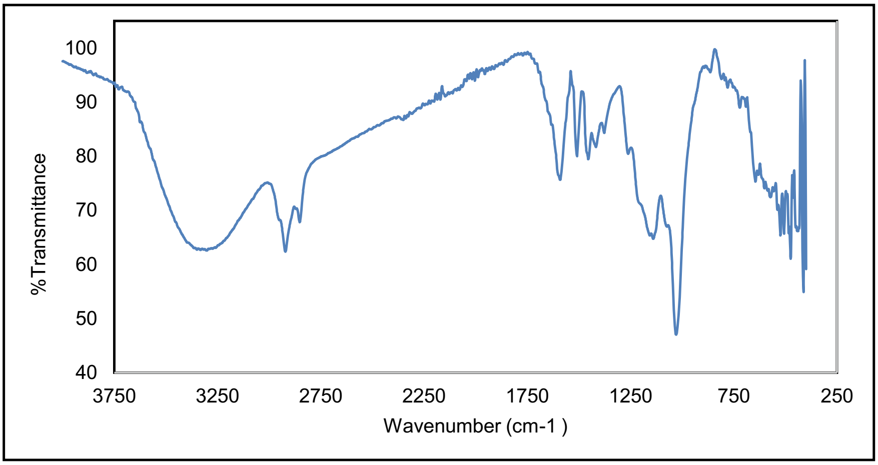

Fig. 5 FTIR results of WEO

at 2812 cm^{-1} and 2648 cm^{-1} result in the presence of hydrocarbon compounds. The existence of the C–OH plane band was confirmed by the peak at 1628 cm^{-1}. The absorbance peak at 1104 cm-1 and 448 cm^{-1} signified the presence of functional groups like C–O–C and metals respectively [10].

4.7 Scanning Electron Microscopy (SEM) Analysis

The SEM picture of Waste Engine Oil (WEO) acquired at 500 °C, shown in Fig. 6 turned into an uneven look and was dense. The surface of the material is appearing like an uneven mass, and is rough in character. However, the surface of the WEO acquired at 525 °C, seems like porous and cracked. Few cracks are clearly seen on the surface. At a temperature higher than 550 °C, the visibility of the cracks was laid off from the surface and the lumpiness increased. The debris have been looking like a dense solid. It may be predicted that the WEO formed at this stage was a porous solid hydrocarbon compound containing selected metals and the apertures are created with the aid of using the release of the volatile hydrocarbons.

The hydrocarbon compounds having better decomposition temperature were launched at a better temperature. The liberating of hydrocarbon vapours remained a few non-decomposable minerals and heavy metals along with a few high-temperature registrant hydrocarbons because the WEO was looking like a solid element. No such porous shape changed into note at the floor of the WEO at a higher temperature [10].

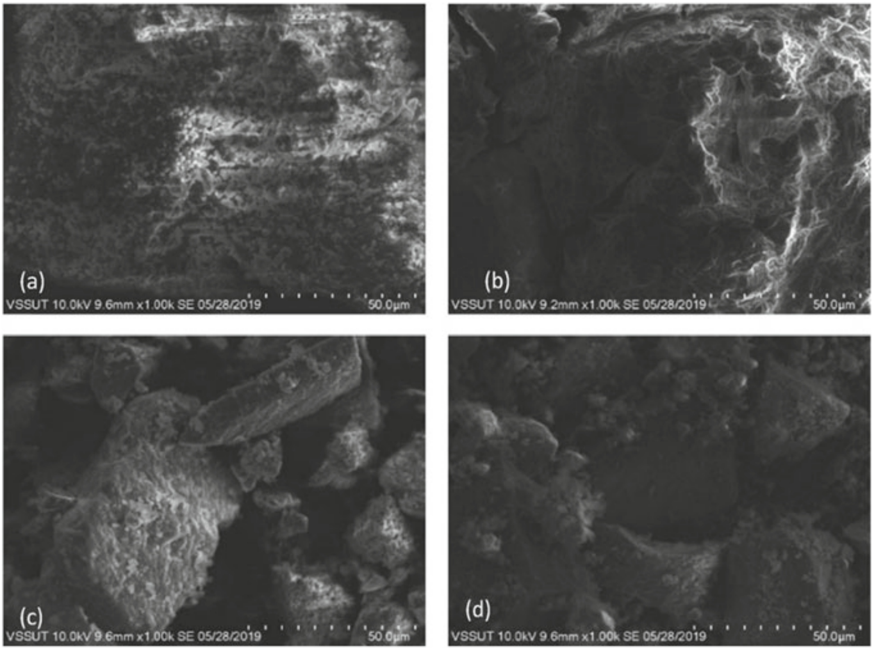

Fig. 6 SEM images of WEO [10]

5 Conclusions

In order to provide enhancement in the performance of the bituminous binder, a softening additive called waste engine oil is used in this study. In sequence to do this, a number of laboratory experiments were used to characterise the physical, chemical, rheological, storage stability, mixing and compaction temperatures. The following summarises the main conclusions drawn from the results of the numerous tests: The softening point of the bituminous binder decreases with an increase in WEO content. Thus, it increases the penetration value and reduces the viscosity of the binder since the WEO function acts as a softener to the bituminous binder. The addition of WEO to the bituminous binder results in less amount separation is within the limit. Hence, storage stability for WEOMB is within limits. The viscosity of WEOMB is reduced due to softening of the binder with the addition of WEO. However, it affects the mixing and compaction temperatures. The MSCR results show that the %R is less and J_{nr} value is more at an optimum WEO dosage of 3.0% by weight of the bituminous binder. So, it results in low rutting resistance. The SEM evaluated the surface texture of WEO; the presence of extremely small metal nanoparticles had an effect on the performance improvement. Optimum content of 3.0% WEOMB is improved fatigue life at a higher temperature due to its softening function. This paper presents only the performance of characteristics of the bituminous binder with WEO additive. This study can be extended to the performance assessment of the bituminous mixes such as rutting and fatigue performances that can be determined at intermediate and low temperatures.

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Rheological Investigation of Nano Silica-SBS Modified Soft Grade Bitumen

Kulsuma Salam, Mohammad Shafi Mir, and Bijayananda Mohanty

Abstract The purpose of this study aims at investigating the effect of using nano silica as a modifier for SBS modified asphalt binder. In this study, the concentration of SBS (3%) was kept constant (taken from previous published research) and the Nano-silica concentration was varied from 1% to 3% (decided after literature survey). The effect of varying concentrations (1, 1.5, 2, 2.5 and 3%) of the nano silica (by weight of binder) on the SBS modified binder was evaluated by utilizing various physical tests like penetration, softening point, and ductility. The Rotational viscosity (RV) and Dynamic Shear Rheometer (DSR), Tests were used to analyze rheological properties of the base binder and nano silica polymer modified asphalt binder. In addition, the performance of modified asphalt after Thin Film Oven (TFO) (short-term aging) and Pressure aging Vessel (PAV) test (long term aging) were assessed as well. Furthermore, the storage stability of the modified asphalt binder was evaluated. Results showed that the addition of nano silica has a positive effect on the rutting performance of SBS modified asphalt binders. Storage stability of the SBS modified asphalt binders improved significantly after the addition of nano silica. Using softening point and rheological parameters (complex modulus (G*) and phase angle (δ)), the best values were possessed by 1.5% nano silica/3% SBS modified binder. During rheological characterization, it was found that as the complex modulus increases, phase angle decreases, Superpave Rutting parameter increases and failure temperature increases with increasing nano silica content. It was also found that Brookfield viscosity increases with an increasing nano silica concentration as the binder becomes stiffer. All the test results confirmed the fact that the nano silica-SBS modifier is effective in enhancing the high temperature properties (rutting resistance) of the soft grade binders and at the same time, it increases the elasticity of the binders.

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Keywords Styrene butadiene styrene · Nano silica · Superpave rutting parameter · Superpave fatigue parameter · Viscosity · Aging · Storage stability

1 Introduction

For the construction of roadways, bituminous asphalt materials made of mineral aggregates are frequently used. In order for a country to develop economically, its roads must be well paved. It results in significant investment in their manufacturing [1, 8]. Nearly, 95% of bitumen produced now in the globe is used in the asphalt pavement sector [30], which also corresponds to the total length of road built with asphalt pavement. As the traffic volume and climate variations rise, it becomes more difficult to maintain pavement surface condition, which causes numerous severe distresses in road surfaces and necessitates bitumen modification. Asphalt binder stiffness can be increased by polymers at high temperatures, and asphalt binder ductility can be enhanced at low temperatures. On account of this, they lessen rutting at high temperatures and fatigue cracking at moderate and low temperatures. However, when the polymer is stored at high temperatures, it tends to separate from the asphalt binder because of its poor compatibility with the asphalt binder. Additionally, polymer characteristics degrade after prolonged exposure to heat, oxygen and ultraviolet rays [28]. Phase separation may happen when the SBS and asphalt binder are blended because the molecular weights of the polymeric chains are higher or equal than those of asphaltenes [5], (Goolayak et al. 2010). As a result, numerous studies have demonstrated that polymers cannot be used in isolation to relieve all pavement distress, including rutting, fatigue, moisture-induced damage and aging [27]. Therefore, there is still a need for novel, superior bitumen (asphalt binder) modification techniques, particularly those that enhance bitumen-aggregate adhesion, increase aggregate resistance to aging, ensure recyclability, and reduce modification costs. The advancement of technology has made it possible to produce more nanomaterials in recent years. Nanomaterials are made up of particles smaller than 100 nm and have a pure, straightforward chemical composition. Because of surface area to volume ratio, shape, and chemical composition of particles, nanomaterials exhibit different physical properties than traditional materials [40]. One of the nanomaterial, nano silica, has a positive effect on the rheological and physical characteristics of the asphalt binder when added to it. Previous studies have demonstrated that the inclusion of the nano silica improved asphalt binders' resilience to rutting. The asphalt binder's elasticity, ageing resistance, and storage modulus are all enhanced by the addition of nano silica [13]. It stiffens the asphalt binder and enhances temperature susceptibility as evidenced by decrease in penetration and increase in the softening point [22]. Asphalt binders and mixtures' ability to self-heal is improved by the nano silica, making pavements more resilient and long-lasting [21].

Researchers have recently used a variety of polymer nanocomposites to study the mechanical, rheological, and physical characteristics of the asphalt binder as well as the performance of the asphalt mixture. Ameri et al. (2016) studied the effect of nano clay on the resistance to rutting and moisture damage of unmodified and SBS-modified asphalt binder and mixture. In comparison to neat 4% SBS and 4% SBS modified mixes, 2% nano clay and 6% nano clay were used in the 4% SBS blend. The SBS modified asphalt binder's thermal stability is improved by the nano clay. The viscosity and OBC of the neat asphalt binder are increased by nano clay, as those of SBS-modified asphalt binder. Regarding the asphalt mixtures utilized, the improvements in rutting resistance provided by nano clay and SBS-modified are equivalent. The addition of nano clay to the SBS-modified asphalt binder also boosts rutting resistance. The impact of nano silica's high temperature performance on the SBS modified binder was explored by Rezau et al. and Yousoff et al. (2018). Results revealed that adding nano silica to asphalt binders with SBS modifications can considerably increase their ability to rut. The performance at high temperatures was improved in mixes made with SBS/nano silica composite, as evidenced by their higher flow number. A notable decrease in PMB's oxidative aging is seen as a result of the addition of the nano silica, which enhances the various performance attributes of PMB. After incorporating nano silica into PMB, a significant improvement in moisture susceptibility is seen. At intermediate and high temperatures, the dynamic elastomer-modified asphalt binders were studied. SBS and SEBS polymers at three different concentrations (3, 4, and 5%) were used. SBS was found to work best at a concentration of 3%. According to the results, polymer modification can effectively increase rutting resistance at high temperatures and fatigue resistance at intermediate temperatures [31].

Al-Hamali et al. [9] assessed the durability characteristics of the modified asphalt binder with polymer blended with the nano-silica. As a result, the impact of nano-silica on storage stability as well as physical and rheological parameters were assessed. The addition of nano-silica, when compared to the asphalt binder with the polymer, enhances the complex modulus at low frequencies or high temperatures, according to the findings of DSR testing, which results in higher resistance to rutting.

This study's objective is to determine the impact of adding nano silica to the SBS-modified asphalt binder. Physical tests like penetration, softening point, and ductility were used to assess the impact of different concentrations (1, 1.5, 2, 2.5 and 3%) of the nano silica (by weight of binder) on the SBS modified binder. The rheological properties of the base binder and nano silica polymer modified asphalt binder were examined using Rotational viscosity (RV) and Dynamic Shear Rheometer (DSR) tests.

2 Objectives of the Study

The objectives are enlisted below:

1. To evaluate mixing time for the preparation of Nano silica and SBS.
2. To evaluate the optimum Nano silica content for the SBS modified binder.

3. To investigate the influence of SBS-Nano silica nanocomposite on viscosity of the asphalt binder.
4. To study the effect of Nano silica-SBS content on rheological behavior of the asphalt binder based on rutting and fatigue.
5. To evaluate the effect of Nano silica-SBS on aging and high temperature storage stability of the asphalt binder.

3 Experimental Procedure

3.1 Material Characterization

The base binder for this investigation is 80/100 penetration grade bitumen (VG-10), which was bought from a nearby distributor. Table 1 displays the mentioned base binder's numerous characteristics. For the alteration of the asphalt binder, nano silica and Styrene Butadiene Styrene (SBS) were taken into consideration. Platonic Nanotech Private Limited provided the nano-silica, and Dycon Chemicals sold the SBS. Tables 2 and 3 provide the fundamental characteristics of SBS and Nano-silica, respectively (supplied by the supplier). Table 4 (supplied by the supplier) lists the elements that make up the nano silica. Nano-silica and SBS SEM images are depicted in Figs. 2 and 3, respectively. The optimal SBS concentration used in this investigation was determined to be 3% [31], while the concentration of nano-silica was changed from 1 to 3% [10, 17]. Additionally, the temperature of the mixture was maintained at 163 °C [9, 20] (Fig. 1).

3.2 Sample Preparation

Polymer and nano-silica were added to the asphalt binder using a high shear mixer. To start, a uniform liquid was created by heating the asphalt binder on its own. Next, the SBS polymer was added to the asphalt binder at 3% [31] by weight of the asphalt binder. Finally, the nano-silica was slowly added to the mixture at different concentrations (0, 1, 1.5, 2, 2.5, and 3%) and mixed with the asphalt binder at 3000 rpm at a temperature of 163 °C [9]. Two hours was determined to be the ideal mixing time using the softening point approach. By conditioning the base and modified asphalt binder in a thin film oven (TFO) in accordance with ASTM:D 1754 for 5 h at 163 °C, the materials underwent short-term ageing. According to ASTM:D 6521, modified asphalt binder underwent long-term ageing for 20 h at 100 °C and 2.1 MPa in a pressure ageing vessel (PAV). Two steps make up the entire experimental strategy (Fig. 1).

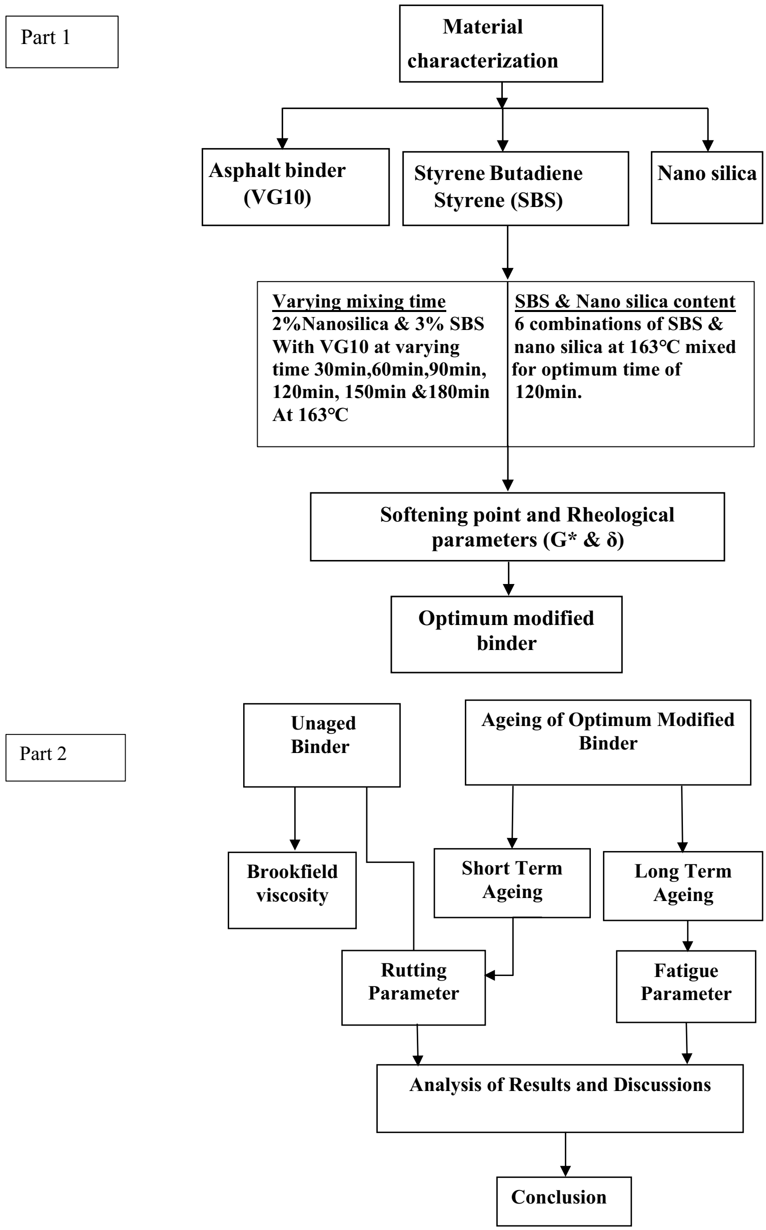

Fig. 1 Experimental flow chart

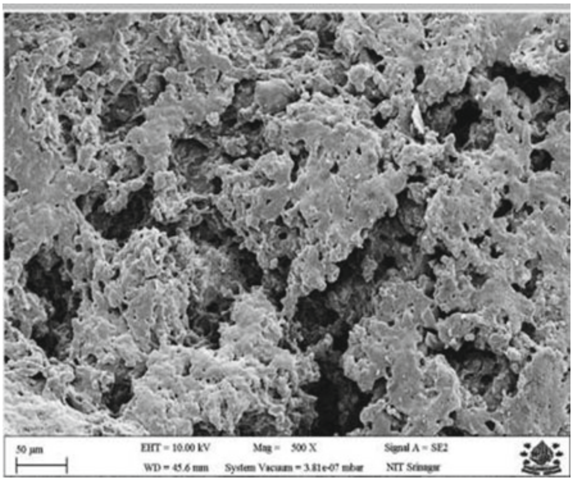

Fig. 2 SEM image of SBS

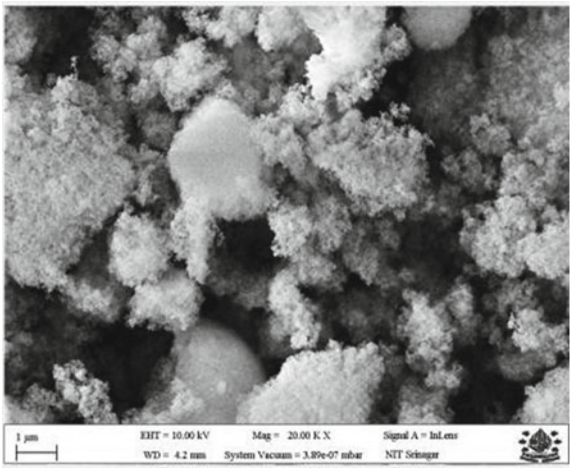

Fig. 3 SEM image of nano silica

Table 1 Results of tests conducted on VG10 Bitumen

Test	Standard code	Value	Specification limit (minimum)
Softening point (°C)	IS:1205	46	40
Ductility value (cm)	IS:1208	100+	75
Penetration value 0.1 mm at 25 (°C)	IS:1203	88	80
Dynamic viscosity at 60 °C	IS:1206 (Part II)	1064	800
Kinematic viscosity at 135 °C	IS:1206 (PartIII)	278	250

Table 2 Physical properties of SBS

Specification	Values
Form	Crystal
Melting point	160–200 °C
Specific gravity	0.94
Tensile strength	20 MPa
Viscosity	1.2 Pa.s

Table 3 Physical properties of nano silica [17]

Specification	Value
Purity	99.5%
Average size of the particle	30–50 nm
Specific surface area	200–250 m^2/g
Bulk density	0.10 g/cm^3
True density	2.5 g/cm^3
Morphology	Porous

Table 4 Elemental composition of nano silica particles [17]

SiO_2	99.5%
Al	0.02%
Fe	0.05%
Mg	0.1%
Ca	0.08%

4 Test Methods

4.1 *Scanning Electronic Microscope*

The SEM analysis was conducted to investigate the morphology of the nano silica powder and SBS.

4.2 *Physical Characterization of the Binders*

The basic properties of base binder and nano silica-SBS modified binder were evaluated, using penetration (IS: 1203), softening point (IS: 1205), viscosity (AASHTO T 316) and ductility (IS: 1208).

4.3 Viscosity Test

In the Brookfield apparatus at AASTHO T316-13, rotational viscosity is employed to calculate viscosity at mixing and compaction temperatures. The amount of torque needed to rotate the vertical shaft was measured to ascertain the viscosity of the binder. In a chamber kept at a temperature of 135 °C with the spindle revolving at a speed of 20 rpm, 8–11 g of base and modified binders were evaluated [35]. Because 135 °C is thought to be the laying temperature, viscosity is computed at that temperature.

4.4 Rheological Characterization of the Binders

Using a dynamic shear rheometer (DSR), an Anton Paar MCR 102, the linear viscoelastic characteristics of aged and unaged bitumen were evaluated over a range of frequencies and temperatures in accordance with D 7175-15. Complex modulus, phase angle, rutting parameter, fatigue parameter, TSHRP, and ageing index were the parameters collected. While maintaining an angular frequency of 10 rad/s, the temperature is adjusted between 46, 52, 58, 64, 70, 76, 82, and 88 °C [17]. For fatigue characterization, the samples needed for testing were created in a silicon mould that was 2 mm thick, 8 mm in diameter, and had a gap of 2 mm between parallel plates. For rutting, however, a mould that was 1 mm thick, 25 mm in diameter, and had a gap of 1 mm was utilized. According to SHRP recommendations, aged samples are tested at a strain rate of 10%, whereas unaged samples are tested at a strain rate of 12%. For unaged binders and aged binders, the value of G*/sin is restricted to 1 kPa and 2.2 kPa, respectively.

Rutting resistance using Superpave Rutting Parameter

It is given by G*/Sin δ. It is employed to evaluate the asphalt binder's resistance to rutting. Rutting resistance will be stronger for an asphalt binder with a higher G*/Sin δ ratio because less energy will be lost during each cycle of loading.

Fatigue resistance using Superpave Fatigue Parameter

It is given by G*.Sinδ. It indicates that there will be less stress accumulation if there is less energy expended per cycle. As a result, materials with lower G*.Sin δ have superior fatigue resistance. The highest value of G*.Sin δ is 5000 kPa, according to SHRP and 25 °C test temperature was used.

4.5 Aging Process

According to ASTM:D 1754, the thin film oven was used to age the base and modified binders over a short period of time. A moving film of the asphalt binder was heated in this oven for 5 h at 163 °C. This technique's primary objective is to gauge the impact of heat and air on a moving film of semi-solid asphaltic materials. Specific properties are examined before and after this test to ascertain the impact of ageing on the binders. "Short-term ageing determines the resistance to hardening under the impact of air and heat. This tool simulates the conditions that exist during the mixing and placement of asphalt mixes" [16]. Pressure ageing vessels (PAVs) were used to age base and modified binders over an extended period of time in line with ASTM:D 6521. 20 h were spent doing long-term ageing at 100 °C and 2.1 MPa air pressure. It simulates the field situation for a pavement's first 5–7 years of use.

4.6 Effect of Nano Silica-SBS on Aging Resistance

When we talk about ageing, we mean the asphalt binders oxidatively ageing. During the mixing and laying process as well as throughout their service life, binders oxidise. Some of the binder's lighter and more flammable components evaporate during this ageing process, making the material brittle and increasing the risk of pavement damage. Consequently, binders with a reduced level of oxidation are desired. Using two parameters—the ageing index and the incremental softening point—the current study compares the ageing resistance of unmodified and changed binders.

Softening Point Incremental (SPI):

It is the difference between the softening point of the aged binders and unaged binders. Lower value of softening point incremental is required for binders having good ageing resistance.

$$\mathrm{SPI} = \mathrm{S}_{\mathrm{Paged}} - \mathrm{S}_{\mathrm{Punaged}} \tag{1}$$

where SPI is the softening point increment, SPaged is the softening point of the short term aged binder and SPunaged is the softening point of the unaged binder [19].

Ageing Index:

The ageing resistance is calculated using the superpave rutting parameter, which also determines the ageing index. As the usefulness of AI rises, so does the vulnerability to ageing (Ashish et al. 2016). At a frequency of 10 rad/sec, 60 °C is maintained as the temperature.

$$\mathrm{AI} = |(\mathrm{G}*/\mathrm{Sin}\delta)\mathrm{aged}/(\mathrm{G}*/\mathrm{Sin}\delta)\mathrm{unaged}| \tag{2}$$

where ($G^*/\mathrm{Sin}\delta$) aged and ($G^*/\mathrm{Sin}\delta$) unaged are the rutting factor parameter of the short-term aged and unaged asphalt binder respectively [16].

4.7 Storage Stability Test

The stability of the modified asphalt binder is examined using a storage stability test because at higher temperatures, the modifier easily separates from the asphalt binder. The uniformity of the mixing of the modifier with neat bitumen is tested. The difference in the polymer and asphalt densities is what causes the separation. Storage stability test is used to gauge the storage stability of modified binders. A hot sample is poured into an aluminium tube. According to ASTM D 7173-14, the tube is sealed and placed vertically in an oven that is kept at a temperature of 163 °C for 48 h [11]. The aluminium tubes are removed from the oven after 48 h and stored in a freezer for 4 h at a temperature of −10 °C. After that, the tube is divided into three equal lengths, the middle section is removed, and the top and bottom sections of the asphalt are collected and subjected to a softening point test. It is discovered that the top and bottom sections have different softening points. If the softening point difference is less than 2.5 °C, stability is deemed adequate [37].

5 Results and Discussion

5.1 Determination of Optimum Mixing Time

The softening point was used to determine the optimum mixing time. The softening point of the modified 2% NS-3% SBS binders was discovered at various mixing times, including 30, 60, 90, 120, 150, and 180 min. Figure 4 illustrates how the softening point changes as the mixing time changes. As can be seen, the softening point rises till 120 min as the mixing time increases. But as mixing time is extended from 120 to 180 min, it gets smaller. This is because prolonged mixing degrades the qualities of the binders, which may be linked to changes in the binders' physical and chemical properties. At 120 min, or 66 °C, the greatest softening point improvement was discovered. So, based on the softening point, we can say that 120 min is the ideal mixing duration.

5.2 Determination of Optimum Mixing Concentration

Figures 5, 6, and 7 display the outcomes of the softening point test, complex modulus (G*), and phase angle (δ). As demonstrated in Fig. 5, every changed binder has a

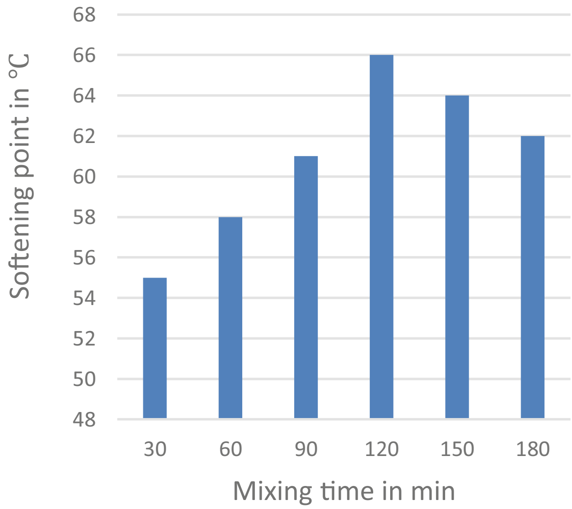

Fig. 4 Variation of softening point with mixing time for 2% NS/3% SBS modified binder

higher softening point than the base binder. They are therefore stronger and less prone to long-term deformation [39]. As shown in Fig. 5, the nano-silica softening point increases up to 1.5% at constant SBS concentrations while decreasing at lower concentrations. The distribution of asphalt changes as a result of the hydrogenated butadiene groups of SBS absorbing soft asphalt ingredients (saturates and aromatics), which may be attributable to the structural properties of the SBS molecular chain. Additionally, the styrene group plays a crucial role in raising the softening point of the binders due to its higher glass transition temperature. The softening point of 1.5% NS/3% SBS modified binders is 69.5 °C, which is the highest [15]. Therefore, 1.5% NS/3% SBS might be thought of as the ideal concentration.

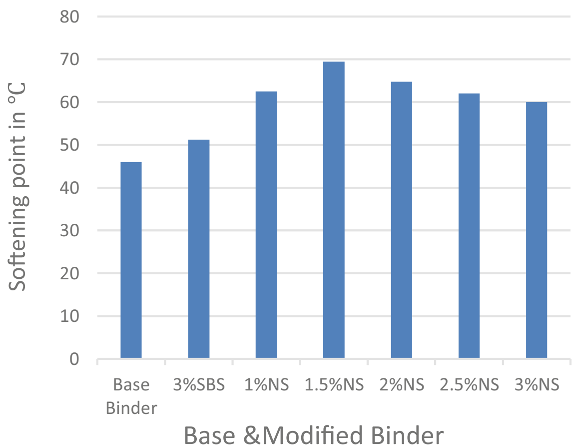

Fig. 5 Softening points of base and modified binder for 120 min

In the DSR test, complex modulus and phase angle at various concentrations of NS and 3% SBS were discovered for aged and unaged samples at test temperature of 60 °C, respectively, as shown in Figs. 6 and 7. Figures 6 and 7 show that the G* value increases as the concentration of NS in the polymer modified binder increases, however it drops after 1.5% NS concentration. Thus, it demonstrates that the bituminous binder becomes more stiff as the NS content in PMB increases. Additionally, it is clear that as the binders age, they stiffen and exhibit higher values for complex modulus and lower values for δ.

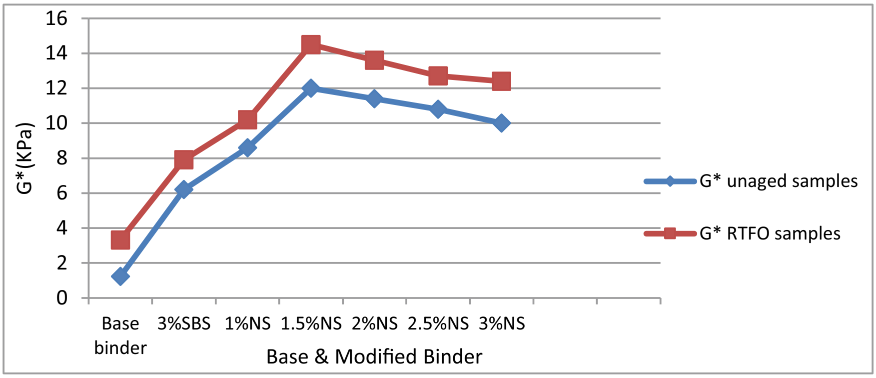

Fig. 6 Complex modulus of unaged and TFO samples at test temperature of 60 °C

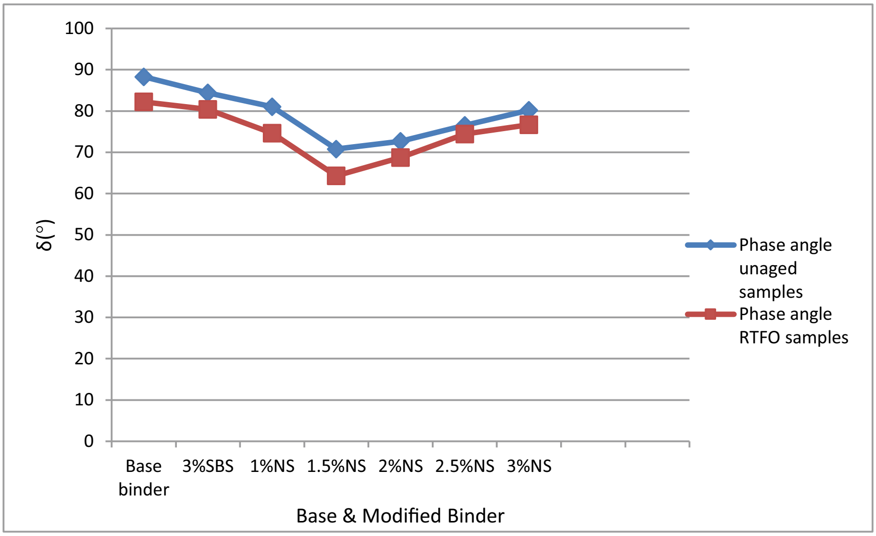

Fig. 7 Phase angle of unaged and TFO samples at test temperature of 60 °C

5.3 *Determination of Brookfield Viscosity*

The purpose of this test, which was carried out in line with AASHTO T 316-13, is to compare the viscosity of modified and unmodified asphalt binder at 135 °C. At the 135 °C test temperature, the inclusion of nano silica increases the base binder's viscosity. The modified asphalt binder sample, however, demonstrated a significant difference as compared to the base binder, as seen in Fig. 8. The hardening impact of nanoparticles is the cause of the rise in viscosity. The improved dispersion of the additional nanomaterial layers in the base binder may be the cause of the modified asphalt binders' higher viscosity, which increases bonding strength by limiting asphalt flow (Hossian et al. 2014). This boosts the asphalt's physical qualities and makes it tougher, which is a good sign that the asphalt binder's properties have improved. Figure 8 illustrates how the viscosity of asphalt increases as the concentration of nano silica increases. Binders treated with 1.5% nano silica and 3% SBS had the highest viscosity. At 135 °C, modified asphalt has a viscosity that is significantly greater than the base binder, this difference may be the influence of polymers from the composition analysis of the modifier. The improvements in the high temperature properties of asphalt binders are shown in rotational viscosity measurements. Binders with a high viscosity create thicker films surrounding the aggregates. Because of this, cohesive forces between components grow and there is an increase in resistance to water and environmental degradation [26].

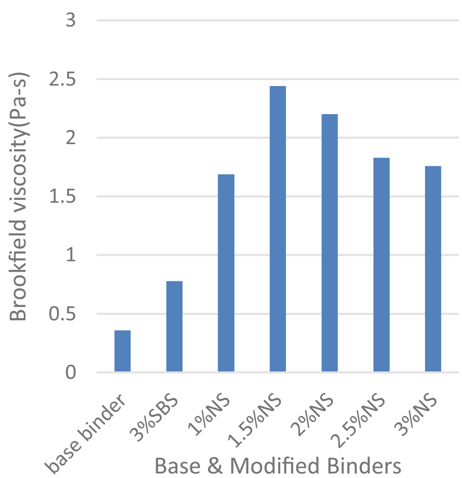

Fig. 8 Brookfield of base and modified binder

5.4 Physical Properties of Optimum Modified Binder

The physical characteristics of 1.5% NS-3%SBS modified binder are given in Table 5 and Fig. 9

Complex Shear Modulus (G*)

A binder's overall resistance to deformation during repeated shearing is measured by its complex shear modulus. At higher G* values, the binder becomes stiffer. G* showed an exponentially declining trend with rising test temperature when G* was plotted against temperature for all modified and unmodified samples at all aging levels (Fig. 10). With the addition of the nano silica (NS) modifier, the asphalt binder's G* value increased, indicating that the modified asphalt binder is more rigid than the unmodified one. In contrast to unmodified binders, NS-modified binders are thus somewhat stiffer, more resilient, and more deformation-resistant. The outcomes of [2, 10] for NS are in agreement with this result. A higher complex modulus value for the nanocomposite indicates that it has undergone development.

Table 5 Physical characteristics of 1.5% Nano-silica /3% SBS modified binder

Test	Standard code	Value
Softening point (°C)	IS:1205	69.5
Ductility value (cm)	IS:1208	68
Penetration value (0.1 mm)	IS:1203	76
Brookfield viscosity (centipoise)	AASTHO T316-13	2.44

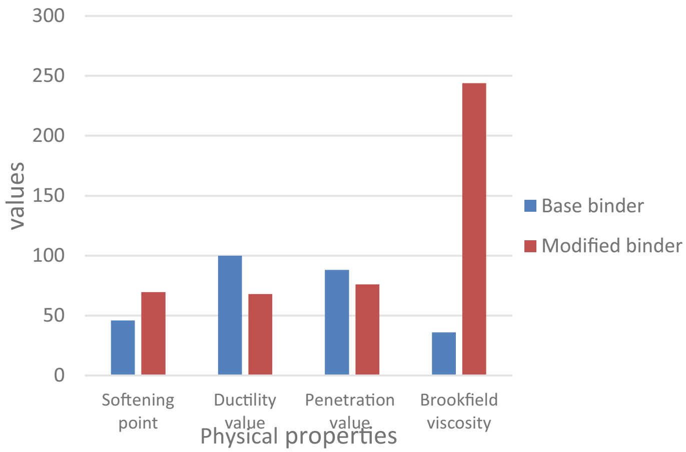

Fig. 9 Physical properties of optimum modified binder

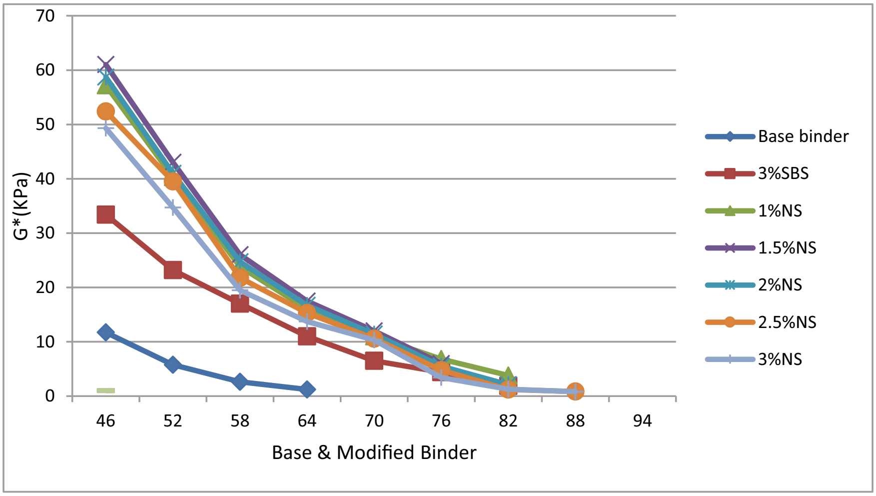

Fig. 10 Variation of G* with temperature for base and modified binder

Phase Angle (δ)

Figure 11 depicts how the phase angle changes with temperature for base and modified binders. A decrease in the elastic portion and an increase in the viscous portion of the asphalt binder resulted in a tendency for the behavior of the binder to be more viscous for all binder samples as test temperature increased. "Measurement of phase angle is generally considered to be more sensitive to chemical and physical structure than complex modulus for modification of asphalt" [41]. Figure 11 makes it evident that adding nano silica to PMB reduces the phase angle and improves the elastic behavior of the asphalt. Base binder begins to become viscous at higher temperatures when the phase angle approaches 90°, and makes a transition to Newtonian fluid [14]. The nature of polymer network depends upon chemical and physical properties of the SBS polymer and base asphalt. 1.5% Nano-silica/3% SBS modified binder possess the lowest values of the phase angle.

Superpave Rutting Parameter (G*/sin δ)

The rutting parameter (G*/sin δ) versus temperature was plotted for the control and modified asphalt binders at different percentages of nano-silica (Fig. 12). An exponential decrease in the rutting parameter was noticed with an increase in testing temperature for all tested asphalt samples.

Failure Temperature TSHRP

According to Superpave binder grade criteria, the temperature of asphalt failure is determined by the point at which the factor of G*/sin(δ) drops below 1.0 kPa. The performance grade of asphalt binders is determined by the failure temperatures of

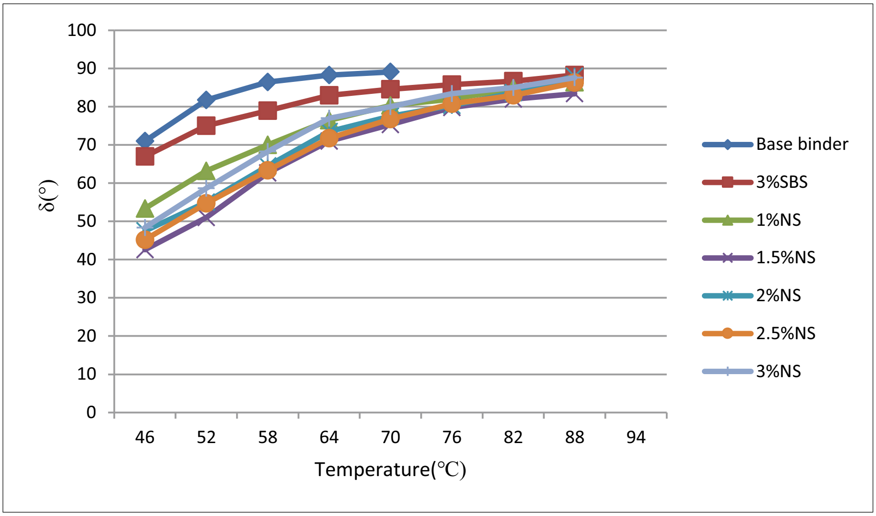

Fig. 11 Variation of phase angle with for base and modified binder

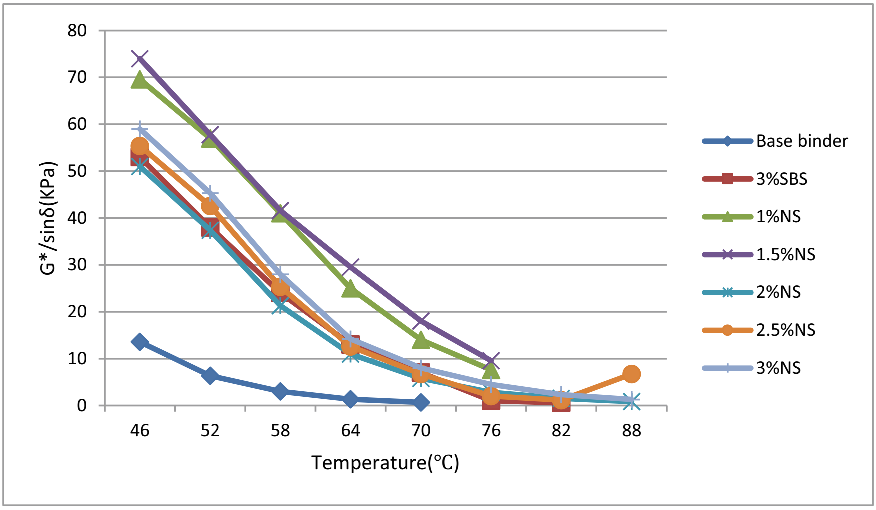

Fig. 12 Variation of superpave rutting parameters with temperature for base and modified binders

unmodified and modified binders [18]. In addition, the base binder's failure temperature of 70 °C is the lowest. The TSHRP values are displayed in Fig. 13. Asphalt binder modified with nano silica had improved high temperature rheological properties. Asphalt binder had the maximum failure temperature at 1.5% NS in 3% SBS PMB.

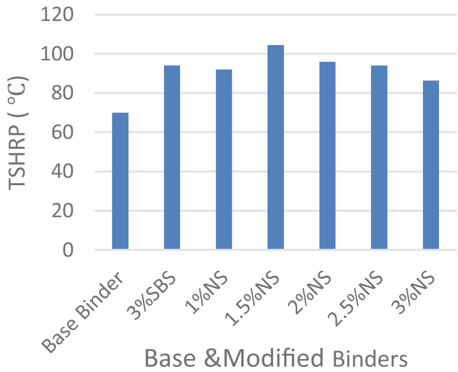

Fig. 13 TSHRP values for modified binders

Superpave Fatigue Parameter (G*.Sinδ)

For both unmodified and modified asphalt binder, Fig. 14 shows the fluctuation of the fatigue parameter (G*.sin δ) with various nano-silica concentrations in the polymer modified binder at a temperature of 25 °C. The graph demonstrated that the G*.sinδ value falls when 1.5% NS/3% SBS are added to the base binder. The fact that this value has decrease shows that the resistance to fatigue failure has improved. Furthermore, by incorporating additional nano silica percentages, an increase in G*.sinδ was seen, demonstrating the detrimental effect on fatigue failure. As a result, the addition of 1.5% NS/3% SBS can improve the fatigue resistance performance at intermediate temperatures.

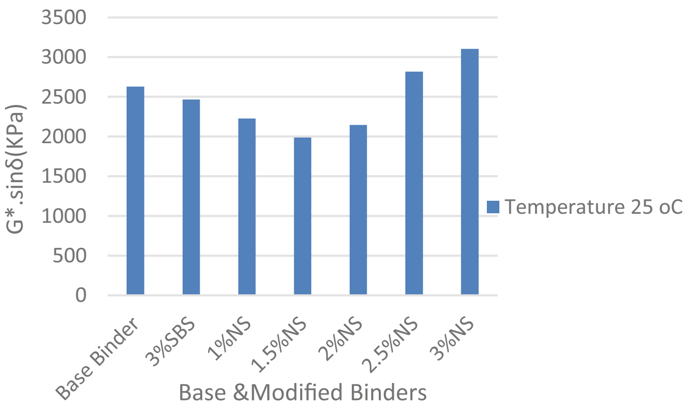

Fig. 14 G*.sinδ values for modified binders

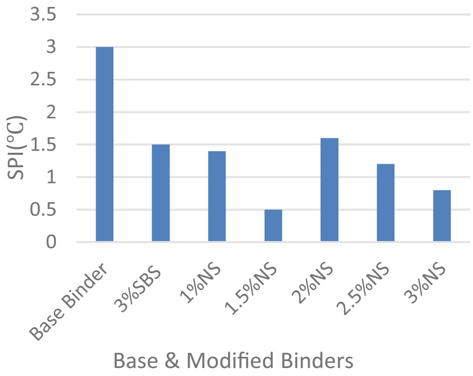

Fig. 15 SPI values for base and modified binders

5.5 *Determination of Aging Effect*

The effect of modifiers on the ageing resistance of the binders has been studied using two parameters.

Softening Point Incremental

Figure 15 compares the softening points of aged and unaged binders. As binders age, they get tougher, hence the softening point of aged binders is higher than that of unaged binders. The resistance to ageing rises as the values of SPI fall with rising nano-silica concentration. 1.5% Nano-silica/3% SBS modified binder has the lowest SPI value.

Aging Index

SBS and nano-silica addition lower the AI value. As a result, the addition of nano-silica slows down the oxidation process in the asphalt binder, which increases the modified asphalt binder's ability to resist ageing. By using 1.5% Nano-silica and 3% SBS modified binder, the lowest value of AI is displayed (Fig. 16).

5.6 *Storage Stability*

The stability of modified asphalt binder is examined using a storage stability test because at higher temperatures, the modifier easily separates from the asphalt binder. The uniformity of the mixing of modifier with neat bitumen is tested. The difference in polymer and asphalt densities is what causes the separation. It is discovered that the top and bottom sections have different softening points. If the variation in the softening point is less than 2.5 °C [37], stability is considered to be satisfactory. Table 6 presents the outcomes.

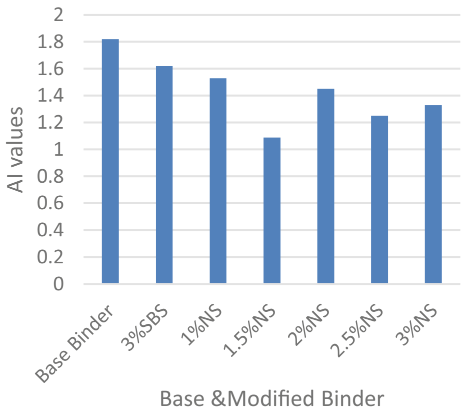

Fig. 16 AI values for base and modified binders

Table 6 Storage stability results

Binder	Difference in softening value
Base binder	0.1
3% SBS	8.1
1% NS/3% SBS	1.8
1.5% NS/3% SBS	1.3
2% NS/3% SBS	1.1
2.5% NS/3% SBS	1.2
3% NS/3% SBS	0.8

6 Conclusion

This research work was carried out to develop Nano silica-SBS modified asphalt binder. This research work evaluates the effect of NS-SBS on the Superpave rutting, fatigue and storage stability behavior of the modified binder. The following conclusions can be drawn from results and discussions;

1. The test result showed that the modification of the VG-10 asphalt binder with 1.5% Nanosilica and 3% Styrene Butadiene Styrene blended at 163 °C for 120 min results in a homogeneous modified asphalt binder.
2. The softening point increased by 51%, penetration and ductility by 33.6% and 32% respectively as compared to neat asphalt. This demonstrates the decreased temperature susceptibility and increased stiffness of NS-SBS modified bitumen.
3. High temperature rutting resistance improved as reflected by an increase in the value of complex modulus and Superpave rutting parameter. Complex modulus increased by 11.72 kPa to 60.8 kPa and Superpave rutting parameter increased from 23.1 kPa to 74.13 kPa at 46 °C (of neat and optimum modified asphalt binder respectively). It was also observed that as temperature increased from 46 °C to 100 °C, G* and G*/sinδ decreased.

4. The failure temperature of pure bitumen was observed as 70 °C and that of the optimum modified binder was 100 °C. Thus, NS-SBS addition improve the high temperature failure in the unaged binder state.
5. NS-SBS improves the elasticity of binders as the value of phase angle decreases. The value of phase angle for virgin bitumen was 71° and that of the optimum modified binder was 42.6° indicating a decrease of 40% and thus, showing an increase in elasticity.
6. Nano silica plays an important role in increasing the aging resistance of the SBS polymer modified binder which can be derived from the fact that the softening point incremental and aging index decreases. The SPI of neat asphalt was obtained as 3 °C and minimum value was obtained for the optimum modified binder (0.5 °C). Also, aging index of neat asphalt was 1.82 and minimum value was obtained for the optimum modified binder i.e., 1.09.
7. The viscosity of the NS-SBS modified binder increases as the NS content increases and the maximum value was shown by 1.5% NS, added to 3% SBS which is 2.44 Pa-s.
8. The results of Superpave fatigue parameter showed that there is an improvement in the fatigue resistance as G*.sinδ value decreases. Hence, the intermediate temperature performance is enhanced and maximum improvement was shown by 1.5% NS-3% SBS for which the value is 1989 kPa.

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Performance Evaluation of Conventional and Field-Produced High Modulus Asphalt Binders and Mixtures

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Abstract High modulus asphalt (HiMA) is a bituminous mixture made up of modified binders. It is one of the alternatives to increase the load-bearing capability of pavement structure against structural distresses like fatigue and rutting crack. In the present study, a conventional binder (VG-40) and HiMA binder, collected from the field were used for the binder testing. Various tests like SARA, FTIR, and LAS (Linear amplitude sweep) were conducted for both conventional and modified binders. Mechanical characteristics of asphalt mixtures were determined by conducting tests like stability and flow, resilient modulus, and indirect tensile strength. Performance characteristics of the asphalt mixtures were determined in terms of moisture damage by conducting a resilient modulus test before and after 'MIST' (moisture-induced sensitivity test). The asphalt mixture fatigue functioning was evaluated using 'TOT' (Texas overlay tester). Results show that the HiMA binder gives a low penetration value and high softening point compared to conventional VG-40. LAS test indicates that HiMA does not damage in the test condition and transferred the applied load continuously compared to VG-40. The results show that the HiMA mixture is stiffer than the conventional mix as the resilient modulus and moisture resistance (MR Ratio) of the HiMA mixture is higher than the conventional mix. TOT results show that HiMA requires a higher number of cycles to cause fatigue failure compared to the conventional mixture.

Keywords High modulus asphalt · SARA · FTIR · LAS · Moisture-induced sensitivity · Texas overlay tester

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1 Introduction

Indian Road Congress (IRC) recommended that the bituminous layer should be resistant to both rutting and fatigue cracking (IRC:37-2018). Thus, IRC recommends adopting a conventional DBM mix prepared with VG-40 as it is the highest quality virgin binder available in India. With the increase in modulus values of bituminous layers, the thickness of the pavement can be reduced which ultimately saves the cost of road construction and reduces material consumption. This can be achieved by using innovative technology that uses binders that are stiffer than VG-40.

High modulus asphalt (HiMA) technology is widely adopted in different parts of the world. HiMA or Enrobé à Module Elevé (EME) developed in France was adopted for the base course material in which hard paving grade bitumen is used at a high binder content with the dynamic modulus of 14,000 MPa at 15 °C and 10 Hz. HiMA mixes are suitable for pavements carrying a high volume of heavy vehicles and requiring strengthening to protect underlying layers. Using HiMA technology the thickness of the pavement can be reduced which ultimately reduces the cost of construction. HiMA is also eco-friendly technology as it saves raw materials during construction.

To achieve the excellent performance of HiMA mix the penetration value of the hard grade binder with 25 mm/10 at 25 °C is recommended. The selection high modulus binder is very much important for the production of HiMA mix. In general, a high modulus binder can be obtained through three different methods that increase the stiffness of the bituminous mixture and they are (1) hard binder with low penetration and high softening point [7, 22], (2) polymer modified binder, such as, SBS, gilsonite, ethylene–vinyl acetate (EVA), etc. [14, 26, 30], (Zou et al. 2014), and (3) using modulus agent [27, 28]. To produce a hard grade binder air blown technique was used that was performed excellently against the rutting performance. However, HiMA mixes developed thermal cracking at low temperatures for the cold climatic region [7, 21]. A solution to this problem is the inclusion of additives or modifiers into the bitumen to improve the mechanical properties of the binder (Wang 2018).

Researchers used different types of polymer modifiers such as gilsonite, ethylene–vinyl acetate (EVA), styrene–butadiene–styrene (SBS), polyethylene (PE), crumb rubber (CR), and hydrated lime in the conventional bitumen to develop a hard grade binder which helps to improve the permanent deformation of the bituminous mixture for the construction and rehabilitation of the road network [7, 14, 22, 26, 30], (Zou et al. 2014). It is also reported that the rheological properties of the binder improved significantly if the polymer content is about 6%. The selection of polymer types depends on the aging properties of the binder. Xiao et al. [29] evaluated the performance of modified asphalt and mixtures where they concluded that the high-temperature performance of the modified binder and mix improved with the addition of a high modulus agent whereas it shows adverse effects at low temperatures. Cao et al. [6] report a similar conclusion where they used an SBS-modified binder and an anti-rutting agent for the modification of the bituminous mixture. HiMA technology used with an SBS modifier (relatively high polymer content) was found to be effective in fatigue resistance and it reduces the pavement thickness [26]. Rys et al. [21] reported that HiMA bases had a higher probability of being cracked compared to a conventional mix. Few scholars reported that HiMA can sustain maximum shear stress generated due to vehicular load when HiMA is applied to the middle surface layer. Si and Cao [23] studied the mechanical properties of HiMA and developed structural models of HiMA and conventional pavement and concluded that HiMA improves the rutting resistance and reduces the overall deformation of the pavement. Some researchers studied the modification mechanism of HiMA modifiers using Fourier transform infrared spectroscopy (FTIR) and a scanning electron microscope (SEM) techniques. Quite a few researchers studied the effect of Gilsonite at different dosages to check the performance of the binder and found that rutting resistance of asphalt mix was improved along with moisture resistance whereas it has an adverse effect on fatigue resistance [16, 20, 30], (Nasekrani et al. 2016). Few researchers carried out laboratory performance studies of HiMA (including stiffness, permanent deformation, fatigue cracking, and moisture susceptibility tests) and correlates these with field performance of the HiMA mix [8, 10, 11 15, 24].

The existing laboratory findings suggest that the high content of the polymer improves the performance of the binder and mixture to resist distress, rutting, fatigue, and thermal cracking. Further, the unique features of the SBS modifier slow down the oxidative aging of the binder. This feature of the SBS modifier could affect positively the resistance of the HiMA mixtures to various types of cracking. Therefore, this study is proposed to evaluate the performance of conventional and HiMA (field-produced) binders and mixtures through various laboratory investigations.

2 Objective

The overall objective of this study is to evaluate the performance characteristics of conventional and field-produced high-modulus asphalt mix through various laboratory tests. The specific tasks to achieve this goal are:

1. To evaluate the rheological and chemical properties of the available conventional grade binder and field-produced modified binder.
2. To study the mechanical characteristics of conventional and HiMA mixtures through various laboratory tests.
3. To investigate the performance characteristics of conventional and HiMA mixtures in terms of moisture susceptibility and fatigue resistance.

3 Materials and Methods

3.1 Bituminous Binder and Modifier

In this study available hard grade binder:VG-40, and field-produced high-modulus asphalt (VG-10 blended with SBS modifier at 7.5% dosage): HiMA binders were used. The physical properties of these binders are shown in Table 1. The test results of VG-40 and HiMA binders show that they meet the requirements of Indian standard specifications.

3.2 Aggregate

The physical properties of the aggregate were determined as IS 2386. The obtained results can be seen in Table 1. In this study bituminous concrete graded-I (BC-I) is adopted based on MoRTH guidelines [17] where mid-point gradation is considered for the conventional mix whereas field-produced HiMA mixture (Bharathpur, Rajasthan, India) is named as design gradation. The design gradation is achieved with a certain tolerance limit as prescribed in MoRTH. The obtained gradation for conventional and HiMA mixtures is shown in Table 2.

3.3 Bituminous Mixtures

VG-40 mixture is commonly used to evaluate the effect of the high modulus performance of roads in India. In this study, VG-40 and field-produced HiMA (VG-10 blended with SBS modifier at 7.5% dosage) mixtures' performance were evaluated. The Marshall mix design was conducted by following the Asphalt Institute Manual MS-2. Specimens of 100 mm diameter and 63 ± 2 mm height were prepared by applying 75 blows of Marshall hammer on both the faces of the specimen. Marshall stability test was conducted at 60 °C. Binder content corresponding to 4% air voids

Table 1 Physical properties of VG-40 and HiMA binders

Tests	Test results		IS specifications (Minimum)
	VG-40	HiMA	
Penetration at 25 °C , mm	51	33.5	35 mm
Softening point, °C	53	88.5	50 °C
Ductility, cm	>100	24.75	25 cm
Specific gravity	1.03	1.09	0.99
Kinematic viscosity at 135 °C , Cp	480	2100	400

Table 2 Physical properties of aggregate and aggregate gradation

Test	Results	Sieve size (mm)	Cumulative % weight passing	
			Mid-point (Target)	Design gradation
Aggregate impact value (%)	17	26.5	100	100
		19	95	98.6
Combined flakiness and elongation index	26	13.2	69	83.2
		9.5	62	68.3
Specific gravity	2.61	4.75	45	53.6
		2.36	36	36.9
Water absorption (%)	1.1	1.18	27	27.1
		0.600	21	17.6
Los angeles abrasion value (%)	14	0.300	15	13.2
		0.150	9	10.6
		0.075	2–8	2.32

content was selected as the optimum binder content (OBC). The designed optimum binder content for HiMA and VG 40 mixture are 5.1% and 5.15% respectively.

3.4 Dynamic Shear Rheological (DSR) Test

A linear amplitude sweep (LAS) test was performed using a dynamic shear rheometer (DSR) as per AASHTO TP 101. The fatigue life of both the binders, VG-40 and HiMA was evaluated through LAS test.

3.5 Fourier Transform Infrared (FTIR) Spectroscopy Test

The FTIR test was used to investigate the changes in the chemical composition of bitumen due to aging. In this study FTIR test was performed on the unaged, RTFO aged and PAV aged binders. A Perkin Elmer BX II FTIR spectrometer equipped with an attenuated total reflection (ATR) unit was used in the present study. The spectra were collected in the range of 5000 cm^{-1} to 500 cm^{-1}. Omnic v6 software was used to monitor changes in the spectra of the binders considered in the present study. The carbonyl (I_C=O) and sulfoxide (I_S=O) indices were estimated by using Eqs. (1) and (2) respectively.

$$\text{Carbonyl Index}(\mathrm{I_C} = \mathrm{O}) = \frac{A_{1660-1753}}{\sum A} \tag{1}$$

$$\text{Sulfoxide Index}(\mathrm{I_S} = \mathrm{O}) = \frac{A_{970-1070}}{\sum A} \quad (2)$$

where $\sum A = A_{(2953,2862)} + A_{1700} + A_{1600} + A_{1460} + A_{1376} + A_{1030} + A_{864} + A_{814} + A_{743} + A_{724}$.

3.6 SARA (Saturates, Aromatics, Resins, Asphaltenes)

The Thin-Layer Chromatography with Flame Ionization Detection (TLC–FID) was used for the measurement of four generic fractions (saturates, aromatics, resins, and asphaltene) of bitumen for evaluation of changes in the chemical composition of bitumen due to aging. The solution prepared by dissolving the 10 mg of bitumen sample in a 10 ml DCM solution was used to conduct the test.

3.7 Indirect Tensile Strength (ITS) Test

The ITS test was conducted on the bituminous concrete samples at $4 \pm 0.5\%$ air voids. The test was performed as per ASTM D6931. The test was conducted at 25 °C. The tensile strength of the bituminous mixes was calculated by using Eq. (3).

$$\text{ITS} = \frac{2P}{\pi t D} \quad (3)$$

where, ITS = Indirect tensile strength (kPa), P = Max load (N), t = Sample thickness (mm) and D = Diameter of the sample (mm).

3.8 Resilient Modulus (M_R) Test

The resilient modulus test was conducted on the bituminous mixes compacted at 4 ± 0.5% air voids. The test was conducted at 25, 35, and 50 °C as per BS DD 213 (1993). Three samples were tested for each combination. The resilient modulus of bituminous mixes at different temperatures was estimated by using Eq. (4).

$$\text{Resilient Modulus } (\mathrm{M_R}) = (\mathrm{P} * (\mu + 0.27))/(\Delta \mathrm{h} * \mathrm{t}) \quad (4)$$

where, M_R = resilient modulus (MPa), μ = Poisson's ratio, P = Max load (N), t = sample thickness (mm), D = diameter of the sample (mm), and Δh = resilient deformation resulting from the applied load (mm). μ was assumed to be 0.35.

3.9 Moisture Induced Sensitivity Test (MIST)

To evaluate the effect of moisture on the stiffness of the bituminous mix the moisture-induced sensitivity test (MIST) is performed. The pore pressure developed in the bituminous mix due to repeated movement of vehicles on the roads will be simulated in the laboratory by using the MIST. The resilient modulus test was conducted on the unconditioned and MIST conditioned bituminous mixes compacted at 7 ± 1% air voids. The moisture conditioning of bituminous mixes in the MIST was done by following the ASTM D7870. The resilient modulus of the bituminous mixes in the unconditioned and moisture-conditioned states was measured at 25 °C. The resilient modulus of estimated by following the BS DD 213 [5]. Three specimens were tested for each mix.

3.10 Texas Overlay Tester

The reflective cracking resistance of bituminous mixes was evaluated using the Texas Overlay Tester (TOT). The bituminous mixes of 150 mm diameter and 62 ± 1 mm height was prepared using the Gyratory compactor and compacted to target air voids around 7 ± 1% air voids. The specimens are then sliced from both ends to get the specimens of 76 mm width. Further, specimens were trimmed from top and bottom to get specimens of 38 mm height. The test was performed as per the Tex-248-F at 25 °C. The number of cycles to failure at which the maximum load measured from the initial opening cycle drops to 93% or more was used to compare the reflective cracking resistance of bituminous mixes. The higher value of the average number of cycles to failure indicates a better resistance to reflective cracking. Three specimens were tested for each mix. Before the testing, the specimen was glued to the base plate as shown in Fig. 1. The specimen kept in the Texas overlay tester is shown in Fig. 2.

4 Results and Analysis

4.1 LAS Analysis

To identify the initial properties of the tested material the LAS test is performed with a frequency sweep test. The initial characteristics of the tested material are measured as the alpha (α) parameter. The α parameter obtained at 25 °C for the tested bituminous binders is presented in Table 3.

Table 3 indicates that HiMA is having more fatigue resistance compared to VG-40 as the value of α parameter is higher. During the amplitude sweep test, strain is varying from 1 to 30% by keeping frequency as a constant. The fatigue criterion for the tested materials is registered as τ_{max} which indicates the maximum shear

Fig. 1 Specimen glued to base plate

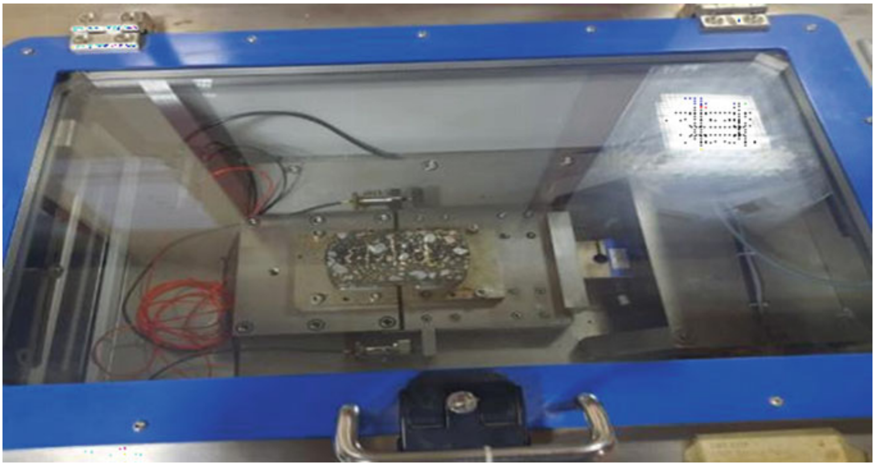

Fig. 2 Specimen in Texas overlay tester

stress occurs during the amplitude sweep test. The shear stress obtained in the tested samples is shown in Fig. 3.

Figure 3 indicates the stress–strain curve for the VG 40 and HiMA binders. This figure shows that in the case of VG-40 it is possible to identify the maximum value of the stress whereas in the case of HiMA that point is clearly not visible. There is no sudden decrease in the stress after the maximum value is reached for HiMA whereas it can be seen in the case of VG-40. This shows that the HiMA binder does not damage during the test condition and it transfers the applied load continuously. In this study fatigue life criterion is obtained corresponding to $\gamma = 5\%$ as seen in Table 3. The obtained fatigue value for VG-40 and HiMA binders are 49,280 ESALs

Table 3 LAS analysis results of VG-40 and HiMA

Parameters	VG-40	HiMA
A	2.80	3.09
ID	7.19	3.18
C0	4.99	2.10
C1	0.048	0.069
C2	0.547	0.45
Df	1701.06	687.73
K	2.26	2.70
A	4×10^8	6.3×10^9
B	5.60	6.18
τ_{max}	5%	5%
Nf	49,280 ESAL	299,124 ESAL

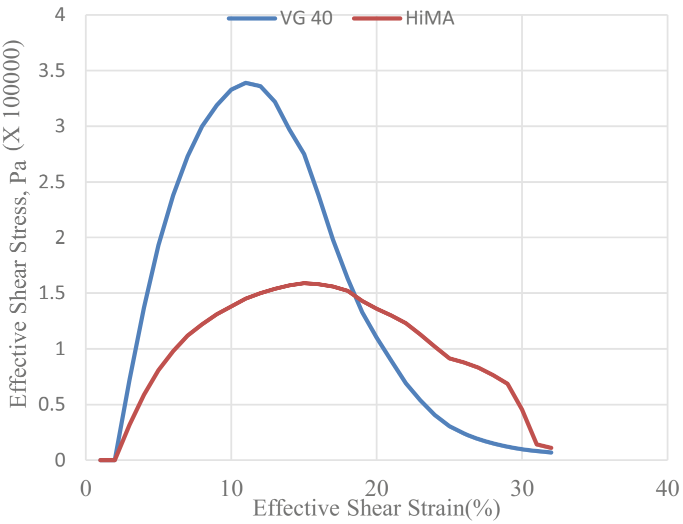

Fig. 3 Shear stress–strain diagram of the selected samples

and 299,124 ESALs respectively which shows that the fatigue properties for the applied strain $\gamma = 5\%$ of HiMA is better as compared to VG-40. Thus, it can be concluded that HiMA possess more resistance to fatigue compared to VG-40.

Table 4 FTIR analysis results of VG-40 and HiMA

Binder type	Aging conditions	Carbonyl index	Sulfoxide index
VG-40	Unaged	0.038	0.139
	RTFO	0.056	0.121
	PAV	0.055	0.14
HiMA	Unaged	0.017	0.057
	RTFO	0.026	0.092
	PAV	0.029	0.066

4.2 FTIR Analysis

The chemical characterization of aging due to oxidation is conducted using FTIR analysis. "Carboxylic acids, hydrides, ketones, and sulfoxides" are the important functional groups obtained during oxidation of binder. The most prevalent functional groups are ketones and sulphoxides, but ketones are less concerned with age hardening. The measure of the aging rate of binders is provided by the peak value of ketones and a measure of oxidative ageing is given by the carbonyl field. Studies show that the ratio between the peak values of these functional classes can be used instead of using peak values to reflect the ageing status of the binder [4, 13, 19]. The IR spectrum of unaged, RTFO aged and PAV aged conducted for VG-40 and HiMA binders. The evaluated results of these binders at different aging conditions can be seen in Table 4.

From Table 4 it is clear that oxidative ageing results in an increase in the sulfoxide and carbonyl indices which results in stiffening and more elastic behaviour, thus increases the brittleness of binder.

4.3 SARA Analysis

In the present study analysis for SARA components (Saturates, Aromatics, Resins, and Asphaltenes) were conducted for VG-40 and HiMA binders at different aging conditions, unaged, RTFO aged and PAV aged as shown in Table 5.

Asphaltene is the most polar and heaviest bitumen fraction, which has the highest effect on bitumen colloidal stability. Table 5 indicates that with aging, aromatics decreases as aromatics converts to resins and asphaltenes due to oxidation. Also, Table 5 shows that asphaltene content increases with aging. Increasing the bitumen asphaltene content decreases the susceptibility to temperature and increases the resistance of the sample to thermal decomposition.

Table 5 SARA analysis results of VG-40 and HiMA

Binder type	Ageing	Saturates	Aromatics	Resins	Asphaltenes
VG-40	Unaged	11.93	52.6	29.84	5.98
	RTFO	12.36	46.33	34.03	7.28
	PAV	12.33	28.77	50.01	8.89
HiMA	Unaged	18.56	29.76	46.51	5.17
	RTFO	20.79	27.87	44.78	6.57
	PAV	19.82	21.36	52.33	6.49

Table 6 ITS values of VG-40 and HiMA mixtures

Sl. no.	VG-40		HiMA	
	Peak load (kN)	ITS (kPa)	Peak load (kN)	ITS (kPa)
1	10.3	1016	14.7	1497
2	9.8	967	14.5	1476
3	10.5	1036	13.7	1393

4.4 Analysis of Indirect Tensile Strength Test

This test is used to evaluate the tensile properties of bituminous concrete mixtures to simulate the cracking properties of the pavement. In this study peak load of the tested samples was obtained and corresponding to each sample tensile strength is calculated using Eq. 3. The evaluated tensile strength of VG-40 and HiMA mixtures is shown in Table 6.

Table 6 indicates that HiMA mixture exhibits more tensile strength than VG-40 which shows HiMA is strong in resistance compared to the VG-40 mix.

4.5 Results of Resilient Modulus Test

The stiffness evaluation of the bituminous mixture is important from pavement design and performance evaluation perspective. Thus, resilient modulus test were conducted for VG-40 and HiMA mixtures at various temperatures as shown in Table 7.

Table 7 Resilient modulus values of VG-40 and HiMA mixtures

Temperature (°C)	Average resilient modulus (MPa)	
	VG-40 mixture	HiMA mixture
25	4200	5521
35	3100	4038
50	1500	2569

Table 8 Moisture resistance of VG-40 and HiMA mixtures

Mix type	Resilient modulus (MPa)		Moisture resistance (%)
	Before MIST	After MIST	
VG-40 mixture	4200	3050	72
HiMA mixture	5521	4135	75

Table 7 indicates that HiMA mixture possess more resilient modulus than the conventional VG-40 mix because of less recoverable strain in case of modified mixtures. With increase in temperature resilient modulus decreases. Because with increase in temperature, bitumen loses its ability to bind the aggregates, therefore the recoverable strain needed increases as a result resilient modulus decreases.

4.6 Analysis of Moisture Induced Sensitivity Test

MIST is conducted to know the asphalt mixture sensitivity to water and stripping. The ratio of resilient modulus after MIST and before MIST gives moisture resistance of the asphalt mixture. Moisture resistance of VG-40 mixture and HiMA mixture is presented in Table 8.

From the result it is clear that HiMA mixture possesses more moisture resistance than the conventional VG-40 mixture which means moisture damage is more in case of the VG-40 mixture. This damage occurs because of the interaction between moisture and asphalt binder aggregate which results in reduction of adhesion between the aggregate and asphalt binder which in turn causes stripping of the bitumen and also leads to various pavement distresses like rutting and fatigue cracking. From the statistical analysis, the t-test is used to compare the means of the two groups. a significant difference for both mixes at 95% confidence interval was found.

4.7 Results of TOT Test

It is an effective method to assess asphalt mixture resistance to fatigue/reflection cracking resistance. In this study VG-40 and HiMA mixes were tested in TOT and thus the required number of cycles corresponding to failure was obtained, as shown in Table 9.

Table 9 Number of cycles to failure for VG-40 and HiMA mixtures

Average number of cycles to failure	VG-40 mixture	HiMA mixture
	88	172

Table 9 indicates that; High modulus asphalt mixture requires more number of cycles to failure compared to conventional VG-40 which shows that the HiMA mixture is more resistant to fatigue/reflective cracking compared to the conventional mixture.

5 Conclusions

From the bitumen and bituminous mix performance tests conducted on conventional viscosity grade bitumen and HiMA bitumen, the following conclusions are drawn.

- HiMA bitumen has higher fatigue life compared to viscosity grade bitumen.
- Lower value of carbonyl index and asphaltene after RTFO and PAV aging indicate that HiMA bitumen has better resistance to aging.
- Higher value of indirect tensile strength of the bituminous mix prepared with HiMA as compared to viscosity grade bitumen indicates that the HiMA modified mix has better resistance to strength, and cracking.
- Higher value of resilient modulus at different test temperatures indicate that the HiMA mix was stiffer than the VG40 mix. In addition to this, dless reduction in resilient modulus after moisture conditioning in MIST indicate that the HiMA mix has better resistance to moisture damage also.
- From the TOT test, the higher value of number of cycles to failure for the HiMA mix indicates that the HiMA mix has better resistance to fatigue/reflective cracking resistance as compared to the VG40 mix.

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Influence of Feeding Sequence on the Fluidity and Fluidity Time of Cement Asphalt Mortar for High-Speed Rail Slab Track

Rahul Reddy Banapuram, Kranthi K. Kuna, and M. Amarnatha Reddy

Abstract Cement Asphalt Mortar (CA mortar) is one of the most important materials in the non-ballast slab track for High-Speed Rail (HSR) systems. The fluidity and fluidity time of CA mortar are important parameters for this application. Both these requirements are governed by the early age interaction between cement, bitumen emulsion, and admixtures. The present study investigated the influence of the feeding sequence of these primary constituents of CA mortar on its fluidity and fluidity time. Flow time measurement by Brass J funnel was used as a measure of fluidity and the mechanism behind the flow time variation is analyzed through viscosity and adsorption behavior of CA paste. The sieving method was adopted to investigate the adsorption behavior of bitumen droplets on the cement under different feeding sequences. The results showed that adding bitumen emulsion, water, and water-reducing admixture (WRA) initially increased flow time and apparent viscosity, which is not ideal. Test results of adsorption indicated that the increase in particle size was mainly attributed to higher adsorption (adsorption value) resulting in higher viscosity and flowtime. The feeding sequence in which water-reducing admixture (WRA) was premixed with cement and water prior to introducing bitumen emulsion proved to be beneficial to obtain the desired fluidity and fluidity time.

Keywords Cement asphalt mortar · Bitumen emulsion · Fluidity

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1 Introduction

With the introduction of High-Speed Rail (HSR), there is an increasing need to have a new type of track system in lieu of traditional ballasted track systems that undergo excessive deterioration due to the crushing of the ballast under high-frequency loading. The resulting safety and concern issues in the HSR were addressed by introducing a non-ballast track system in the Shinkansen slab track system employed by the Japanese National Railways. Since then, this non-ballast slab track has found its application in HSR tracks of countries such as Germany, China, and Korea due to its popularity over conventional ballasted tracks for reduced structural height, low maintenance, better damping properties, and increased service life [1–3]. A non-ballast track system consists of a Cement Asphalt Mortar (CA mortar/CAM) sandwiched between a concrete roadbed and a precast concrete slab at the top. A typical cross section of a non-ballast slab track is shown in Fig. 1. CA mortar is a composite material made of Portland cement (C), bitumen emulsion (BE), sand (S), water (W), and chemical admixtures as its primary components. The CA mortar is produced at the site and injected between the track slab and track bed through an inlet valve [1]. CA mortar used for HSR can be classified into two types based on the strength requirement. One with high elastic modulus and strength is called CAM II (B/C ratio: 0.2–0.6) as used in BÖgl slab track, Germany. The other with low elastic modulus and strength referred to as CAM I (B/C ratio: 0.6–1.2) employed in the Shinkansen slab track, Japan [4]. With a zero-fatality record for Shinkansen technology since its inception, India's first HSR will emulate Shinkansen technology with the target of achieving speed, service, and safety. Therefore, this study focuses on producing a low-strength CAM I, hereafter referred to as CA mortar.

As an injecting material, CA mortar is mixed on-site and has a space of 50 mm in actual conditions between the concrete roadbed and precast slab, thus it must possess

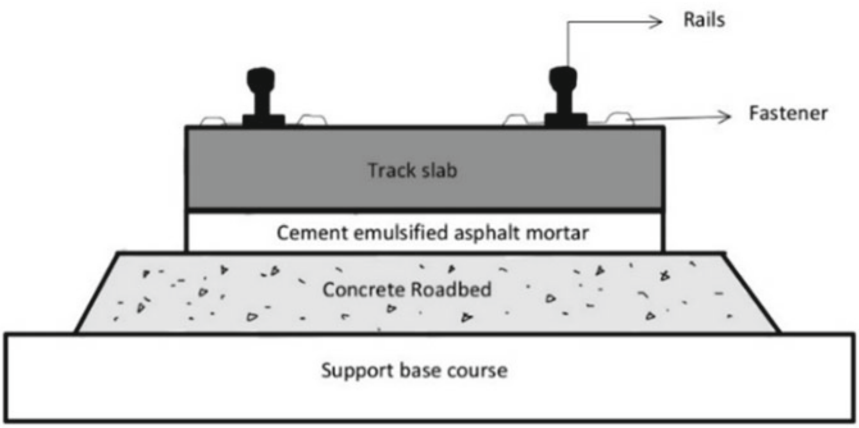

Fig. 1 Schematic diagram of HSR non-ballast track structure

good and stable fluidity of 16–26 s of flow time for a duration of 30 min or more for a successful application without bleeding or segregation [5]. Any variation in the fluidity of CA mortar will have an adverse effect on the density, air content, and mechanical properties of hardened CA mortar. The fluidity of CA mortar is influenced by the early age interaction between cement, bitumen emulsion, and Water-Reducing Admixture (WRA). During the process, two counterbalancing processes occur when bitumen emulsion comes into contact with cement: cement hydration and bitumen emulsion breaking, in which the gain of water promotes the former and the loss of water accelerates the latter. However, controlling the interaction between cement and bitumen emulsion is no easy task and if left unchecked, it will result in undesirable flocculation or coalescence of bitumen emulsion [6]. Fluidity also dictates the density and mechanical properties of hardened CA mortar [7]. Tao and Shuguang [8] studied the effect of direct interaction of bitumen emulsion and cement and indicated that direct interaction resulted in a retarding effect on the hydration of cement [8].

The key to producing a consistent CA mortar is to make sure that the raw materials are mixed uniformly and to prevent the agglomeration of constituents. Ouyang et al. studied the interaction between cement, WRA, and bitumen emulsion and concluded that the direct interaction of emulsifiers with WRA affects the water-reducing ability and weak dispersion of cement [7]. If the flowability is too high, the fine aggregate material would settle, and if it's too low, it would result in CA mortar not filling the gap between track bed and track slab causing a void phenomenon [9]. One way to meet the fluidity requirement is by understanding the interaction between bitumen emulsion, WRA, and cement. Previous research by Hu et al. studied the particle size variation of CA mortar over time by means of adsorption theory and concluded that the adsorption behavior of bitumen droplets onto cement contributed to an increase in particle size affecting the fluidity [10]. Ouyang et al. studied the compatibility between cement and bitumen emulsion as a function of mixing stability by analyzing the viscosity of CA paste. Their results indicated that viscosity can be used as an index of demulsification [11, 12].

To obtain consistent results is to ensure homogeneous mixing of the constituents. In this process, the existing guidelines by China Railway Track System (CRTS) specify a proper feeding sequence of constituents, adequate mixing time, and mixing speed as the key parameters that are to be decided based on the trials [5]. The performance of bitumen emulsion-based materials is highly sensitive to the type and nature of other constituents. The type of cement and the nature of sand influence the fluidity and interaction of CA mortar. Past research on CA mortar focused mainly on the interaction and optimization of bitumen emulsion with cement and its evolution of breaking behavior, but the right CA mortar preparation on the order of bitumen emulsion addition lacks due attention. Therefore, a fundamental investigation of the fluidity and fluidity time of CA mortar is in dire need of both scientific and engineering applications. The objective of the research is to study the fluidity of CA mortar as a criteria in choosing the right feeding sequence and present a hypothesis on the mechanism behind the progression in fluidity and fluidity time of CA mortar. The study also aimed to recommend an approach to feeding materials to obtain a stable and consistent CA mortar for successful application. This will benefit the practitioners in the production of CA mortar with the desired fluidity with a consistent fluidity time for successful application.

2 Materials and Methods

2.1 Materials

Ordinary Portland Cement, OPC 53, conforming to IS 269: 2015 along with calcium sulphoaluminate cement (CSA), was used as an inorganic binder. The properties of cement used in the study are listed in Table 1. Cationic slow setting bitumen emulsion consisting of a quaternary amine as an emulsifier was specifically formulated to meet the performance requirements of CA mortar, and the properties are shown in Table 2. Tap water (W) and dry river sand (S) with a fineness modulus of 1.6 were used in the preparation. Along with the above constituents, polycarboxylate-based water-reducing admixture (WRA), CSA as shrinkage resisting agent, aluminum (Al) as an expansive agent, and rosin-type air-entraining agent (AE) were used to meet the technical requirements of CA mortar (CAM 1) as adopted in Japanese HSR and CRTS [5].

For the CA mortar production, inorganic binder, sand, and bitumen emulsion proportions were maintained in the ratio of 1:2:1.4. The relative ratios are expressed w.r.t. inorganic binder. The dosage of admixtures and water arrived by random controlled trials to meet the requirements of CA mortar as per CRTS [5].

Table 1 Properties of Portland cement used in present study

Properties	OPC 53
Fineness (m^2/Kg)	324
Normal consistency, %	30
Setting time, min	
Initial	135
Final	240
Soundness	Satisfied
Compressive strength, MPa	
3 days	30
7 day	40.5
28 day	56

Table 2 Properties of bitumen emulsion tested as per IS 8887: 2018

Test on emulsion	Value	Test on residue from evaporation	Value	Test standards
Residue on 600 microns IS sieve, %	0.03	Residue by evaporation, %	60.91	IS 8887: 2018
Viscosity by saybolt furol at 25 °C, s	51	Penetration 25 °C, dmm	47.6	
Particle charge	Positive	Ductility 27 °C, cm	33	
Stability to mixing with cement, %	0.1	Solubility in trichloroethylene, %	99.4	

2.2 Methodology

The tailor-made bitumen emulsion was formulated to meet the specific requirements of CA mortar. This involved the production of water phase of emulsion with relatively high dosage of emulsifier to address the rapid breaking behavior of bitumen emulsion when it interacted with other CA mortar constituents. AMT lab-pilot colloidal mill was used to produce the bitumen emulsion. The raw materials as per the proportion mentioned above were fed into the mortar mixed in the below-specified feeding sequence (see Fig. 2). CA mortar at the end of the mixing sequence was taken into Brass J funnel for the measurement of flow time. Rheology by means of viscometer was studied to understand the viscosity variation under the effect of feeding sequence. The adsorption behavior of bitumen droplets over cement was characterized by studying the adsorption ratio in fresh CA paste prepared under different feeding sequences. As understanding the flowability in terms of interaction between cement, WRA and bitumen emulsion is our point of interest, sand is not considered in the rheological and adsorption ratio studies.

Flow Time Measurement and Feeding Sequence

The flowability of CA mortar was evaluated by flow time. Flow time in seconds is the time required for CA mortar to flow through a Brass J funnel having a volume of 640 ml and a top diameter of 70 mm and a bottom orifice diameter of 10 mm. The test was repeated for every 5-min interval till it is in the workable range of 16–26 s.

In fresh mix CA mortar, the flowability is mainly governed by the mutual interaction between bitumen emulsion breaking and cement hydration. In order to understand the flowability behavior of CA mortar under different feeding sequences, mix proportions involving cement, water, CSA, WRA, and bitumen emulsion were added and mixed in the same sequence for the given rpm and mixing time as mentioned (see Fig. 2). In the feeding sequence, the mixing time and mixing speed were chosen in a way that it would result in low foaming after the introduction of bitumen emulsion and air-entraining agent. A relatively higher mixing speed and mixing time was followed for better dispersion when dry materials such as cement and sand were introduced.

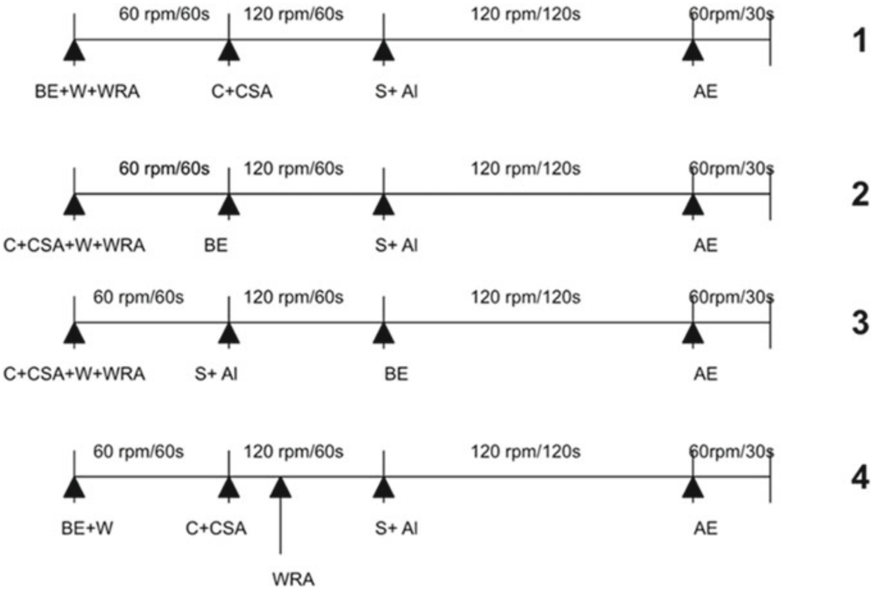

Fig. 2 Feeding sequence adopted in the present study

Constituents with different feeding sequences were introduced in the mortar mixer. Sequence 1 involves the addition of BE, W, and WRA first and mixed at 60 rpm for 60 s followed by introducing C and CSA with mixing speeds maintained at 120 rpm for 60 s. Next premixed sand and aluminum powder are introduced by maintaining 120 rpm for 120 s. Finally, with a reduced mixing speed of 60 rpm, AE is added and mixed for 30 s. The mixed sample is taken into the brass J funnel for the measurement of flowability. Similarly, constituents were introduced into the mixer in different feeding sequences as illustrated (see Fig. 2) and the flow time reading was recorded.

Rheological Testing and Adsorption Test

To capture the interaction between cement, CSA, water, bitumen emulsion, and WRA, constituents were mixed in the same sequence as mentioned (see Fig. 2) except sand to form a cement-bitumen emulsion paste. The test was carried out at controlled room temperature with a Brookfield viscometer, DV 2T with the spindle number 29. Test protocol includes shearing the fresh CA paste with a gradually increasing shear rate of 5 s^{-1} and corresponding apparent viscosity was recorded.

One of the first forms of interaction when a bitumen emulsion is exposed to a mineral material is the adsorption of bitumen droplets over the mineral surface and in this case it is the cement grains. Studying the adsorption behavior under different mixing sequences can help in understanding the particle size variation and consequently the fluidity of the CAM. As a result, the adsorption behavior between a bitumen emulsion and cement grains is characterized by adsorption ratio [10, 13]. Adsorption ratio is the amount of bitumen emulsion adsorbed on cement divided by the amount of bitumen content in bitumen emulsion. The constituents excluding sand were weighed and mixed as per the mixing sequence mentioned above (see Fig. 2). The mixing time is restricted to 120 s. A representative sample of the freshly mixed CA paste is passed through 1.4 mm sieve and the residue on the sieve with a pan was placed on an oven at 110 °C for 90 min. The adsorption ratio is calculated from the below-mentioned Eq. (1):

$$Adsorption\ ratio,\ \% = \frac{m_2 - m_1}{m_0} \tag{1}$$

where m_1 is the empty weight of sieve with pan, m_2 is mass of oven-dried residue of CA paste retained on 1.4 mm sieve with pan, and m_0 is bitumen content in emulsion before mixing.

3 Results

Figure 3 shows the fluidity of different mixing sequences. From the fluidity curve, it can be seen that the feeding sequence has a significant impact on the flow time of CA mortar. Sequence 3 resulted in an instant breakdown of bitumen emulsion making the CA mortar with no flow properties. This is because the fine dry materials (sand and cement) added initially consumed all of the free water that was added. Due to the bitumen emulsion upon contact with large surface materials like cement, CSA, and sand, the reactivity accelerated resulting in the instant breaking of bitumen emulsion. Sequences 1 and 2 resulted in CA mortar with lower flow time than sequence 4. This is due to added WRA that affects fluidity in three ways, firstly, WRA can prevent the flocculation of cement powder, and secondly, the ability of cement particles to adsorb bitumen droplet reduces if the particle has already absorbed WRA, and lastly, WRA effectively prevents the direct interaction of cement and bitumen emulsion. Among sequences 1 and 2, sequence 2 performs better in terms of flow time as the added WRA has effectively coated and dispersed the cement particles, whereas the added WRA into bitumen emulsion in sequence 1 resulted in competitive adsorption of WRA with the emulsifier of the bitumen emulsion and cement causing partial dispersion. This could have resulted in an increase in particle size in CA mortar.

The viscosity tests carried out on the feeding sequence prove to be in line with the observations from flow time tests. It can be seen that with sequences 1 and 2 (see Fig. 4), CA paste following sequence 1 exhibited higher viscosity than sequence 2. This is because when cement is premixed with water and WRA (sequence 2), the cement particles first adsorbs the available WRA effectively, as opposed to what happened in cases 1 and 4. The reason for an increase in viscosity for sequences 1 and 4 can be because of the partially dispersed cement particles adsorbing the emulsifier and water from the bitumen emulsion directly causing flocculation and destabilizing the emulsion.

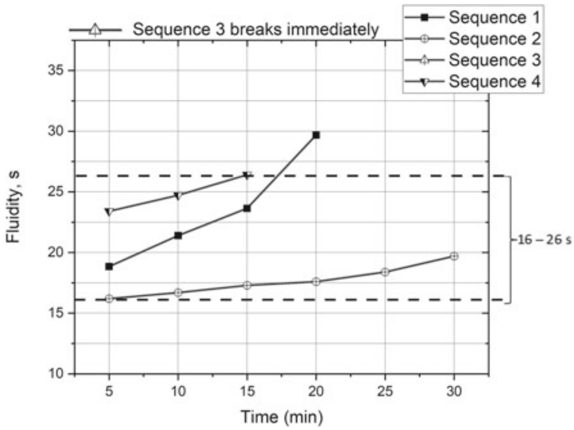

Fig. 3 Fluidity curve of CA mortar over time for different feeding sequence

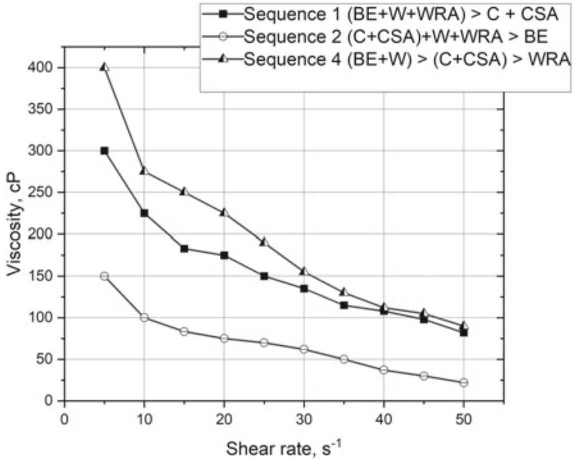

Fig. 4 Apparent viscosity and shear stress of CA mortar with different feeding sequence

As seen in Table 3, the adsorption ratio for sequence 2 is significantly lower than that of sequences 1 and 3. The adsorption of bitumen emulsion on cement was lower in sequence 2 as WRA was premixed with cement. Sequence 4 had a higher adsorption ratio than sequence 1 which might be due to the direct interaction between bitumen emulsion and cement. Figure 5 shows an illustration on the hypothesis of the possible interaction between a bitumen emulsion, cement, and WRA. It is to be noted that the increasing magnitude of adsorption ratio is analogous to an increase in particle sizes in CA paste which attributes to higher apparent viscosity.

Table 3 Adsorption ratio in CA paste with different feeding sequence

Time, min	Adsorption ratio, %		
	Sequence 1	Sequence 2	Sequence 4
5 min after mixing	16.8	8.4	20.1

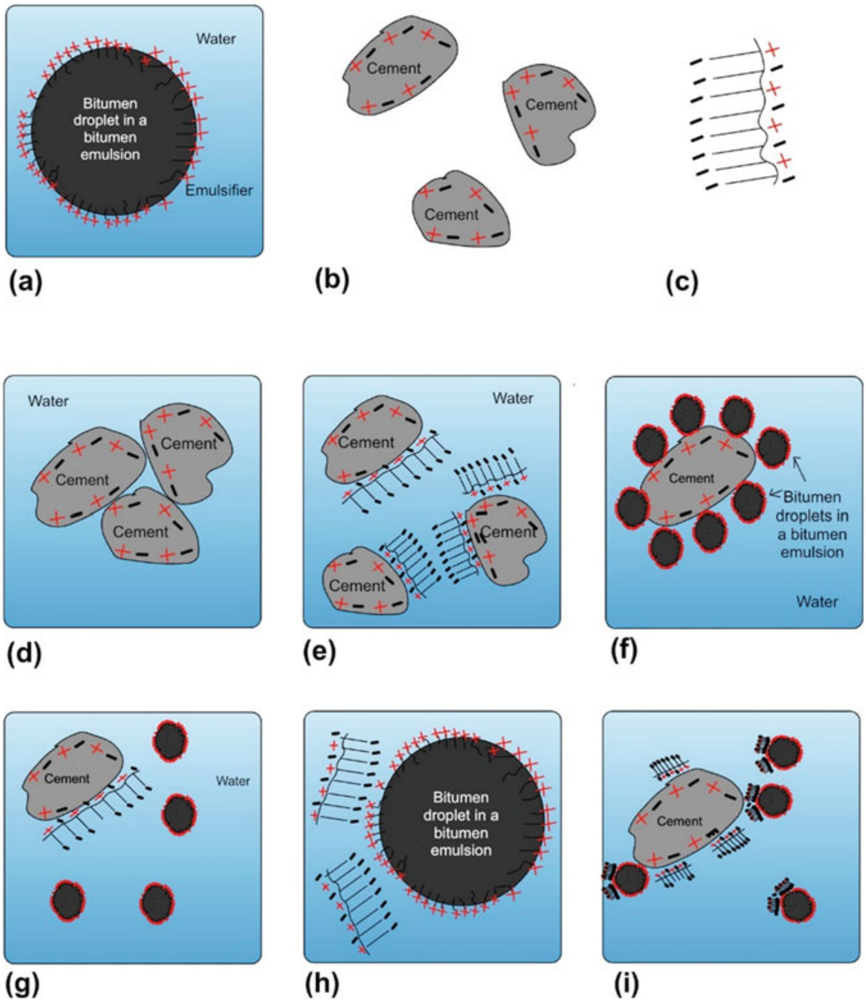

Fig. 5 Interaction of bitumen emulsion with cement under different conditions. **a** Bitumen emulsion structure; **b** and **c** Cement grain and WRA structure; **d** Cling effect of cement grains when exposed to water; **e** Dispersed cement grains in water medium in the presence of WRA; **f** Adsorption of bitumen droplet over cement grains w/o WRA; **g** Adsorption of WRA over cement grains preventing the BE and C interaction (a possible scenario in sequence 2); **h** and **i** Competitive adsorption of WRA between BE and C causing partial dispersion (a possible scenario in sequence 1 and 4)

4 Conclusions

In order to produce a homogenized and stable CA mortar for injecting application in non-ballast slab tracks, it is essential to accurately study the fluidity characteristics of the CA mortar. Hence, the primary objective of the study was to demonstrate the fluidity of CA mortar under different feeding sequences. The paper also proposes a procedure for the sequence of feeding the constituents for a stable CA mortar with optimum fluidity. Based on the results, the following conclusions can be drawn:

- Selecting appropriate materials and constituent proportions does not always guarantee the desired fluidity of CA mortar. For a given CA mortar composition, there exists an order of feeding the constituents for which the desired fluidity is observed.
- The addition of WRA had an adverse effect when it directly interacted with bitumen emulsion. Direct interaction of WRA with emulsifiers affects the water-reducing effect of WRA on cement paste. This could result in lower fluidity.
- The feeding sequence should be selected in a way that it should avoid the direct interaction of bitumen emulsion and cement. The order of feeding sequence in which cement was premixed with water and WRA and then introduced bitumen emulsion proved to have a stable CA mortar with desired fluidity.
- The variation pattern of adsorption ratio correlated well with that of viscosity of CA paste for different feeding sequences indicating that the particle size variation is partially accountable for the fluidity of CA mortar.

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Stiffness and Cracking Resistance Evaluation of Cold Bitumen Emulsion Mixtures Incorporated with Waste Glass Aggregates

Mohammad Iqbal Malik, Mohammad Shafi Mir, Bijayananda Mohanty, and Mehnaza Akhter

Abstract Continuous use of hot mix asphalt (HMA) accelerates environmental deterioration, fossil fuel consumption, global warming, and depletion of natural resources. Further, waste generation and its disposal problem are also a threat to environment. The production of waste and the use of energy/virgin materials in HMA construction must be addressed concurrently. A right step toward the creation of environment-friendly road infrastructure is the use of Cold Bitumen Emulsion Mixtures (CBEMs), a form of Cold Mix Asphalt (CMA). Cold mix asphalt may be made more environment friendly by using waste materials as fine aggregates. In this study, Waste Glass (WG) is substituted for virgin fine aggregate at various percentages ranging from 0 to 100% (with 20% increments) in the binder layer of the CBEM. As per Marshall stability, Marshall flow, indirect tensile strength (ITS), and resilient modulus, the mechanical performance of CBEM-WG mixtures is assessed in this work. The performance of various CBEM-WG mixes is compared with each other, normal CBEM (NCBEM) and also with HMA. According to the findings, mechanical performance of CBEM having WG contents up to 60% was equivalent to that of normal CBEM (NCBEM) and conventional HMA, and it demonstrated superior performance at 60% plus WG content levels. The statistical analysis was performed to prove the feasibility and validity of replacing virgin materials with waste glass in terms of mechanical properties. The coefficient of determination $R^2 > 0.9$ for all properties indicated addition of waste glass has significant impact on mechanical performance.

Keywords Cold mix asphalt · Waste glass · Sustainability · Environment friendly

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1 Introduction

The most often utilized ingredient for flexible pavements worldwide is hot mix asphalt (HMA). Studies show that flexible pavements make up roughly 95% of the world's roadways [1]. Use of Hot Mix Asphalt (HMA) is constrained because to concerns about greenhouse gas emissions, energy use, and the depletion of natural resources [2]. Another issue, especially in colder parts of the world, is the shorter working time for HMA usage due to the high temperature necessary while mixing, hauling, and paving. Contrarily, Cold Mix Asphalt (CMA) mixes are made by combining cold particles and binder at room temperature [3–6]. For the construction of pavement, cold mix asphalt combinations have shown to have reduced environmental effects, be cost-effective, and be safe [3]. The most popular form of CMA is Cold Bitumen Emulsion Mixtures (CBEMs), which has the advantage of energy and material saving along with cost reduction when compared to HMA [7, 8]. However, because to its weak early strength and increased porosity, CMA has seldom been employed in structural layers [9]. This is because it takes more time for the curing process to complete after paving. Additionally, CMA mixtures are extremely delicate to rainfall at a young age [10]. Numerous experiments have been conducted with the goal of improving CBEM performance utilizing fillers like lime, cement, etc. [11]. The utilization of waste materials including motor oil, waste glass, fly ash, palm leaf ash, and paper sludge ash for improving the qualities of pavement mixtures is one of the ecologically friendly efforts in the pavement construction sector [12–15]. Around the world, there is a huge problem with the production of waste glass since more than 10 million tons of material need to be disposed of annually [16]. Crushed glass has been used as aggregate in asphalt mixes with improved characteristics and in road subgrades [17]. "Glassphalt mixes", which utilize discarded glass in HMA, have been in use since the late 1960s [11]. Large-scale trials in the United Kingdom have assessed the performance effects of replacing 30% of the aggregates with waste glass [18]. It will be significantly more advantageous to use waste materials in Cold Mix Asphalt (CMA) rather than Hot Mix Asphalt (HMA). The primary worldwide issues driving the switch from HMA to CMA are excessive energy use, high greenhouse gas emissions (GHGs), challenging working conditions, and worker health risks. Numerous researches have been conducted to investigate at the use of waste products and byproducts to improve cold mixes. Waste glass aggregate incorporation into CMA has not yet been fully investigated. In the first case, Thanaya [19] integrated waste materials in CBEMs for his dissertation study in 2003 and looked into the impacts on cold mixes. He substituted filler with quick setting cement in cold mixtures and fine aggregates with waste glass in quantities of 2.5, 5, 10, 15, and 20% by weight, concluding that these substitutions produced appropriate mechanical characteristics. He also came at the conclusion that using waste glass to substitute fine aggregates by 30% resulted in indirect tensile stiffness modulus (ITSM) values of about 4852 MPa. But to prevent fracture while handling, waste glass should be included, especially as fine aggregates with a size of less than 4.75 mm [20]. A mixture's resistance to raveling and stripping decreases if glass particles are larger than 4.75 mm [19].

Numerous researches claim that adding lime or cement (typical Portland cement) reduces stripping issues [20, 21]. Numerous studies have recommended using mixes with 10–15% crushed glass and a maximum particle size of 4.75 mm in wearing course [22]. Glass particles have an elevated friction angle (about 50°) and high angular form with greater crushing, which helps to improve the lateral stability of asphalt mixes [23]. A recent study by Kadhim et al. [24] on the integration of waste glass into CBEMs focused on surface layers and heavy traffic load circumstances and was completed in 2019. They used fine waste glass instead of fine aggregates in doses of 25, 50, 75, and 100%, with a maximum size of 2.36 mm, and observed greater resistance to rutting, plastic flow, and water sensitivity than HMA.

There is currently very little literature available about studies for assessing bituminous mix for CBEM binder courses incorporating waste glass, despite extensive study on the manufacturing of many types of CBEMs. This endeavor is intended to have two benefits: first, it will solve the problem of waste disposal by using WG in pavements, and second, it will limit the use of virgin raw materials in asphalt pavements, promoting the sustainability of the pavement sector. The main goal of this study is to create a unique Cold Bitumen Emulsion Mixture (CBEM) for binder course that uses waste glass as a partial replacement of fine aggregates (0–100%). This study assesses the mechanical performance of CBEM-WG mixtures in terms of Marshall stability, Marshall flow, Indirect Tensile Strength (ITS), and Resilient Modulus (ITSM). The performance of various CBEM-WG mixes is compared with each other, normal CBEM (NCBEM) and also with HMA. According to the findings, mechanical performance of CBEM having WG contents up to 60% was equivalent to that of normal CBEM (NCBEM) and conventional HMA, and it demonstrated superior performance at 60% plus WG content levels. The statistical analysis was performed to prove the feasibility and validity of replacing virgin materials with waste glass in terms of mechanical properties. The coefficient of determination $R^2 > 0.9$ for all properties indicated addition of waste glass has significant impact on mechanical performance.

2 Objectives of Study

The following are the study’s goals:

i. To evaluate the stiffness and cracking resistance qualities of CBEM for the binder layer using waste glass (WG) as fine aggregate.
ii. To determine and suggest a sustainable CBEM for the binder layer by comparing the performance of various CBEM-WG blends with normal CBEM/HMA.

3 Methodology

Figure 1 displays the study's research methodology. Waste glass, binders, fillers, and aggregates were acquired and characterized. HMA, normal CBEM, and CBEMs with waste glass as fine aggregate were developed, produced, and evaluated for a variety of qualities including Marshall Stability, Marshall Flow, Indirect Tensile Strength, and Resilient Modulus. The test results of CBEMs including waste glass as fine aggregate were studied and compared. It was determined what percentage of waste glass provided the optimum overall performance.

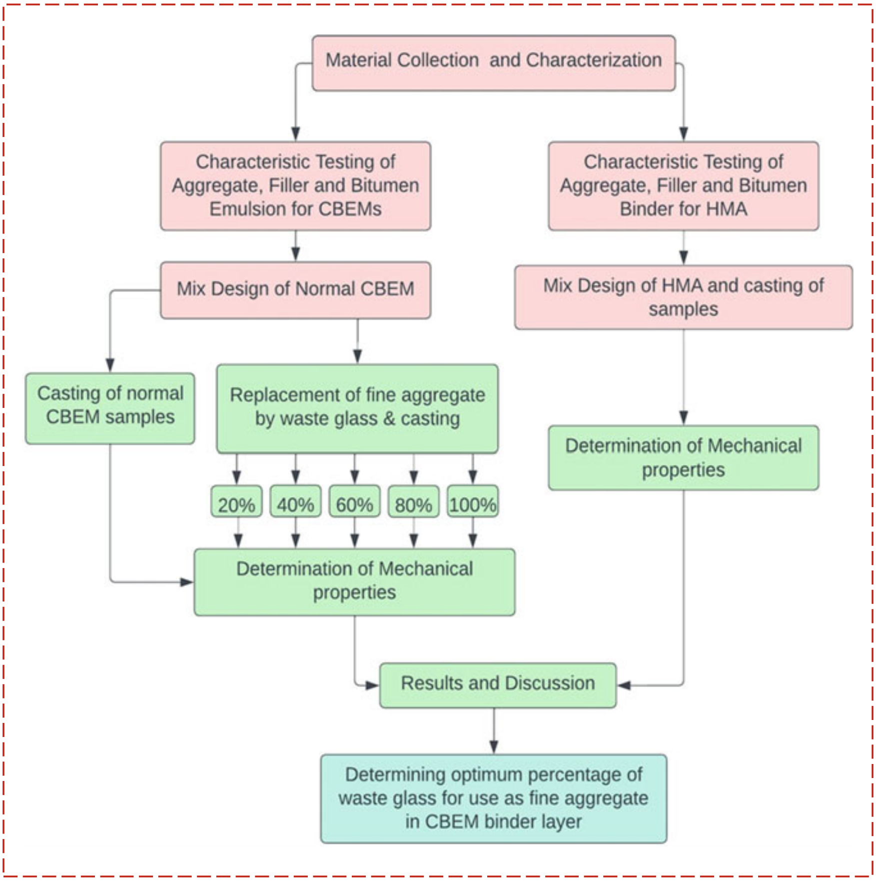


Fig. 1 Flowchart depicting methodology of study

4 Material Characterization

4.1 Aggregates

In Srinagar, Jammu and Kashmir, India, a stone crusher supplied the coarse and fine aggregates. Table 1 shows the distinctive qualities of these siliceous aggregates. The aggregates were left to dry and sorted according to MORTH's sieve analysis [25]. Dense Bituminous Macadam-Grade-2 (DBM-II), which has an acceptable compacted layer thickness of 50–75 mm, was employed as the binder layer for both the HMA and the CBEM. The particle size distribution of the chosen binder layer for HMA and CBEM is shown in Fig. 2.

4.2 Filler

In both HMA and CBEMs, ordinary Portland cement (OPC) was used as filler. OPC (43 grade) was procured from Khyber Cement Industries, a local cement producer. In order to enhance interface bonding, increase early strength, remove trapped water from the mixture, and lessen the effect of stripping, OPC filler was added at a rate of 6% to all combinations in accordance with MORTH [25].

4.3 Binder

For the conventional HMA mixes employed in this investigation, Indian Oil Corporation Ltd. (IOCL) provided viscosity grade 10 bitumen binder (VG 10), which is equivalent to penetration grade 80/100 designed for usage in cooler climates. For better adhesion between aggregate particles, cationic slow setting type-2 bitumen emulsion for CBEMs was also purchased straight from IOCL under the trade name Durapave.

4.4 Waste Glass

The mostly annealed glass waste was procured free of charge from a neighborhood glass seller in Srinagar as shown in Fig. 3. To satisfy the required fine aggregate gradation, the dirt-free waste glass was ground and sieved through 2.36 mm IS sieve. Table 2 lists the chemical characteristics of waste glass aggregate as determined by the X-ray diffraction (XRD) method. To fully comprehend the qualities, Field Emission Scanning Electron Microscopy (FESEM) was used to determine the shape of waste glass. The microscopic picture of glass waste is shown in Fig. 4. Waste glass

Table 1 Characteristic properties of aggregates

Aggregate variety/size	Test	Value obtained (%)	Standard values of DBM [MORTH]
40 mm	Impact value	10	<27%
	Flakiness and elongation index	27	<35%
	Aggregate crushing value	14	<30%
	Los Angeles abrasion	12	<35%
	Specific gravity	2.76	2.5–3.2
	Water absorption	0.8	<2%
20 mm	Impact value	12	<27%
	Flakiness and elongation index	25	<35%
	Aggregate crushing value	15	<30%
	Los Angeles abrasion	15	<35%
	Specific gravity	2.77	2.5–3.2
	Water absorption	0.88	<2%
10 mm	Impact value	13	<27%
	Flakiness and elongation index	24	<35%
	Aggregate crushing value	17	<30%
	Los Angeles abrasion	19	<35%
	Specific gravity	2.75	2.5–3.2
	Water absorption	0.76	<2%
Fine aggregate, Sand	Specific gravity	2.72	2.5–3.2
Stone-dust	Specific gravity	2.78	2.5–3.2
Filler (cement)	Specific gravity	3.04	2.5–3.2

was analyzed using XRD as shown in Fig. 5. Shape, size, and sharp edges of waste glass particles were thought to be very influential in improving mixed qualities [20].

(The above figure presents X-ray diffraction analysis of waste glass powder where in it is found that SiO_2 is predominant component present which can enhance formation of cement hydration products in presence of water).

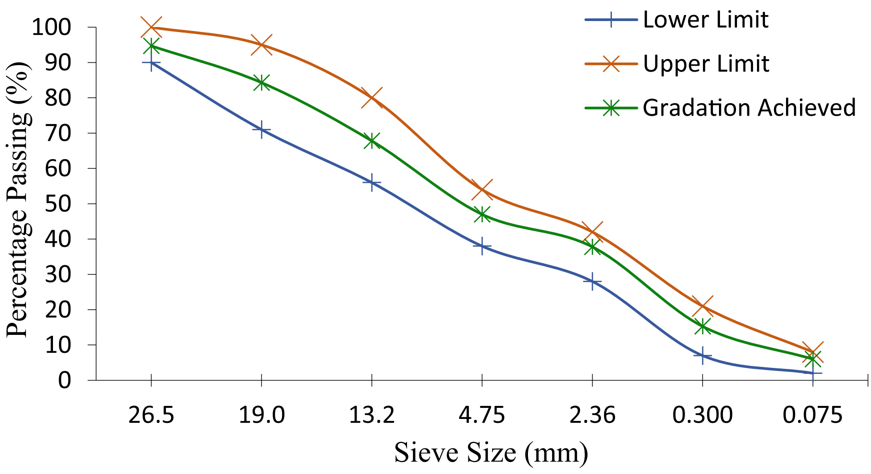


Fig. 2 Aggregate gradation curve

Fig. 3 Waste glass (unpulverized)

Table 2 Chemical composition of waste glass (%)

SiO_2	CaO	Al_2O_3	MgO	Na_2O	SO_3
72.1	8.1	1.6	2.5	13.1	0.3

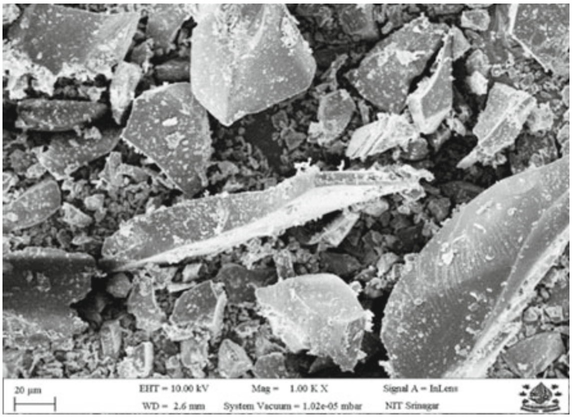


Fig. 4 Microstructural image of waste glass

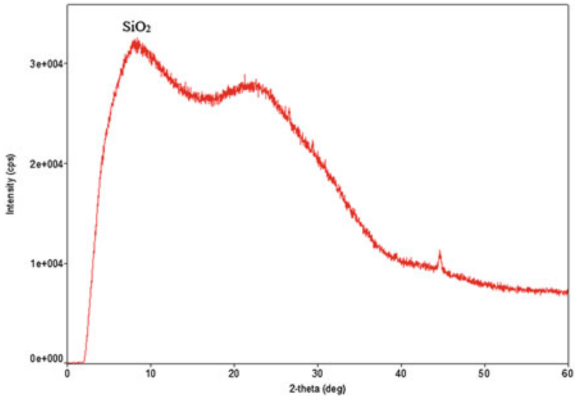


Fig. 5 XRD analysis of waste glass (WG)

5 Laboratory Program

5.1 Preparation of Samples

Marshall Mix Design was used to create conventional HMA specimens. According to MORTH [25], four distinct bitumen binder content groups were chosen: 4.5, 5.0, 5.5, and 6.0%. The optimum bitumen concentration (OBC) was discovered to be 5%. For each binder content, three specimens were cast. For conventional HMA, VG10 binder and an aggregate gradation based on Dense Bituminous Macadam DBM-II binder layer [25] were chosen.

The "Marshall method for emulsified asphalt aggregate cold mixture design MS-14" (Asphalt Institute, 1989) [26] was used to create all CBEM samples for this inquiry in combination with regional specifications for thick graded HMA binder layer. The initial residual binder content (IRBC) was found to be 6% by weight of aggregates. Initial emulsion content (IEC) value was calculated as 9.23% by weight of total mix. Finally, the optimum bitumen emulsion content was arrived at which amounts to 9.5% by weight of aggregates. Optimum pre-wetting water content (OPwWC) was calculated as 3.5%. Total liquid content (TLC) came out to be 13%. The mixing of CBEM was achieved as per MS-14 and compaction was done by means of 75 blows on each face of the sample in order to simulate heavily loaded traffic condition. In order to develop the required strength, all CBEM samples were cured as per Jenkins protocol [27] comprising of keeping specimens in mold for 24 h at normal laboratory temperatures followed by 24 h at 40 °C after removing from molds which is also in accordance with curing protocol as mentioned by MS-14 (Asphalt Institute, 1989) [26].

6 Testing Program

6.1 Marshall Stability and Flow

With the exception of the conditioning technique, the Marshall testing procedure for stability and flow was the same for HMA and CBEMs. Prior to testing, HMA samples were treated to conditioning in a water bath kept at 60 °C for 30 min. CBEM samples were dry conditioned for 4 h at 20 °C after curing period in accordance with MS-14 (Asphalt Institute, 1989) [26].

6.2 Indirect Tensile Strength (ITS)

Using the ITS test, asphaltic mixtures' resistance to tensile fracture failure is assessed. The ITS test involves applying compressive load to Marshall specimens by means of two strips inserted into and out of the specimen's diameter.

ITS value was calculated by using Eq. (1) as given below:

$$ITS = \frac{2P}{\pi DT} \tag{1}$$

where *ITS* = indirect tensile strength (kPa), P = max. load (N), *T* = height of sample (mm), and *D* = sample diameter (mm).

6.3 Resilient Modulus

Samples are exposed to multiple load pulses, followed by a rest interval, along their vertical diameter, using two loading strips that are each 12.5 mm wide. The test's loading time is controlled and half-sine wave loading is used. This test is conducted at 20 °C in accordance with IRC 37 [28] for both HMA and CBEMs using Pavetest HYD 16 testing device.

6.4 Statistical Analysis

A statistical analysis was performed to prove the feasibility and validity of replacing virgin materials with waste glass in terms of mechanical properties. The level of significance and coefficient of determination (statistical parameters) were calculated for the test results to infer the validity of test outcomes.

7 Results and Discussion

7.1 Marshall Test Results

All mixes underwent a test for Marshall stability and flow, and the results are displayed in Figs. 6 and 7. A gradual increase up to 100% WG content was seen after a modest fall in stability value at 20% WG content. Beyond 20% WG dosage, increase in Marshall stability is witnessed attributed to enhanced interlocking of sharp-edged waste glass particles. The Marshall flow value abruptly increased to 7.6 mm with the addition of 20% waste glass, and then gradually decreased with the addition of more WG.

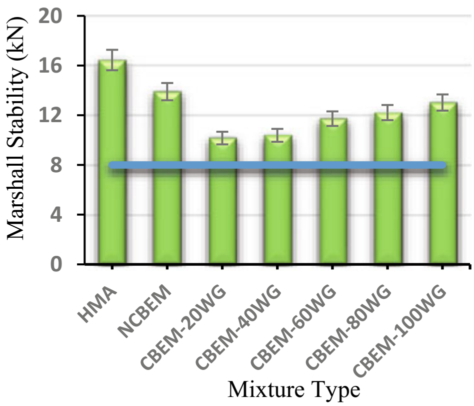


Fig. 6 Stability variation with WG content

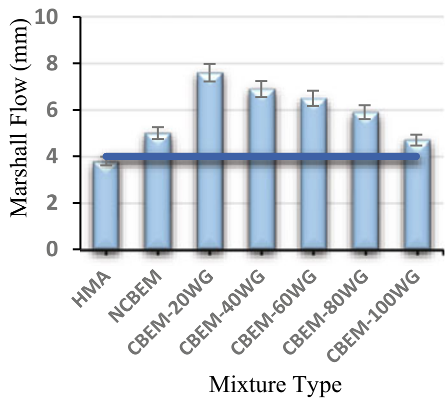


Fig. 7 Flow variation with WG content

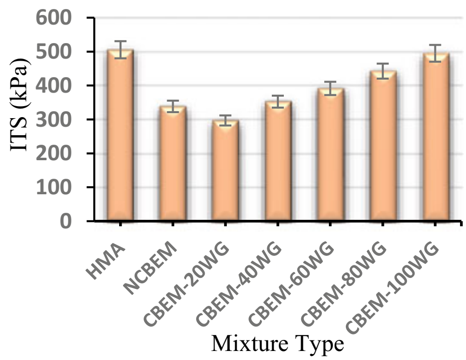


Fig. 8 ITS variation with WG content

7.2 ITS Test Results

When WG was added to CBEM, ITS decreased at a dosage of 20% but then increased beyond that, probably because to the high angularity of the glass particle, which had a notable impact on the mixture's breaking resistance. When compared to HMA with ITS value of 505.5 kPa, CBEM-100WG had an ITS value of 495 kPa, making former and the latter similar as observed from Fig. 8.

7.3 Resilient Modulus Test Results

Figure 9 shows that the resilient modulus increases as the amount of waste glass material does. And for all WG concentrations, resilient modulus increased by around 500–700 MPa after curing time. This might be as a result of strong interlocking between aggregates and glass particles with sharp edges.

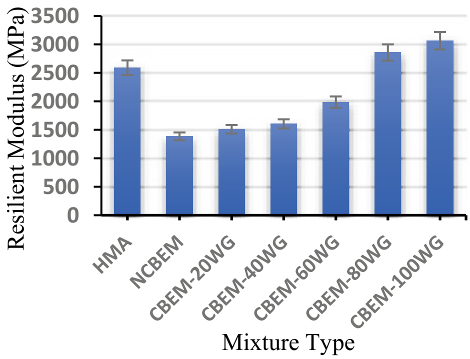


Fig. 9 Resilient modulus with WG content

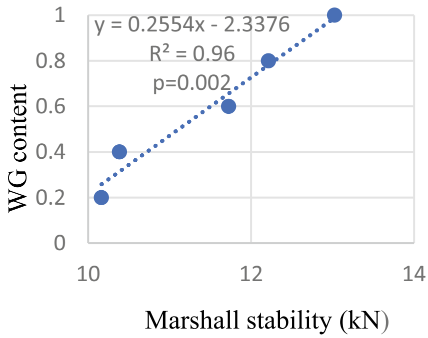


Fig. 10 Regression for stability

7.4 *Statistical Analysis*

7.4.1 Effect on Marshall Stability/Flow Measure

Regression study of the Marshall stability, as seen in Fig. 10, demonstrates that the amount of waste glass has a substantial influence on stability properties with a p-value of 0.002 < 0.05 (level of significance). Regression value for flow value results came out to be 0.003 < 0.05 as seen in Fig. 11.

7.4.2 Effect on Indirect Tensile Strength Measure

As can be seen from Fig. 12, regression analysis on ITS reveals that the amount of waste glass produced a p-value of 0.008 < 0.05, showing that the amount of waste glass has a significant influence on ITS.

Fig. 11 Regression for flow

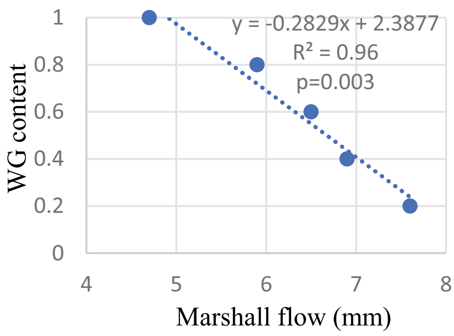
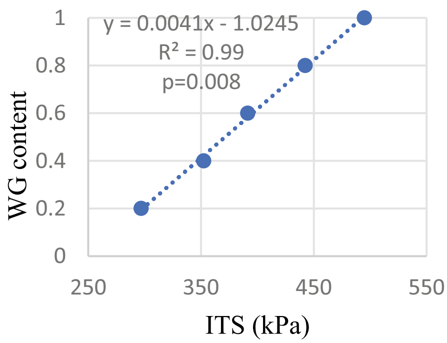


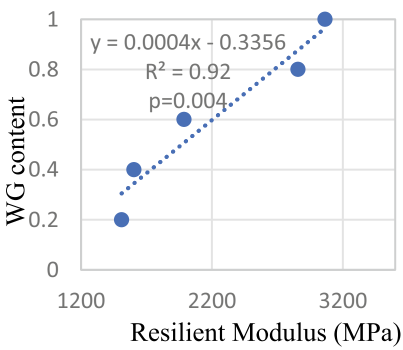
Fig. 12 Regression for ITS



7.4.3 Effect on Resilient Modulus Measure

Regression study on the resilient modulus, as seen in Fig. 13, demonstrates that the amount of waste glass had a substantial influence on the resilient modulus, as indicated by the p-value of $0.004 < 0.05$.

Fig. 13 Regression for res. modulus



It is observed from the above-mentioned regression analysis of various mechanical properties of CBEMs incorporated with waste glass (WG) that all the test results obtained were significant.

8 Conclusion

This study sought to provide a more affordable and environmentally friendly alternative to HMA by using waste glass as a replacement for fine aggregate in CBEMs. Improvements in performance metrics were proven by the examination of test data for CBEMs with waste glass (WG). The following conclusion may be drawn from performance measurement data and discussion:

i. Marshall stability was initially reduced by approximately 26% by adding 20% WG, possibly as a result of glass particles having less emulsion coating than fine aggregates. Marshall stability increased further as WG content increased, reaching a maximum value close to 13 kN, which was similar to conventional CBEM (NCBEM) but somewhat less than that of HMA. This rise might be explained by the bitumen-induced increase in the interlocking of sharp-edged glass particles with aggregates. This infers that if all of the fine aggregates in CBEM are replaced by waste glass, it behaves nearly as HMA.
ii. The Marshall flow increases initially at a dose of 20% WG but shows decreasing trend thereafter, maybe as a result of increasing particle interlocking. At 100% WG, a flow value of 4.7 mm was observed and was discovered to be lower than that of NCBEM. It can be inferred that all of the fine aggregates can be replaced by waste glass in order to have a superior lateral stability over normal CBEM.
iii. At 20% WG dose, ITS value reduced marginally, but it then gradually increased when more WG was added, reaching a maximum value of 495 kPa for 100% WG, which is remarkably similar to that of HMA (505 kPa). This increase in ITS may be due to better interlocking between aggregates as a result of glass particles with sharp edges. Replacing fine aggregates in CBEM fully by waste glass results in improved cracking resistance which is nearly similar to that of HMA.
iv. The resilient modulus (ITSM) improved as the WG content increased. CBEM with 100% WG exhibited a resilient modulus of 3065 MPa after 2 days of curing, which was 18.34% higher than that of conventional HMA. It can be implied that CBEM-100WG mix can have higher tensile stiffness in comparison to HMA.

We can conclude that the optimum recommended dosage for waste glass substitution in place of fine aggregates is 100%. Whole of the fine aggregates can be substituted by waste glass (WG) for maximum improved performance of CBEM in comparison to traditional HMA and normal CBEM. In-depth study of CBEMs incorporating waste glass as fine aggregates needs to be conducted with regard to volumetric, mechanical, and durability measures before suggesting it for large-scale industrial use.

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Effect of Warm Mix Additive, RAP, and Waste Oil on Rheological Properties of Binder

Mayank Mehrotra, Abhishek Mittal, and Dipak Rathva

Abstract Warm mix asphalt (WMA) has grown in popularity in recent times due to the energy benefits and environmental advantages. In this work, a wax-based additive called Sasobit Redux, which was induced with synthetic RAP binder and industrial waste oil, was used to lower viscosity, and hence mixing and compaction temperature during which asphalt blends are prepared in this investigation, VG30-grade bitumen binder is employed. The goal of this study is to assess the impact of a warm mix additive (Sasobit redux) incorporated with a higher percentage of rejuvenated artificial synthesis RAP binder and industrial waste oil (IWO). Four types of blends were prepared with one viscosity grade of virgin binders (VG30) induced with RAP (70%), WMA Sasobit Redux (3% and 4%), and waste oil, 41.14% dosage. It was found that adding rejuvenator to the blend has a detrimental influence on the rutting qualities, while the WMA addition improves the same, but the test results show that using 4% of WMA (Sasobit Redux) additive is far more optimal.

Keywords Reclaimed asphalt pavement (RAP) · Waste oil · Warm mix asphalt (WMA)

1 Introduction

Incorporating recycled asphalt pavement (RAP) in pavement building has lowered project costs in recent years, while also helping to conserve naturally occurring aggregates [4]. To include RAP into hot mix asphalt or warm mix asphalt, it is vital to comprehend the quantity and quality of recycled binder and its rheological properties. However, the amount of RAP to be utilized in the recycled asphalt mixture

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is normal, as more RAP may have a negative impact on low-temperature cracking resistance, fatigue resistance, blending efficiency, and other properties. Furthermore, high construction temperatures should be considered in the hot recycling process, which may result in the aging of virgin binder and secondary aging of RAP. As a result, to overcome the adverse effect of a high quantity of RAP, warm mix additive, and industrial oil comes into the picture. Numerous studies have demonstrated that combining warm mix asphalt (WMA) with high RAP consumption can result in a win–win situation that decreases the production temperature while also permitting effective RAP utilization in road building, hence contributing to lower green gas emissions and energy costs [5]. Using the WMA additive, Sasobit, 75% RAP was effectively added to recycled asphalt mixture at lower temperatures. Other researchers increased the number to 30–70% [7, 9]. The performance of different blends and mixtures having a high amount of RAP has also piqued the curiosity of researchers [6], the laboratory performance of recycled asphalt binders with a high proportion of artificial RAP binder (30–70%), and two types of WMA additives was examined.

Nonetheless, rutting and fatigue potential may remain a major worry of WMA mixtures including a high proportion of RAP, since relevant research indicates that WMA additives and a high RAP concentration contribute to inferior rutting and fatigue resistance. Current research mostly focuses on the qualities of WMA binder induced. There is a high amount of aged RAP and industrial waste oil, and little research has been done to investigate asphalt including both WMA additives and RAP binder. As previously investigated [3], the performance of asphalt is critical in determining the performance of asphalt mixtures, with rutting and fatigue resistance remaining the crucial focus of WMA combinations with high RAP. Thus, comprehensive experiments to investigate the rutting and fatigue performance of WMA mastic with high RAP binder concentration are required for improved design of matching mixtures.

2 Objectives and Scope

Examining the effect of industrial waste oils on the rheological properties (viscosity) of the binder with high RAP content induced with and without warm mix additive The rutting performance of an asphalt binder mix including a high proportion (70%) of artificially regenerated RAP binder and two different dosages of WMA additives is evaluated in this article. The artificial RAP binder contributes for 70% of the entire weight of the recycled asphalt binder, with the Sasobit Redux accounting for the remaining 3–4%. To investigate the combined effect of artificially rejuvenated RAP binder and WMA additives on binder aging qualities. All laboratory testing were carried out using a dynamic shear rheometer (DSR).

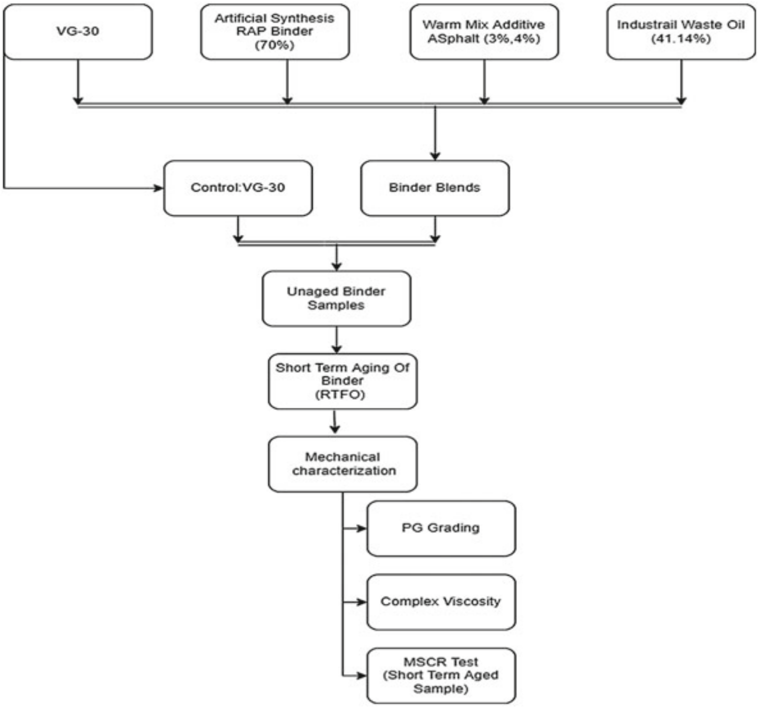

Fig. 1 Experimental plan for the testing

3 Laboratory Experiment

3.1 *Methodology*

See Fig. 1.

3.2 *Materials*

In [2] this investigation, virgin asphalt binder VG30 was used, and artificial RAP binder was produced. The method of oven aging was adopted and the procedure (time and temperature of aging in the oven) has been adopted based on a variety of literatures. The artificial synthesis RAP binder was chosen as an appropriate approach for this investigation [2], with the benefit of a pure source, in order to examine the true effect of WMA additions and industrial waste oil (IWO) as shown in Fig. 2 and ensure the results will not be affected by other variables. According to an earlier study [10], the artificial RAP binder may also display equivalent performance to the actual

RAP binder in rutting, fatigue, low-temperature cracking resistance, and viscoelastic property. Table 1 lists the features of natural and synthetic RAP binders. Two doses of WMA additives were combined with RAP and industrial waste oil. Sasobit Redux is a wax type as shown in Fig. 3, with quantities of 3% and 4% by the total mass of virgin binder indicated by the provider. Table 2 shows the fundamental physical features of industrial oil, while Table 3 shows the properties of Sasobit Redux given by the manufacturer.

There were two phases to the preparation mix. First, the virgin asphalt binder induced with artificial synthesis RAP binder (70% by the total weight of asphalt blend) was blended at 135 °C and mixed for 45 min with the high shear mixer at

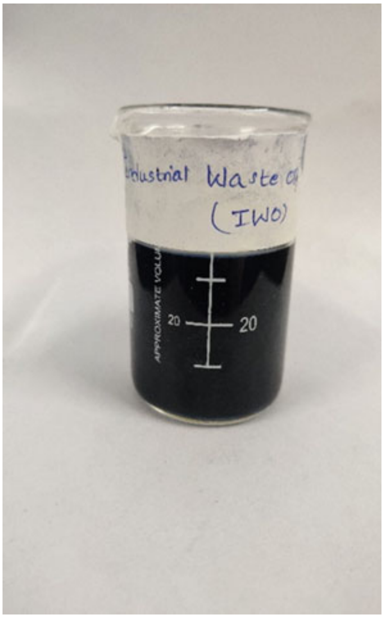

Fig. 2 The industrial waste oil (IWO)

Table 1 Properties of virgin and artificial RAP binders

S. No	Test	VG-30	Artificial synthesis RAP	Test method
1	Penetration at 25 °C, 0.1 mm, 100 g, 5	52	15	IS: 1203
2	Absolute viscosity test at 60 °C (Poises)	2954	–	IS 1206 (Part 2)
3	Viscosity at 135 °C (cPoise)	400	850	AASHTO T316
4	Softening point (°C)	52.8	81	IS: 1205
5	G/sin δ @ 1 (kPa)	67.7	101	AASHTO M 320
6	PG category	64-X	100-X	AASHTO M 320

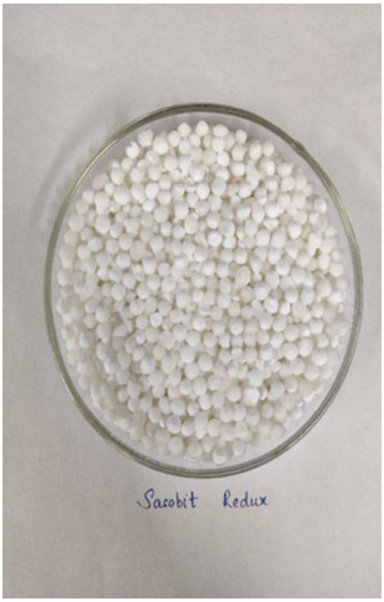

Fig. 3 Sasobit Redux additive

Table 2 Properties of industrial waste oil

Rejuvenator	Viscosity at 60 °C (Pa-s)	Specific gravity	Source	Type
Industrial waste oil	0.03356	0.994	Petroleum-based	Waste

Table 3 Properties of warm mix additive

Warm mix additive	Type	Congealing point [°C]	Penetration (25 °C) [d_{mm}]
Sasobit Redux	Wax	72–83	16–30

1850 RPM to obtain the four-blend containing warm mix asphalt binder induced with a higher percentage of artificial RAP binder and industrial waste oil (IWO). Meanwhile, two control binders were prepared: recycled asphalt binder comprising 70% synthetic RAP binder without WMA additions and virgin asphalt binder, then these asphalt binders were subjected to RTFO for short-term aging according to reference of AASHTO T 240 and ASTM D 2872.

Furthermore, the rutting performance of blends with a high proportion of artificially synthesized RAP binder was investigated. It is worth noting that WMA mixtures, including RAP, will be created at a lower temperature during the field-building process than HMA mixtures having the same RAP component [8]. However, one goal of this research is to look at the combined effect of artificial RAP binder and WMA additives on asphalt rutting ability.

Table 4 Description of the dosage and nomenclature used in this study for blends

Binder blend	% RAP binder/total binder	Industrial waste oil %	(Sasobit Redux) %
VG3	–	–	–
VG370	70%	–	–
VG370R	70%	41.14	–
VG370RW3	70%	41.14	3
VG370RW4	70%	41.14	4

To establish the proportion of rejuvenator oil in the mix, a viscosity-based semi-logarithmic blending chart provided by Asphalt Institute Manual Series MS-20 (1986) was employed. To enhance the quantity of RAP binder to total binder ratios, rejuvenator was doped to RAP binder at doses of 41.14%, as indicated by blending charts.

The blending chart was employed, which shows that mixing RAP and base binder (VG-30) in the proper proportion, i.e., 41.14% by weight of the blend, results in a viscosity similar to that of the target binder, which was 4000 Pa-s (VG40). The RAP binder was mixed with the base binder on a hot plate equipped with a shear stirrer. The blending operation was completed in 45 min at a temperature of 135 °C and a shearing rate of 1850 RPM. All the blends investigated in the study were prepared under the same circumstances.

The blending chart was employed. Using this method, four binder mixes were produced in total. V30 was assigned to the virgin binder. The binder blends were designated as V370 (blend in which 70% RAP binder was added to virgin binder without oil), V370R2 (blend in which 41.14% rejuvenator was added to 70% RAP binder and 30% Base Binder (VG-30), while Table 4 provides a detailed description of the dosage and nomenclature used in this study.

3.3 Experimental Procedure

3.3.1 Complex Viscosity

A DSR was used to determine the viscosity of the mixtures. A 25 mm geometry with a 1 mm test gap was employed, and a total of 20 data points were used to determine the viscosity of the binder using the following standard conditions according to "AUSTROADS TEST METHOD AGPT/T192" at 60 °C, one rad/s oscillating rate, and 0.1 strain amplitude. The average of twenty (20) constant viscosity values was derived from twenty (20) measurement points. The complex viscosity of all four blends is estimated using DSR in order to see the influence on the blend's viscosity and compare it to the control mix, VG30, as shown in the table below.

Table 5 Different traffic categories according to the AASTHO M332

Traffic level (ESAL) and load rate	Designation	Meaning	Jnr value at 3.2 kPa-1
>30 million and <20 km/h	E	Extremely high traffic loading	0.0–0.5
>30 million or <20 km/h	V	Very high traffic loading	0.5–1.0
10–30 million or 20–70 km/h	H	High traffic loading	1.0–2.0
<10 million and >70 km/h	S	Standard traffic loading	2.0–4.0

3.3.2 PG Grading

Superpave high-temperature performance grade test: The rutting factor (G*/sinδ) generated from the DSR test is frequently used to evaluate asphalt binder rutting resistance and mix grading. In this research, the G*/sinδ values of unaged and short-term aged binders were tested using a 25 mm parallel plate with a 1 mm gap from 64 c to the failure temperature (at 6 °C intervals) following the Superpave recommendations (SHRP, 1987). According to AASHTO, the value G*/sinδ should be more than 1 kPa and 2.2 kPa for original and RTFO-aged binders, respectively, to solve rutting. Table 5 displays the PG grade of each mix.

3.3.3 Multiple Stress Creep Recovery (MSCR) Test

The MSCR test is a well-known creep-recovery method for determining the binder's resistance to permanent deformation. This test technique involves applying 1 s of creep loading followed by 9 s of rest for 20 cycles at two distinct stress levels. The non-recoverable creep compliance J_{nr} is computed based on the permanent deformation at the conclusion of the final recovery time, according to AASHTO T 350–14 and ASTM D7405-15 rules. A higher J_{nr} value implies increased binder deformation, which is connected to greater rutting susceptibility. Based on the J_{nr} at 3.2 kPa-1 result, a binder may be selected for the various traffic categories, with "S" indicating standard traffic, followed by "H", "V", and lastly "E" representing rising traffic volumes, according to the specification AASHTO M 332 shown in Table 5.

In the current investigation, the MSCR test was done at 60C using DSR equipment. The objective for doing the test at 60 °C is to compare the performance of all binders at the same temperature. Figure 5 displays the J_{nr} values at a stress level of 3.2 kPa during the creep phase for the various rejuvenated binders and the aged control binder. Average percentage recovery, average non-recoverable creep compliance, and percentage difference of non-recoverable creep compliance are calculated using the below equations:

$$J_{nr}(\tau) = \frac{\sum_{i=1}^{10} \left(\frac{e10}{\tau}\right)_i}{10} \tag{1}$$

$$\%J_{nr\_diff} = \left(\frac{(J_{nr}(3.2kPa) - J_{nr}(0.1kPa))}{J_{nr}(0.1kPa)} \right) \times 100 \leq 75\% \tag{2}$$

$$\%R(\tau) = \frac{\sum_{i=1}^{10} \left(\frac{e1-e10}{e1} \right) i}{10} \tag{3}$$

where R (τ) = average percent recovery for 10 loading and recovery cycles at the stress level of (τ); e_1 = induced stain at the end of each loading cycle; e_{10} = residual strain at the end of the corresponding recovery period; J_{nr} (τ) = average non-recoverable creep compliance at the stress level of (τ); and (τ) = the stress level, set to 0.1 and 3.2 kPa.

4 Results

4.1 Complex Viscosity

According to Fig. 4, the addition of the rejuvenator oil drastically reduces the viscosity in the case of V370R2 as per the target viscosity of 4000 PA-s, but after the addition of the Sasobit Redux, no significant change was observed except for the sample "V370W4", which could be due to improper blending and settling of hard binder. Furthermore, inadequate miscibility of RAP, virgin binder grade, and rejuvenator may contribute to the low viscosity of the resultant blends. This idea, however, can be confirmed by investigating the morphology of the binder blends, which is outside the scope of this study.

4.2 Superpave PG Grading

Grading of blended with artificial synthesis RAP induced with and without industrial waste oil (IWO) and warm mix additive is provided in Table 6, and this demonstrates that the high-failure temperature of blend samples rises with the addition of both RAP binders, showing that the blend stiffens. Grading of blend with rejuvenated RAP and (3% and 4%) Sasobit Redux shows the effect of softening the binder and thus reduce the grading from the 94 °C to 88 °C. However the RTFO aging of blend, grading of blends bumped by one grading interval with the addition of RAP binder in all the four blends, showed a stiffening effect of RAP binder in all blends. In the current investigation, the grade change was observed to be one PG interval. However, adding IWO in the RAP binder results in a less rigid blend than V370R2. True failure of V370W3 and V370W4 is found to be the approximately same in both cases.

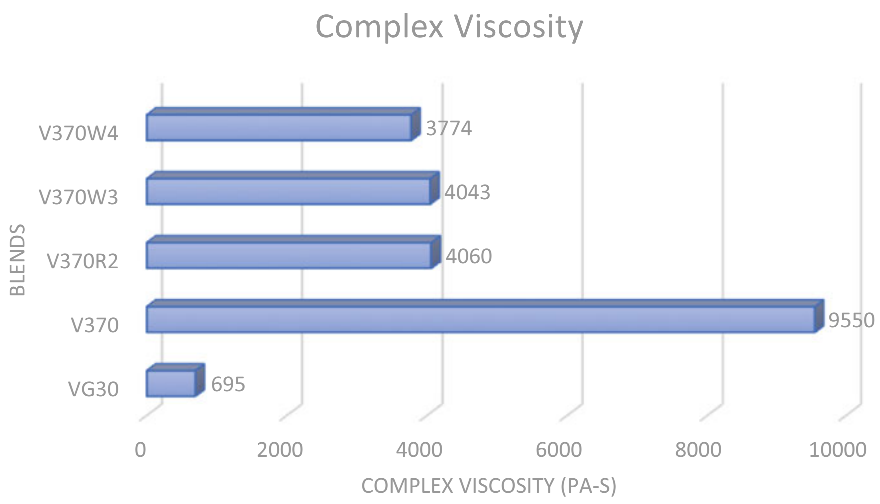

Fig. 4 Complex viscosity of the blends

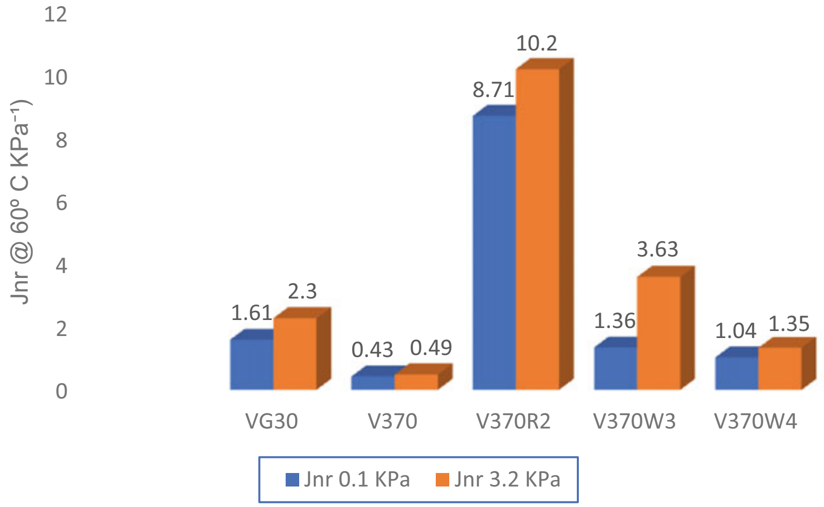

Fig. 5 MSCR test result for different binder blends

Table 6 Shows PG-grading of unaged and RFTO aged sample along with true failure temperature

Sample	Unaged	True failure temperature	RTFO	True failure
VG30	76	76.7	76	80.5
V370	94	98	100	102.7
V370R2	88	90.3	94	94.9
V370W3	88	88.6	94	94.9
V370W4	88	89	94	94.6

4.3 Analysis of J_{nr} Values

It was determined that the J_{nr} value of aged binder was 2.3 kPa. The J_{nr} value varied substantially when different doses of rejuvenator RAP and WMA additive (Sasobit Redux) were added. For the R2 rejuvenated blend (V370R2), the J_{nr} value increases, negatively influencing the rutting performance, however increasing the dose of WMA additive by 3 and 4% stabilizes the rutting performance by 3.63 and 1.35 kPa, respectively. This suggests poor rutting resistance as the rejuvenator concentration increases, although the addition of WMA improves rutting performance to some extent. AASHTO MP19 evaluates the acceptability of a specific binder under various traffic-loading conditions based on the J_{nr} value of 3.2 kPa (Table 2). The V370W4 has been recognized as "H" after demonstrating its appropriateness for a high traffic-loading scenario. The V3 base binder and V370W3 have been designated as "S" because they exhibited a lower viscosity than V370W4; consequently, the control V370W3 and V370W4 binders were graded as PG 88-X and PG 88-X, respectively. As a result, it can be inferred that the addition of Sasobit and industrial waste oil (IWO) to binder has a negative impact on the blend's grading under varied traffic-loading situations. Future investigation is required on this issue.

Whereas the Superpave asphalt binder requirements include a few techniques for investigating stress sensitivity as well as J_{nr} at various stress levels. The percentage of difference in J_{nr} is calculated using the previously stated approach, and the results are compared to 75% of the maximum limiting value. Asphalt binders with $J_{nr\ diff}$ values greater than the highest allowable value are extremely stress sensitive [1]. The $J_{nr\ diff}$ values in Table 7 show that, except for V370W3 at 58 °C, none of the asphalt binders surpassed the maximum value at high pavement temperatures. As a result, they cannot be classified as very sensitive to stress.

Table 7 Values and percentage $J_{nr\ diff}$ [$J_{nr\ diff\ (\%)}$] of the test binders

Sample	J_{nr} 0.1 kPa	J_{nr} 3.2 kPa	%R at 0.1 kPa	%R at 3.2 kPa	J_{nr}% diff	Designation	Traffic level (ESAL) and load rate
VG30	1.61	2.3	9.28E+00	11.3267	42.85	S	<10 million and >70 km/h
V370	0.43	0.49	78.54	74.15	13.95	E	>30 million and <20 km/h
V370R2	8.71	10.2	49.71	43.5	17.11	–	–
V370W3	1.36	3.63	68.93	69.64	166.9	S	<10 million and >70 km/h
V370W4	1.04	1.35	76.85	70.42	29.8	H	10–30 million or 20–70 km/h

5 Conclusion

The purpose of this research was to find out if a warm mix ingredient (Sasobit Redux) combined with a high dosage of rejuvenated RAP might reduce the stiffness of laboratory manufactured blend components without affecting the blend's performance. The researchers also investigated if rejuvenators may help hardened binder from RAP mixes with virgin binder. The following conclusions were obtained based on the data and data analysis:

1. The viscosity of the virgin binder was reduced by the addition of rejuvenators to the high dosage of RAP produced with the virgin binder both with and without Sasobit Redux. This demonstrated the blend's softening impact of the rejuvenator and WMA.
2. The MSCR test demonstrated that the rejuvenators' inclusion enhanced the virgin binder's non-recoverable creep compliance at the two tested stress levels. Again, this indicated that it had a softening effect on the blend despite adding a high percentage of RAP. The addition of the rejuvenator in the blend does have a negative impact on the rutting properties, while the WMA additive improves the same. Still, from the test result, 4% of WMA (Sasobit Redux) additive is way more efficient.
3. Additionally, it was shown that blends' rutting resistance steadily improved as RAP concentration did. Nevertheless, as RAP content increased to 70%, in comparison to V370, V370R2 had the lowest failure temperature, demonstrating that the softening effect of V370W4 can continue to operate at high RAP concentrations.

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Usage of Exclusive Plastic-Based Pre-fabricated Panels in Road Construction: A Feasibility Study

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Abstract The usage of plastic in pavement construction is no new practice as it contributes to cutting down the waste plastic pile and also makes the bitumen mix better. Along those lines, the present study goes a further mile and tries to check the feasibility of constructing exclusive plastic-based pre-fabricated plastic panel pavements. Thus, making the entire construction practice even more sustainable. A 3D finite element modelling (FEM) study has been conducted to check the feasibility of the usage of exclusive polypropylene-based pre-fabricated panels for road construction. A polypropylene plastic slab laid on an elastic solid foundation was subjected to thermal and vehicular loads and then the responses are compared to those of a concrete pavement slab to check the feasibility of this practice. The FEM results showed that the stress responses of the plastic-based slab are better than the rigid pavement slab but there is a need to look into the deflection characteristics of the plastic material panels.

Keywords Finite element modelling · Pre-fabricated plastic panels · Polypropylene

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1 Introduction

Conventional pavement construction practices hold the drawbacks of the requirement of frequent maintenance activities, delay in opening to traffic, depleting natural resources like sand and aggregates, the question of sustainability in the usage of cement and bitumen [1], etc. These problems can be better addressed if present-day construction practices are replaced by modular-based pre-fabricated plastic panels as they act as easy-to-use plug-n-play type. Due to the usage of recycled plastic, this solution in turn addresses the question of sustainability in present-day construction practices.

Construction of pavement with plastic alone marks the novelty of this idea, as this construction practice doesn't require any conventional construction materials. Usage of recycled waste plastic moulded into decks and sleepers is successfully implemented in bridge constructions like Easter Dawyck Bridge in Scotland and as a plastic bike path in Zwolle, the Netherlands [2], etc. Waste plastic is either mixed with other compatibilizers or additives to make the material more robust. Polypropylene (PP), the second highly generated type of waste plastic in India with a 9.9% share as per CPCB report [3], was used in the present study. PP is easy to mould and work with because of its properties like low density, good dimensional stability, etc. [4]. The material is then made into pellets to be fed into a moulding machine to acquire the desired shape and size. The modular panels prepared can be easily arranged together as lego bricks. Finite element modelling acts as a great tool to study various kinds of engineering problems in detail. Thus, many researchers have used various finite element packages like COMSOL Multiphysics [5, 6], Ansys [7, 8], ABAQUS [9, 10], etc., to study the problems in pavement engineering. In the present study, the feasibility of usage of recycled waste plastic moulded into pre-fabricated panels to be used in the place of conventional pavements was checked using the finite element modelling (FEM) in Ansys by comparing with conventional rigid pavement slabs.

2 Finite Element Modelling

2.1 Geometry Model and Constraints

A plastic pavement panel of 2 m × 2 m in the area and 210 mm in depth was considered. The depth of the panel is determined by equating the punching shear resistance of the material to the factored punching load as given in Eq. (1).

$$(Perimeter\ of\ one\ wheel) \times (Thickness\ of\ slab) \times (Design\ shear\ strength) = (Factored\ Punching\ load) \tag{1}$$

The geometry model along with the load positions was shown in Fig. 1. The properties of the polypropylene material used in this study are presented in Table 1. Instead of using a legacy element like SOLID45, a current-technology element like SOLID187, a 10-noded tetrahedron solid element and CONTA174 was used for meshing and for contact interactions, respectively, as suggested by the Ansys manual.

From the mesh sensitivity analysis, a 10 mm meshing for the entire geometry and a 5 mm meshing for the loaded area were finalized to balance the computational

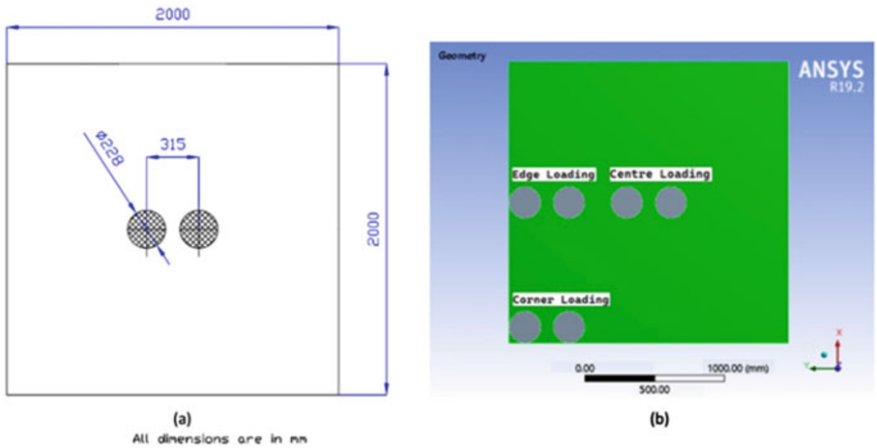

Fig. 1 Model geometry, **a** Dimensioning of the model, **b** Load positions

Table 1 Properties of polypropylene

Polypropylene (PP)			
S. No	Property	Value	Unit
1	Density	902	kgm^{-3}
2	Coefficient of thermal expansion	1.03E-04	$°C^{-1}$
3	Young’s modulus	9.15E+08	Pa
4	Poisson’s ratio	0.443	
5	Bulk modulus	2.68E+09	Pa
6	Shear modulus	3.17E+08	Pa
7	Tensile yield strength	2.62E+07	Pa
8	Tensile ultimate strength	2.99E+07	Pa
9	Compressive yield strength	4E+07	Pa
10	Isotropic thermal conductivity	0.195	$Jm^{-1}\,s^{-1}\,°C^{-1}$
11	Specific heat	1.68E+03	$Jkg^{-1}\,°C^{-1}$
12	Isotropic resistivity	1.47E+15	ohmm

Table 2 Mesh sensitivity analysis

Reduction in mesh size	% Change in deflection	% Change in stress	% Increase in CPU time
30 mm to 20 mm	0.04	0.28	+85.71
20 mm to 10 mm	0.02	0.06	+476.92
10 mm to 5 mm	0.01	0.04	+481.33

time with the accuracy of the results. The results of mesh sensitivity analysis were presented in Table 2.

The panel was assumed to be on the elastic solid foundation as it is more realistic when compared to the Winkler foundation. A foundation stiffness of 0.042 N/mm^3 which is equivalent to a 5% California Bearing Ratio (CBR) [11] was considered. And the vertical side faces of the panel were assigned with frictionless supports which restricts the motion perpendicular to the side faces [6]. To get an idea of how temperature distribution happens in polypropylene, field experimentation as depicted in Fig. 2 was conducted on the smaller interlocking polypropylene tiles having the dimensions of 17.5 * 15 * 2.5 cm (LxBXH) which results in the plan area of 21,931.94 mm^2 and the temperature of various tiles top, mid and bottom fibres were noted at the 1-h interval for a 24-h period. The positive and negative temperature gradient of the PP panel to be used in the Ansys thermal analysis was finalized as shown in Fig. 3. Though the non-linear temperature distribution gives better results, for the sake of simplicity and optimal computational time, a bi-linear temperature variation for positive gradient and a linear variation for negative gradient was considered based on the observations made from field experimentation. The mid-temperature in bi-linear variation [12] is determined using Eq. (2):

$$T_{mid} = T_{bottom} + 1/3(T_{top} - T_{bottom}) \tag{2}$$

For checking the feasibility of the usage of pre-fabricated plastic panels in road construction, the responses of the PP panel were compared with the responses of the conventional rigid pavement. The properties of concrete and the temperature profile considered in the present study are presented in Table 3 and Fig. 4, respectively. And all the other boundary and loading constraints were considered the same as in the case of the PP panel model.

2.2 Loading and Analysis

A load of single axle dual wheel configuration with a c/c wheel spacing of 315 mm and 25 kN wheel load of 228 mm diameter contact area was selected as per IRC. A triangular pulse loading of 1-s duration with a peak load of 25 kN was selected for dynamic loading. These loads were applied at various positions like the corner, edge and centre of the panel to determine the most critical load position. Standard

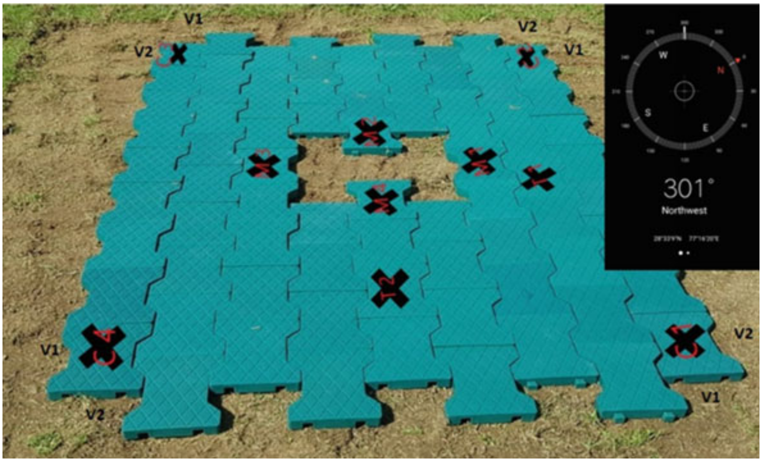

Fig. 2 Experimental setup for tile temperature distribution

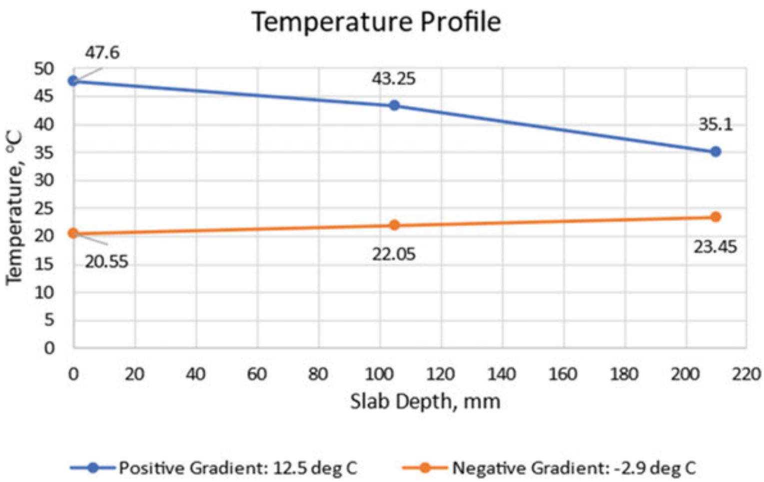

Fig. 3 Temperature distribution in polypropylene tile

Earth's gravity is also applied. Thermal gradients were applied across the depth of the panel to study the thermal stress responses. Various combinations of load like static, dynamic, thermal, combined static and thermal loading, combined dynamic and thermal loading, harmonic and transient loading types were considered in the present study. The research methodology used in the present study is depicted in Fig. 5.

Table 3 Properties of concrete

Concrete (CC)			
S. No	Property	Value	Unit
1	Density	2400	kg m^{-3}
2	Coefficient of thermal expansion	1.00E−05	$°C^{-1}$
3	Young's modulus	3.00E+10	Pa
4	Poisson's ratio	0.15	
5	Bulk modulus	1.43E+10	Pa
6	Shear modulus	1.30E+10	Pa
7	Tensile ultimate strength	3.795	MPa
8	Compressive ultimate strength	40.00	MPa
9	Thermal conductivity	0.72	$Wm^{-1}\ °C^{-1}$
10	Specific heat	780.00	$Jkg^{-1}\ °C^{-1}$

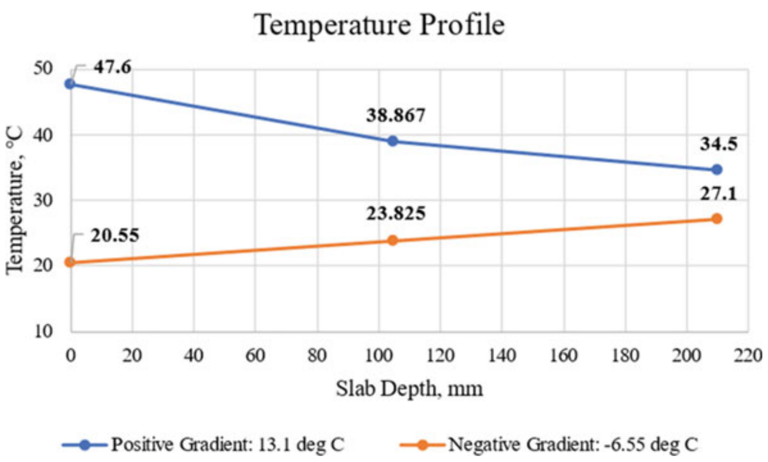

Fig. 4 Temperature distribution in concrete

Static and Transient Structural Analysis. Static and transient structural analysis can be considered as time-independent and time-dependent loads, respectively. The static structural model considered in the present study is shown in Fig. 6.

Damping and inertial effects are neglected in the case of static analysis while they're considered in the case of transient analysis. Mechanistic responses like strains, stresses, deformations, etc. can be determined. In the present study, a sequence was followed in which at first von Mises stress state was determined based on which the most critical region in the entire model will be identified and then to know the nature of stress in the critical region, maximum and minimum principal stresses were determined which gives an idea about maximum tensile and compressive stress acting in that region based on which the model is further optimized.

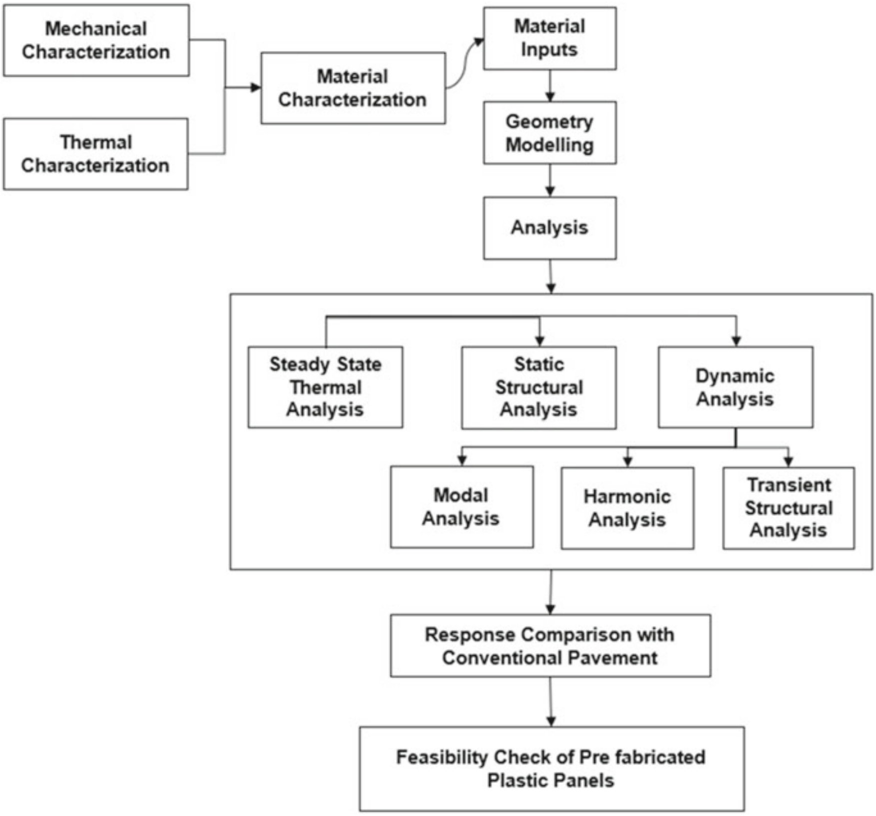

Fig. 5 Research methodology

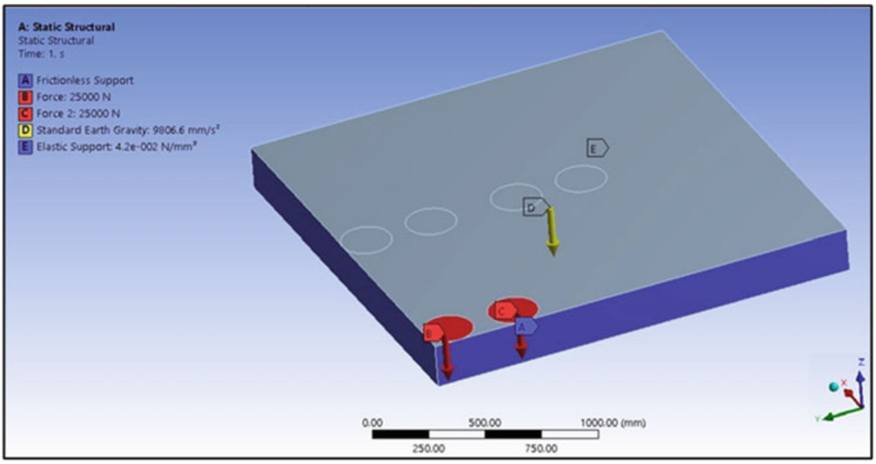

Fig. 6 Static structural model

Table 4 Modal mass summary in load direction

Mode	Frequency, Hz	Participation factor	Ratio	Effective mass	Cumulative mass fraction	Ratio [effective to total mass]
1	74.907	0.870	1	0.757	1.000	1.000
2	78.511	0.000	0	0.000	1.000	0.000
3	78.511	0.000	0	0.000	1.000	0.000
4	89.421	0.000	0	0.000	1.000	0.000
5	122.11	0.000	0	0.000	1.000	0.000
6	122.11	0.000	0	0.000	1.000	0.000
Sum				0.757		1.000

Steady-State Thermal Analysis. In this analysis, time-independent thermal loads are applied by inputting the temperature constraints. Thermal responses like thermal flow rate, flux, etc. can be determined. In order to establish the initial temperature state conditions, a steady-state thermal analysis is linked prior to any other kind of analysis.

Mode Superposition Harmonic Analysis. At first, modal analysis is carried out to determine the system's natural frequency and frequency sweep range for harmonic analysis. The modal analysis gives the mode shapes along with the corresponding natural frequencies under free vibration. Based on the modal mass summary as shown in Table 4, the most dominant mode was determined based on the ratio value which is the ratio of the participation factor of the considered mode to the participation factor of the most dominant mode.

The number of consecutive modes that combine to give a cumulative mass fraction of 80–90% is considered as prominent modes. A sinusoidal load with a frequency sweep range equal to 1–1.5 times the last prominent mode frequency is considered for the harmonic analysis. A frequency–response plot is then obtained from which critical responses were identified.

3 Results and Conclusions

From the various analysis performed, it is evident that the corner loading is the most critical loading position and the response plots for a corner loading are shown in Fig. 7.

The responses of PP and concrete slab obtained from various analyses performed in this study are curated and compared with each other as depicted in Fig. 8.

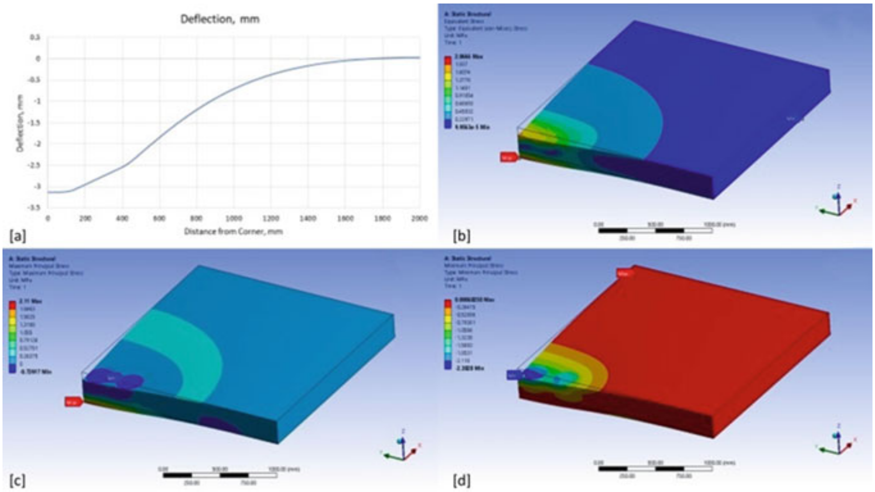

Fig. 7 Static analysis response plots, **a** Deflection plot, **b** von Mises stress plot, **c** Maximum principal stress plot, **d** Minimum principal stress plot

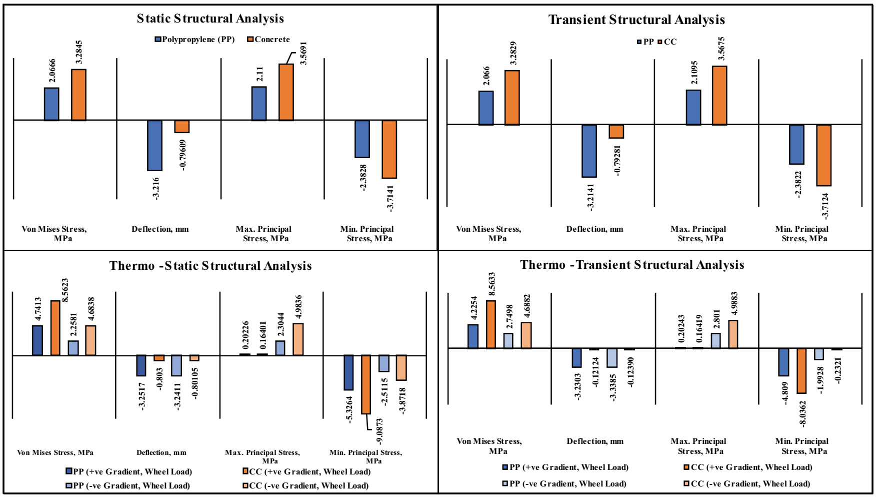

Fig. 8 Comparison of polypropylene and concrete slab responses

3.1 Conclusions from the Study

From the present study, the following conclusions are drawn:

- Left-over margin in yield/fracture is high in the case of PP slab which is 81.9% and 88.15%.

- Maximum tensile stress was observed during vehicular loads combined with a negative temperature gradient and all other responses were high when a positive thermal gradient was considered.
- The stress responses in the concrete slab are too high in comparison to the PP slab. An increment of 58.93%, 69.15%, and 55.87% in von Mises, maximum tensile and compressive stress, respectively, was observed.
- However, the deflections under the wheel are higher in the case of the polypropylene slab due to the fact that the concrete slab has a relatively higher stiffness when compared to the PP slab. Deflection in a concrete slab is 75.25% lesser when compared with a PP slab.
- The concrete slab is better resistant to compressive stresses during nighttime. But more vulnerable to tensile stresses during nighttime.
- The PP slab has shown a significant variation in dynamic responses only when the effect of thermal gradient was considered.

3.2 Implementation Recommendations

Mechanistic responses from FEM analysis conclude that plastic can be a better alternative to conventional pavement materials. However, the design shall be optimized considering the directional deflections for serviceability criteria which can be met to an extent by designing a robust joint for efficient load transfer. Total economics involved in pavement construction using recycled waste plastic-based materials shall also be considered.

If the idea gets implemented successfully on a large scale, it meets the requirements of Make in India and Swachh Bharat Abhiyan initiatives. And usage of recycled waste plastic in the place of concrete, bitumen, natural aggregates and sand makes the entire construction practice more resource productive and also leads to the creation of many skilled jobs in the fields of plastic recycling, moulding and pavement construction.

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Evaluation of Effective Thickness of Bituminous Block Pavement Using Finite Element Approach

Vignesh Dhurai, Padmakumar Radhakrishnan, and Delvin J. Joseph

Abstract Bituminous blocks are utilized for road construction all over the world in the name of historical pavement. Bituminous block pavement has a number of advantages in terms of faster construction and maintenance of road projects. The performance of any pavement structure is generally influenced by the thickness of the layer provided, which is determined by the structural and physical properties of the materials available for construction. Only a few experimental experiments on bituminous block pavement have been conducted. No studies were reported on the effective thickness of bituminous block pavement using the finite element technique. The multiple constraints associated with conducting experimental work on bituminous blocks of different thicknesses can be reduced by employing a finite element technique. Bituminous block pavement was tested in this study by altering the thickness, which was fixed according to the code specification. The bituminous block pavement thicknesses employed in this investigation were 50 mm, 55 mm, 60 mm, and 65 mm. ABAQUS software was used to perform a finite element study of the bituminous block pavement. The analysis showed that as pavement thickness increases, the deformation values decrease. The minimum and maximum deformation obtained is about 8–24% and 7–17%, respectively. The effective thickness of the bituminous block pavement was fixed based on the strain values obtained from the finite element approach.

Keywords Finite element analysis · ABAQUS · Block thickness · Bituminous blocks · Pavement deformation

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1 Introduction

The evolution of pavement started from modular pavement/block pavement using stone as the road surface. Historically, bituminous pavement is used for local streets in various places around the world like the modular method of construction. Bituminous blocks are used as pavement structures in places like Georgia [1]. Bituminous pavement block has many advantages compared to conventional block pavement in terms of material recycling, speedy construction and easier maintenance of road works. Although HMA block pavement has the issue of jointing cracking, it does have some advantages such as ease of construction and low temperature, mass production, simple and speedy repair, and recycling [2]. The author's view about the bituminous blocks was: they can be used for low-volume roads, used as a maintenance material for rapid repair works, and used for service maintenance works, but not for high-volume roads. Evaluation of any structural performance is mostly influenced by the structure's thickness and the material's properties for manufacturing. The performance of the pavement structure was mostly influenced by the thickness of the layer [3]. For doing the performance evaluation, tests on the thickness-varying structure have multiple constraints in various aspects.

The analytical approach using the finite element model is considered as the time-saving technique for evaluating the structural performance. FEM method using software is the best method for solving problems related to the linear and nonlinear nature of materials [4] as well it can be used to simulate any structure under various environmental conditions [5]. Hammin [6] developed a finite element model for the pavement structure of different model size geometry and also considered the viscoelastic properties of the bituminous concrete under two different loading conditions. Another study was done by using the finite element method to model the influence of localized strain on the HMA microstructure on the nonlinear response [7].

Another study used a new method for finding the cracking behaviour of the flexible pavement using the finite element method XFEM approach [8]. Various finite element softwares are used for pavement structure evaluation such as ANSYS, ABAQUS, and PLAXIS. For pavement analysis considering the effect of coir fibre as a reinforcement, ABAQUS, a commercial finite element modelling (FEM) application, has been frequently used [9]. Chunhua [10] conducted a study on the top-down cracking of bituminous pavement using finite element software ANSYS. From the analysis, the potential values of distress in pavement such as deformation are evaluated at various locations.

Finite element model using ABAQUS is considered as a proven technique for pavement structure analysis such as the impact of variations in concrete block thickness on vertical stress [5]. These kinds of techniques are applied for all kinds of pavement materials like thiopave modified asphalt material [11]. Furthermore, the finite element approach can be even used for analysing the pavement structure based on the shape behaviour of the materials used [12]. The finite element approach is a stringent method for designing block pavement that takes into account the discontinuous character of block pavers. Because block pavement layers contain a significant number of extremely small pieces, modelling them by finite element for structural analysis is problematic, especially when utilizing the different laying patterns [13].

From the extensive literature review, no study was reported on the effective thickness on bituminous block pavement analysis using finite element approach. The motivation of this study is to find out the effect of thickness of bituminous block pavement for the selection of appropriate thickness to carry out laboratory investigation. Considering various advantages related to the finite element approach for evaluation of pavement layer, for this study, ABAQUS software was used to analyse the effective thickness of bituminous block pavement. This study's broad aim is to find out the effect of bituminous block thickness in the pavement structure.

2 Methodology

2.1 Finite Element Model

Finite element modelling is considered a satisfactory approach for evaluating the performance of the pavement structure. The finite element method provides the most accurate results for all types of structural elements and also provides the best representation of pavement structure [14]. ABAQUS software was used to analyse a three-dimensional model of block pavement in this study. The pavement model is made up of layers with distinct material qualities. Figure 1 depicts three-dimensional models of bituminous block pavement. In any finite element approach, the steps involved are (i) selecting the model part geometry, (ii) assigning material property, (iii) assembling the geometry, (iv) assigning interaction between layers, (v) loading and boundary condition, (vi) meshing the model, (vii) assigning a job for model and (viii) model run and visualization.

Four different models are analysed in this study with different thicknesses for the bituminous pavement block. The plan view of the bituminous pavement block arrangement is shown in Fig. 2. The loading area and the pressure are kept constant for all four models and the test results are compared based on the deformation. Generally, the subgrade layer acts as a foundation for the pavement structure. In the model below, the subbase layer, elastic foundation-type property was provided. Material properties are assigned to each layer of the model and it is assembled for a single structure. The surface-to-surface contact-type interaction was provided between the layers and for bituminous blocks too. The default meshing layer for the model was applied as shown in Fig. 3. The boundary condition for each layer was assigned as unrestrained in the normal direction with the loading portion as shown in Fig. 4. A standard load of 0.56 MPa was applied over the bituminous block pavement. The same load was applied for all the models of different pavement thicknesses.

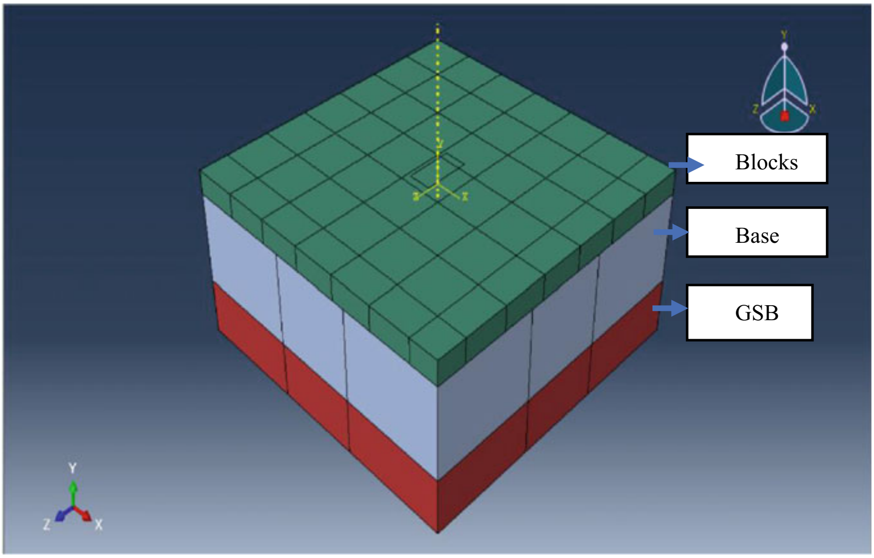

Fig. 1 Finite element model (3D view)

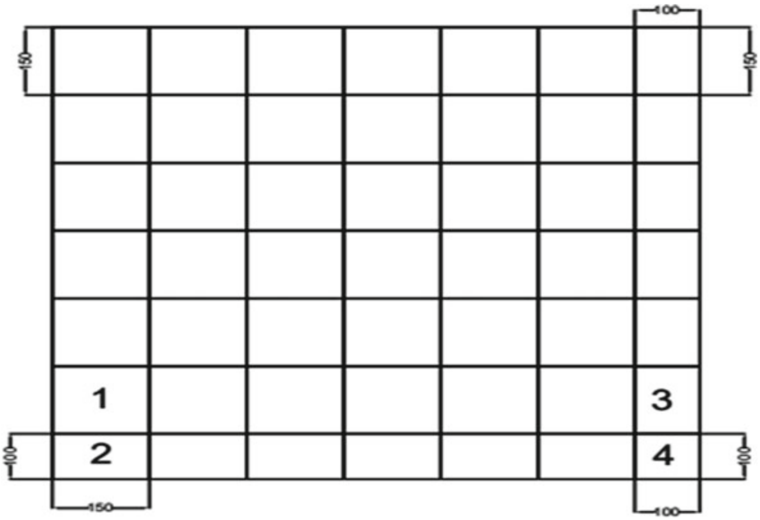

Fig. 2 Pavement block arrangement (plan)

2.2 Pavement Composition

In the finite element model, the following layers are considered such as bituminous block pavement layer, base layer and GSB (subbase) layer. The thicknesses of the

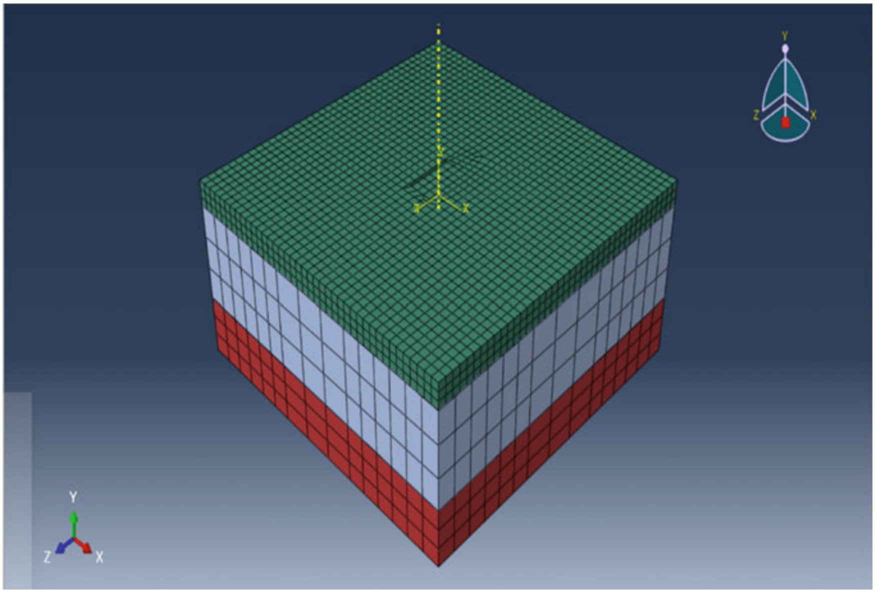

Fig. 3 Meshing

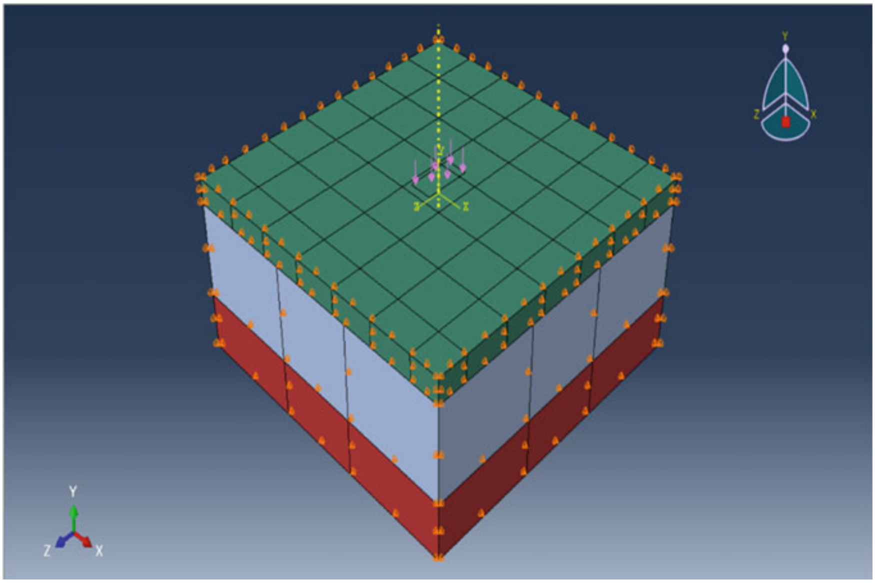

Fig. 4 Loading and boundary condition

Table 1 Pavement material properties

Pavement layer	Thickness (mm)	Modulus (MPa)	Poisson's ratio
Subbase (GSB)	150	136.92	0.35
Base	250	212.88	0.35
Bituminous block	55	1000	0.35

bituminous pavement block for this study are 50 mm, 55 mm, 60 mm and 65 mm. The thickness of pavement layers is considered as per IRC 37:2018 [15]. Initially, the CBR value of each layer is determined based on the gradation of materials as per the MORTH specifications [16]. The size of the bituminous pavement block for this study is 150 mm × 150 mm in the square. The material property of each layer and the thickness of the pavement layer are prescribed in Table 1.

3 Result and Discussion

In this present study, the effect of bituminous pavement blocks with varying thicknesses is analysed using the finite element method. The deformation and strain values are compared for different models developed using varying block thickness alone and all other layers of the pavement structure remain the same. In the case of modular pavement, the vertical compressive strain values above the subgrade values are considered. The output result for the bituminous block with different block thicknesses such as 50 mm, 55 mm, 60 mm and 65 mm as shown in Figs. 5–8, respectively. The minimum and maximum deformation value of bituminous block pavement based on the finite element approach is given in Table 2.

The test results using the finite element method indicated that as the thickness of the block layer increases the deformation of the pavement structure decreases. Minimum deformation values are obtained at the sides of the finite element model nearer to the boundary condition. More stress regions have occurred at the centre portion of the block paving and it was similar for all four types of finite element models. By comparing the minimum deformation values of block pavement, it is observed that there is an 8% increase in the values for each pavement thickness starting from 0.50 mm to 0.62 mm.

Table 2 signifies the pavement deformation values and T1, T2, T3 and T4 signify the pavement block layer thickness of 50 mm, 55 mm, 60 mm and 65 mm, respectively. The maximum deformation area for all the finite element models is falling within the region of distance 375 mm to 575 mm as shown in Fig. 9 the overall surface deformation plot of bituminous block pavement in the X-direction. Considering the same loading pattern for all the pavement models, the maximum deformation values were obtained for 50 mm block thickness. This is due to less resistance of load sustaining capacity by the T1 block pavement.

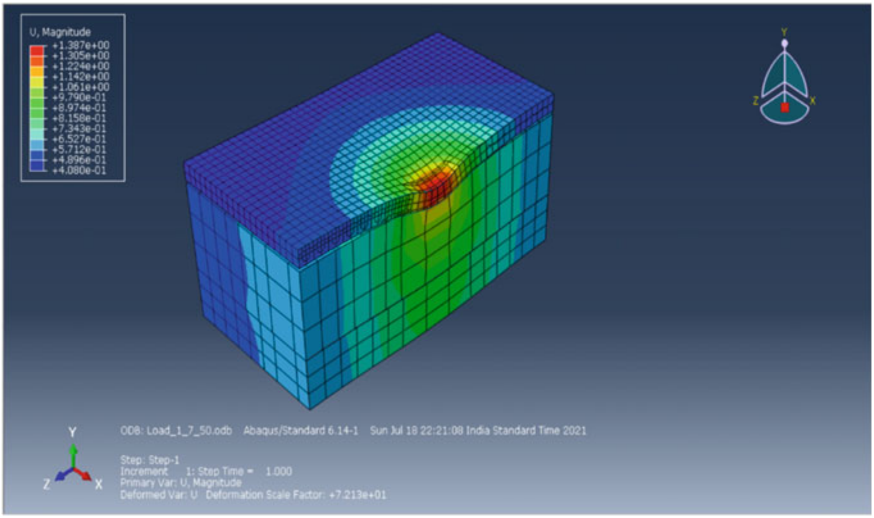

Fig. 5 Deformation of pavement block 50 mm thick

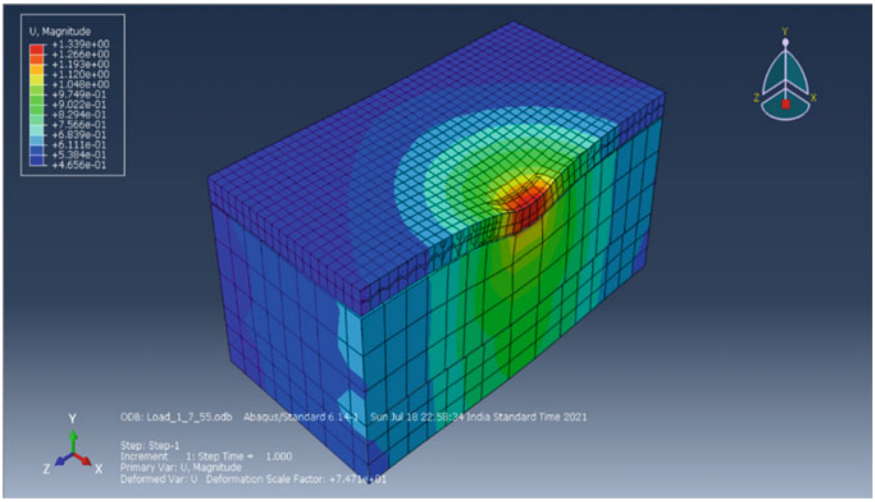

Fig. 6 Deformation of pavement block 55 mm thick

Similar to the minimum deformation values, there is a decreasing trend for the maximum deformation values of block pavement. From the analysis, it is observed that compared to the 50 mm block thickness of pavement, there is a 7% decrease in the maximum deformation values for the 55 mm block thickness pavement. Comparing the block thickness T3 and T4 finite element models, the maximum deformation values of the pavement structure vary about 10% and 17%, respectively. As per the

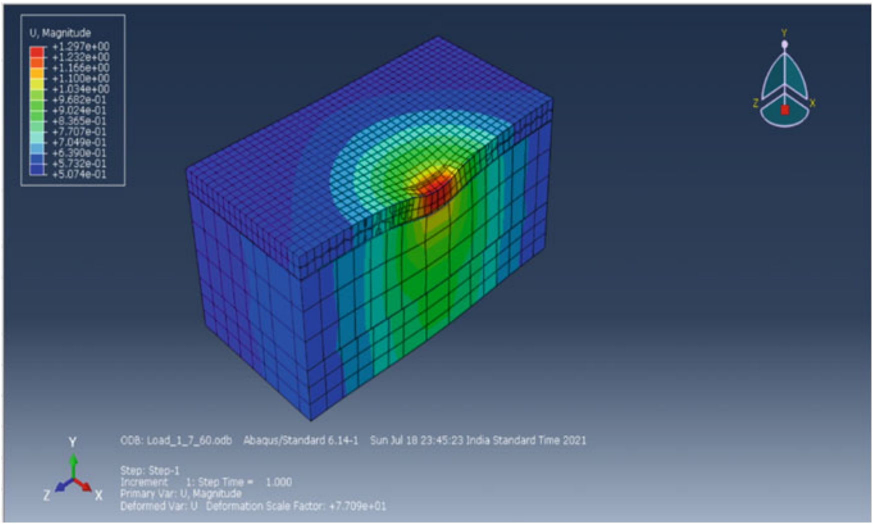

Fig. 7 Deformation of pavement block 60 mm thick

Table 2 Minimum and maximum deformation values

Deformation	T1	T2	T3	T4
Maximum (mm)	1.37	1.27	1.24	1.14
Minimum (mm)	0.50	0.54	0.58	0.62

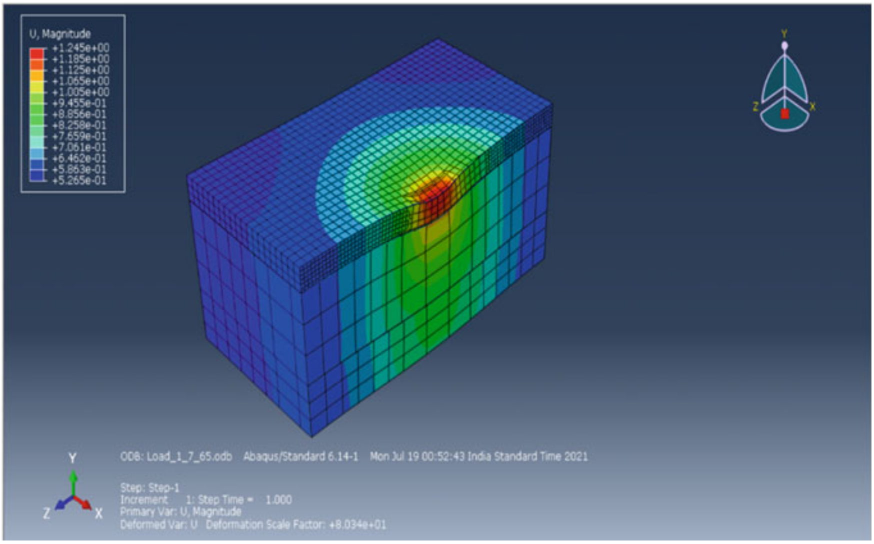

Fig. 8 Deformation of pavement block 65 mm thick

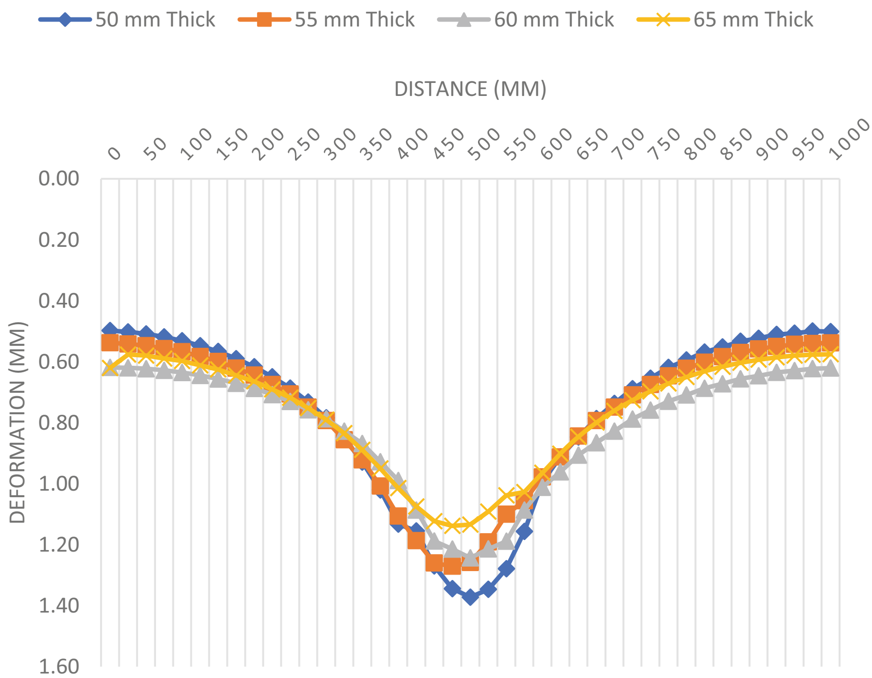

Fig. 9 Surface deformation plot of bituminous block pavement

deformation plot based on the test results from the finite element model shown in Fig. 9, the deformation plots are similar for all types of the thickness of pavement block layer.

Further to the deformation values of the pavement model, the strain values are considered an important factor in pavement evaluation. Based on the analysis, variation was observed for the stain values of the block pavement for different thicknesses. The variation in the maximum strain value of block pavement is shown in Fig. 10. Similarly, the strain value also decreases as the thickness of the block pavement increases. Maximum strain value was observed for the block T1 thickness of about 0.00176. Compared to the 50 mm thickness block model, the 55 mm thickness block model strain value decreases about 10%. For the T3 and T4 finite element model, the maximum strain value varies about approximately 19% and 30%, respectively.

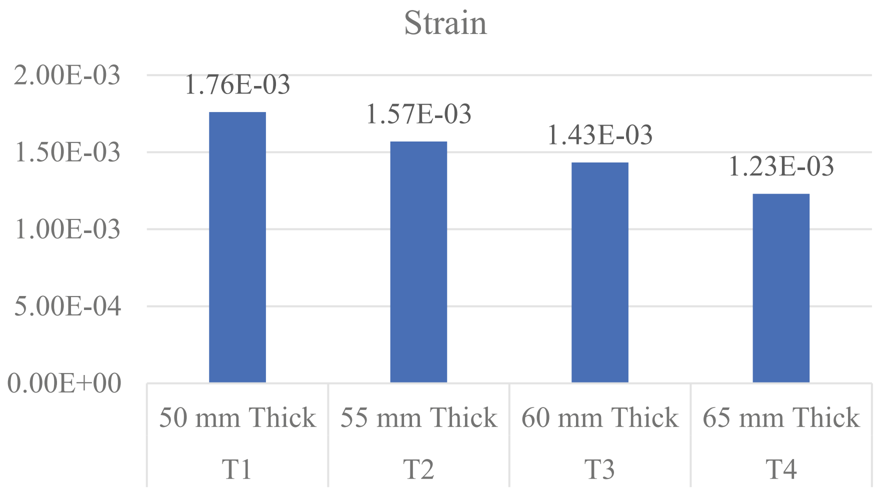

Fig. 10 Maximum strain values of block pavement

4 Conclusion

In this study, the effective thickness ranges from 50 mm to 65 mm of bituminous block pavement which was evaluated using finite element analysis. Based on the finite element model analysis, the major findings are given as follows:

- As the thickness of the pavement block increases the deformation value and strain value decrease.
- Minimum and maximum deformations were obtained for the pavement block of thickness 50 mm, which is about 0.50 mm and 1.37 mm, respectively.
- Variation in the minimum deformation values ranges from 8% to 24% compared to the least block thickness.
- Similarly, the maximum deformation values also varied about 7% to 17% from the least block thickness.
- The maximum strain value for the block thickness of 50 mm is about 0.00176 and for other block thicknesses of 55 mm to 65 mm it varies ranging from approximately 10% to 30%.

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Effect of Cement and RAP Content on Full Depth Reclamation (FDR) of Low-Volume Roads: A Response Surfaced-Based Study

Supratim Kaushik, Anugu Sairaj, Vishal Kirti, and S. Anjan Kumar

Abstract Full depth reclamation (FDR) of asphalt pavements is a sustainable and economical rehabilitation technique used worldwide. Typically, in any FDR process, recycled materials from pavement layers are mixed with a chemical additive like cement to increase the strength and stiffness of the layer. Additional material in the form of RAP and aggregates is used if the desired depth or gradation is not achieved. This study tries to assess the compressive strength characteristics of FDR by considering open graded premix carpet (OGPC) as RAP material and understand the effects of RAP and cement through statistical methods. FDR specimens were fabricated in the laboratory considering a typical low-volume road composition used in India. The response surface method was employed for the experimental design and development of a response model for FDR compressive strength. Laboratory investigation included unconfined compressive strength (UCS) determination at various test conditions as per the design of the experiment. Results showed that the compressive strength of FDR was within limits specified for a treated granular layer using nominal cement contents for the in situ gradation considered for a full depth reclamation. ANOVA analysis showed that the effects of cement and RAP were significant, whereas the interaction between cement and RAP was not significant within the range of testing. However, the effect of cement was more prominent as compared to the effect of RAP. Further, the response model developed showed good capability for predicting the UCS values within the range used to develop the model.

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Keywords Full Depth Reclamation (FDR) · Unconfined Compressive Strength (UCS) · Response surface

1 Introduction

India uses an estimated 15,000 tons of aggregate to construct 1 km of road. Such an enormous demand for natural resources leads to massive energy consumption through burning fossil fuels resulting in air pollution and greenhouse gas emissions. Therefore, tremendous focus is on sustainable construction of roads. A viable alternative to using natural virgin aggregates is recycling distressed pavements. One such technique is the full depth reclamation (FDR) which can be defined as "a pavement rehabilitation and upgradation technique where in bituminous and pavement layers of predetermined thickness are excavated, pulverized and blended with a binder and compacted to act as a bound or hardened base material of the new pavement." The reclaimed material is usually blended with a stabilizer like lime, cement, or fly ash to provide an upgraded homogenous material. Additional material in the form of RAP and aggregates is used if the desired depth or gradation is not achieved. However, in situ recycling with the in-service materials without the addition of supplementary materials is desirable to avoid delays and reduce project cost. In this regard, the objective of this study was to assess the compressive strength characteristics of FDR by considering OGPC as RAP material for a typical low-volume road composition in India. In addition, the effects of RAP and cement on the compressive strength characteristics of FDR were evaluated using statistical methods.

2 Background

2.1 Full Depth Reclamation (FDR)

The first use of Full Depth Reclamation (FDR) dates back to the 1950s, which was first used by pavement engineers in the USA and France [1]. Gradual development over the years has seen FDR adopted as a sustainable rehabilitation technique in many parts of the world. The first step in an FDR construction process is pulverization of the existing distressed pavement layers using a road reclaimer. The additive together with water is blended along with the pulverized material by the reclaimer, which is then compacted using a roller. Benefits of FDR include cost-effectiveness, increased durability and structural capacity, early opening to traffic and reduced carbon footprint [2]. The most commonly used additives are cement, fly ash, lime, and bitumen emulsion [2]. A comprehensive review of mix design methods shows that the unconfined compressive strength (UCS) test is the mostly used design parameter for an FDR [1], and therefore considered in this study.

2.2 Response Surface Methodology

Response surface methodology, or RSM, is a combination of mathematical and statistical multifactorial analysis techniques. It is useful for modeling and analysis applications in which several experimental factors influence a response variable of interest. After selecting the significant factors, the multifactorial analysis is conducted by establishing a second-order polynomial model used to quantify the main effects and interaction effects of the significant factors on the response variable. A typical second-order response modeling is as follows:

$$y = \beta_0 + \sum_{i=1}^{k} \beta_i x_i + \sum_{i=1}^{k} \beta_{ii} x_i^2 + \sum \sum_{i<j} \beta_{ij} x_i x_{ij} \tag{1}$$

where y is the response variable, x_i and x_j are independent variables, β_o is a constant, β_i is the linear coefficient, β_{ii} is the quadratic coefficient, and β_{ij} is the interaction coefficient. The central composite design was used in the present study for modeling the unconfined compressive strength of FDR due to its better ability to model conducting experimental runs at 2^n axial points, which are augmented with 2n star points and n_c center points, where n is the number of variables in the study and can range between 2 and 10 and n_c represents center point experiments and can range between 2 and 6. The experimental data is then used to fit a response surface to explore possible relationships between the response and explanatory variables. The unconfined compressive strength was considered the dependent/response variable and the cement and RAP content were taken as the explanatory or independent variables for model development.

3 Materials and Methods

3.1 Laboratory Fabrication of FDR Specimens

The FDR specimens were fabricated in the laboratory by considering a typical low-volume road composition as per IRC SP:072 (2015). The pavement is composed of a granular sub-base (GSB), followed by a water-bound macadam (WBM) layer, overlain by an open graded premix carpet (OGPC) as shown in Fig. 1a. The range of RAP content was fixed based on the maximum and minimum reclamation depth possible for the pavement composition considered. A full depth reclamation up to the sub-base layer will result in a RAP content of 6% per unit area of reclaimed material, whereas a minimum depth of reclamation up to the base layer (WBM Grade III) results in a RAP content of 21% per unit area as shown in Fig. 1b. Therefore, the RAP content was varied within the range of 5–25% to simulate the field FDR process, which was within the limit of 30% suggested by most state agencies in the USA

[3]. The individual layers (WBM, GSB, and the OGPC) were batched separately as per MORD guidelines and mixed together to obtain the desired FDR gradation [4]. While varying the RAP content, the aggregate gradation was kept constant as a combination of WBM (Grade III and II) and GSB throughout the study to draw inferences regarding the effect of RAP and cement on compressive strength. RAP percentages were varied as a replacement for aggregates. The combined gradation is shown in Fig. 2.

Aggregates. Aggregates used in the study were crushed gravel obtained from a local stone crusher. Basic properties of the aggregates were tested and were found to be satisfying the MORD guidelines for road materials.

RAP. The RAP material used in the study was fabricated in the laboratory from previously tested bituminous mix specimens. The specimens were segregated into sizes of 11.2mm and 5.6mm as per MORD guidelines for OGPC, and mixed together with the aggregates as per proportions. An effective bitumen content of 5.5% was used for the bituminous mix.

Stabilizer. Portland Pozzolana cement (PPC) complying with IS 1489 (Part-II)-1991 was used for the study. The dosage of cement used varied between 2 and 6% of the aggregate weight based on previous studies carried out on FDR [1].

Modified Proctor Compaction Test. Compaction characteristics of the fabricated FDR at each test condition were determined using the modified proctor compaction tests as per IS 2720 (Part 8). No definite trend was observed in the maximum dry density (MDD) variation. However, the optimum moisture content (OMC) decreased

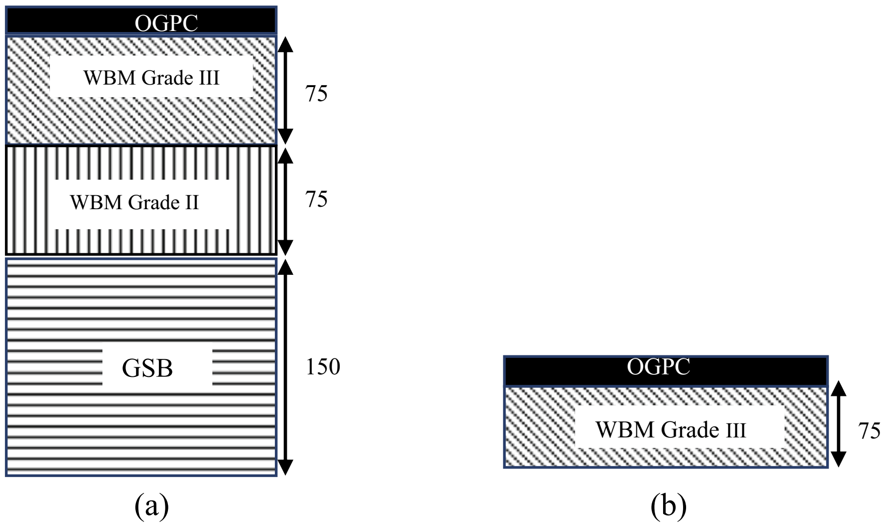

Fig. 1 Composition considered for FDR **a** RAP = 6%, **b** RAP = 21%

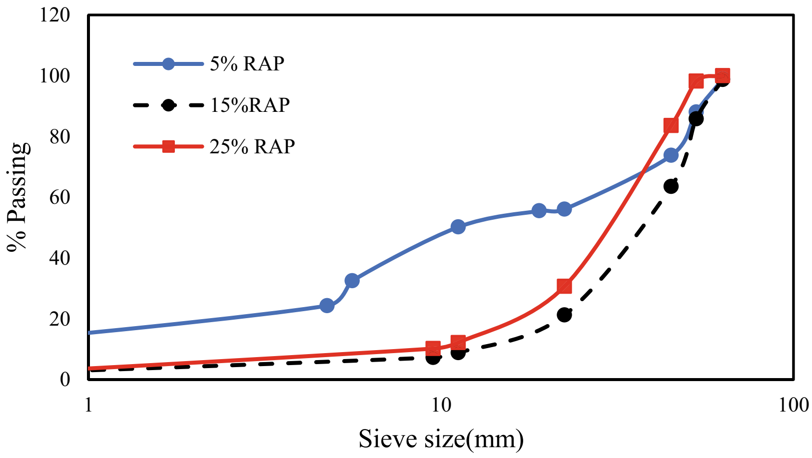

Fig. 2 Aggregate gradation used

with an increase in RAP content. In general, the OMC varied within 5.24%–6.39%, whereas a range of 2.20–2.23 g/cm^3 was obtained for various test conditions.

3.2 Specimen Preparation

Gyratory compacted cylindrical specimens of 100 mm diameter and 150 mm height were prepared by means of gyratory compaction [5]. Previous studies have shown that a good approximation of the modified proctor compaction can be obtained using the gyratory compactor by varying the gyration angle, ram pressure, and number of revolutions [5]. After various trials, a ram pressure of 900 kPa, gyration angle of 1.25°, and 250 revolutions were finalized to obtain the required density. Specimens were compacted at optimum moisture content (OMC) and maximum dry density (MDD), as determined from modified proctor tests. The specimen preparation method consisted of six steps: preparation of batched samples of FDR, addition and dry mixing of cement, addition of a predetermined quantity of water, wet mixing, compaction, extrusion, and curing. The compacted specimen is extruded, and kept for curing in a desiccator to prevent loss of moisture. The process is shown in Fig. 3a through e.

3.3 Unconfined Compression Strength Test

Unconfined compressive strength (UCS) test was conducted on FDR specimens as per IS 4332 (Part V)-1970. A strain rate of 1.25 mm/min was used for testing. The test

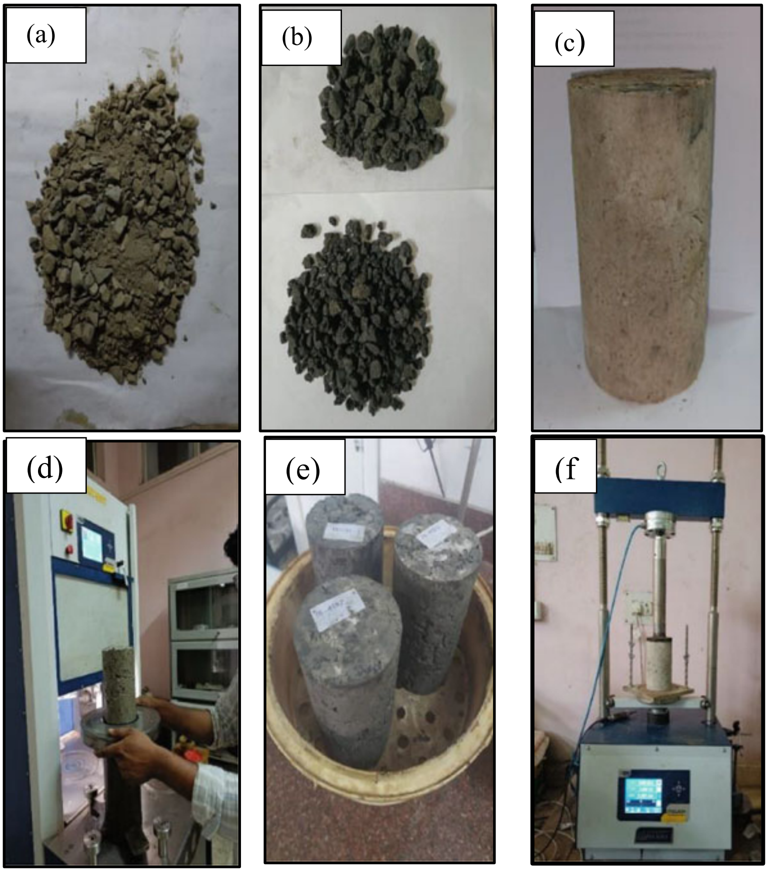

Fig. 3 **a** Aggregate used in specimen preparation, **b** Single-sized RAP material used, **c** Gyratory compaction of specimens, **d** Compacted UCS specimen, **e** Curing of specimens in the desiccator, **f** Unconfined compressive strength test in the laboratory

is shown in Fig. 3f. It should be noted that the terms UCS and compressive strength are used interchangeably in this paper. The peak load at failure and the corresponding deformation was noted and the unconfined compressive strength was determined as follows:

$$\mathrm{UCS(MPa)} = \frac{P}{A} \tag{2}$$

where

P maximum axial load, N
A corrected area of specimen at failure $= \frac{A_0}{1-\varepsilon_a}$
A_o original area of the UCS specimen, and
ε_a strain at failure.

3.4 Design of Experiments and Response Surface Method

The response surface design was performed using MINITAB statistical package considering four axial and four cube points with one rotatability ($\alpha = 1$). Two factors at three levels with three replicates were considered in this design as summarized in Table 1. A face-centered CCD with one center point and zero level of block was used in this study. To avoid potential systematic biases, experiments were conducted in a randomized manner. The significance of the model was assessed using the analysis of variance (ANOVA) and the effect of linear, quadratic, and interaction terms was evaluated. Ten extra experimental runs were made at different combinations so as to generate a dataset for model validation. The experiments were conducted at cement contents 2, 3, 4, and 5% and RAP contents of 10 and 20% for model validation.

Table 1 Experimental matrix for response surface method

Coded variables		Rap content	Cement content	No. of replicates
−1	−1	5	2	03
1	−1	25	2	03
−1	1	5	6	03
1	1	25	6	03
−1	0	5	4	03
1	0	25	4	03
0	−1	15	2	03
0	1	15	6	03
0	0	15	4	01
0	0	15	4	01
0	0	15	4	01
0	0	15	4	01
0	0	15	4	01

4 Results and Discussions

4.1 Unconfined Compressive Strength (UCS)

As the UCS values vary with specimen size and shape, the IRC SP 89–2018 suggests a correction factor of 1.25 to be used for a cylindrical specimen of 100 mm × 200 mm. The obtained UCS values were multiplied with the conversion factor and are presented in Fig. 4. The range of compressive strength for the tested material ranged from 0.425 MPa to 3.8 MPa. The IRC SP 89–2018 recommends a minimum 7 day compressive strength varying from 0.75 MPa for treated sub-base layer to 4.5 MPa for a treated base layer. The 7-day compressive strength achieved at 4 and 6% cement contents was >3.5 MPa, with the highest strength of 3.8 MPa achieved at 5% RAP content and 6% cement, which corresponds to a full depth recycling up to the GSB as per Fig. 1a. Therefore, the existing pavement can be reutilized as a sub-base layer for low-volume roads using nominal cement contents. However, for use as a base material, a slightly higher cement content is warranted. The suitable dosage of cement and RAP for the compressive strength to be within the specified limits can be selected from the contour plot shown in Fig. 5.

4.2 Effect of Cement and RAP on UCS

The main and interaction effects plots were analyzed to further understand the effects of cement and RAP on the compressive strength. The main effects plot shows an increasing trend of mean UCS values with cement, whereas a decreasing trend was observed for RAP. The interaction effects plot indicates the presence of a possible

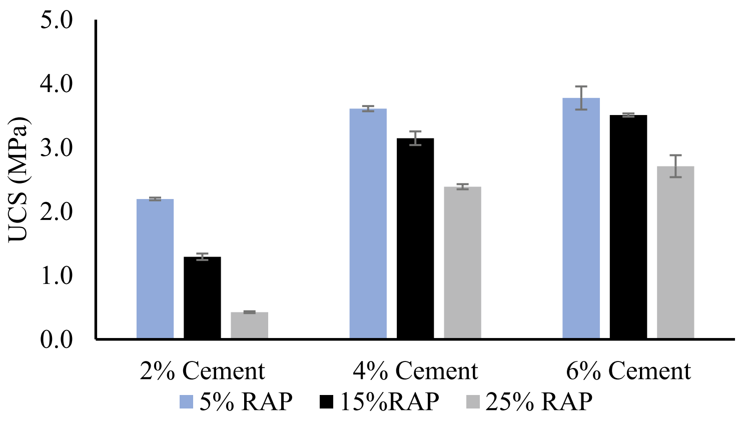

Fig. 4 Unconfined compressive strength results

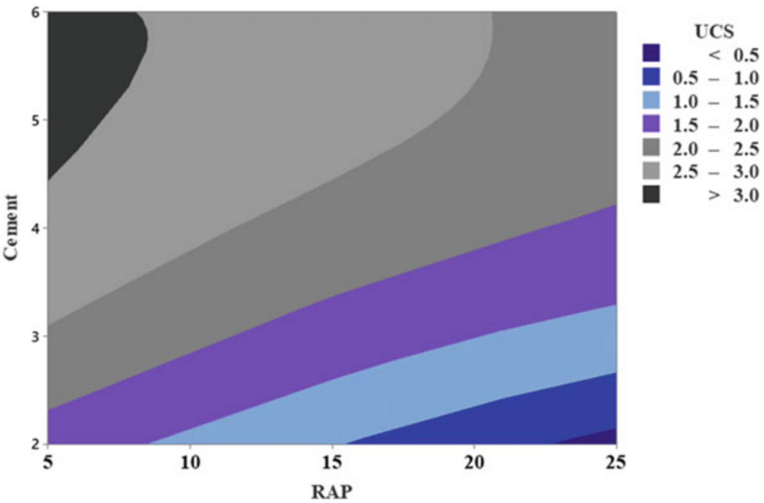

Fig. 5 Contour plot of UCS with various cement and RAP content combinations

interaction at higher and intermediate cement contents. A simple Analysis of Variance (ANOVA) test was conducted to ascertain the significance of main effects and interaction. Results showed that the main effects were significant, whereas the interaction was not significant, even though the interaction effects plot suggested the presence of a possible interaction. Therefore, the main effects plot was used to draw further inferences for the effect of cement and RAP on the compressive strength. The magnitude effect of cement was higher for 2–4% than for 4–6% increase as most of the aggregate cement bonding is completed at 4% cement content, leading to a sharp rise in mean UCS response from 2 to 4% increase. Similarly, the magnitude of the effect of RAP was more evident for an increase in RAP content from 15 to 25% than from 5 to 15%. This is because of the alteration of the aggregate gradation, as a well-graded base material is replaced by single-sized aggregates coated with bitumen, leading to lesser aggregate interlock.

4.3 Statistical Evaluation

Response Surface Model. To statistically evaluate the effect of cement and RAP content on the compressive strength of FDR, a response model was formulated using the experimental data as per Table 1. The response model is of the order of a second-order polynomial which includes square terms as well as interaction terms. The formulated model is as per Eq. 3. The R^2 of the model is 0.948, whereas the lack of fit is 0.893.

$$\begin{aligned}\text{UCS} = & -0.015 - 0.0792\ \text{RAP} + 1.277\ \text{Cement} \\ & - 0.00010\ \text{RAP}^*\text{RAP} - 0.1222^*\text{Cement}^*\text{Cement} \\ & + 0.00705\ \text{RAP}^*\text{Cement}\end{aligned} \tag{3}$$

Model Validation. The model was validated by conducting further ten runs of experiments at several cement and RAP contents. Further, eight data points from the literature were also used to validate the model [6, 7]. The aforementioned studies were selected based on the range of RAP and cement content evaluated, which was within the range used in this study. The prediction ability of the model is shown in Fig. 6. It can be observed from Fig. 6 that excellent agreement was obtained for the laboratory data. Further, the agreement obtained for the data from literature was satisfactory within the range of the RAP content and cement dosage used in this study. Therefore, the model's prediction ability within the range considered for the material used in this study was found to be adequate.

ANOVA Analysis of Statistical Model. The model was statistically significant at $\alpha = 0.05$. The model coefficients and their diagnostics are summarized in Table 2. The coefficients were significant for RAP and cement content with p-value <0.05, which implies a significant effect of RAP and cement individually on the compressive strength. However, for higher order terms and interaction terms, the coefficients were not significant. No multicollinearity issues were observed, as indicated by the lower VIF values in Table 2. Further, the standard error of the coefficients was also on the lower side. An important observation is that the quadratic effects of cement is much more pronounced as compared to RAP, indicated by a lower p-value in Table 2.

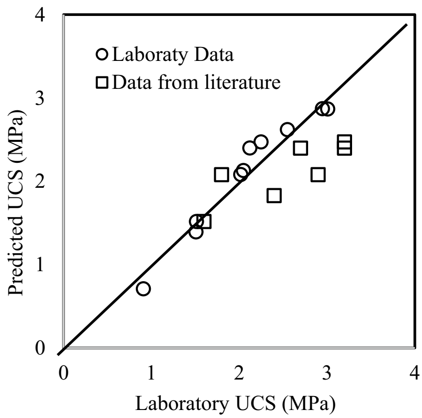

Fig. 6 Validation of response model

Table 2 ANOVA analysis of model parameters

Source	Coefficient	SE coef.	VIF	P-value	Remarks
Constant	−0.0150	0.094		0.000	S
RAP	−0.0792	0.092	1.00	0.001	S
Cement	1.2770	0.092	1.00	0.000	S
RAP^2	−0.00010	0.136	1.17	0.942	NS
$Cement^2$	−0.1222	0.136	1.17	0.009	NS
RAP x Cement	0.00705	0.113	1.00	0.252	NS

*S: Significant; NS: Not significant

5 Conclusion

This study evaluates the UCS of the FDR considering OGPC as RAP material. The following are the conclusions from the study:

- This study demonstrated that with a cement content of 4 and 6% and a full depth reclamation up to the sub-base layer, a UCS >3.5 MPa could be accomplished. Therefore, the FDR material can be utilized as a sub-base layer at nominal cement contents without addition of supplementary materials. However, a slightly cement content may be necessary for use as a base layer.
- Analysis of the main effects plot showed that the magnitude of the effect of cement on compressive strength was higher at lower cement contents (2–4%). Similarly, the magnitude of effect of RAP was greater in the range of higher RAP content (15–25%).
- The response model developed showed good capability of predicting the compressive strength within the limits of the study with $R^2 = 0.948$. ANOVA analysis results showed that the main effects of cement and RAP were significant on compressive strength. However, the interaction effects and the quadratic effects were insignificant. Further, the effect of cement was observed to be dominant over RAP, as indicated by a much lower p-value.

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High- and Intermediate-Temperature Performance of Crumb Rubber-Nano-Alumina-Modified Asphalt Binder

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Abstract The purpose of this study is to ascertain the impact of adding nano-alumina (NA) to an asphalt binder that has been modified with crumb rubber (CR). In this work, the nano-alumina content was changed from 1 to 6% while the concentration of CR was held constant at 12% (wt. of base binder) (decided after literature survey). Physical tests like penetration, softening point and ductility were used to assess the impact of different concentrations of nano-alumina (by weight of binder) at different concentrations (1, 2, 3, 4, 5 and 6%). Theological parameters of the base binder and the nano-alumina polymer-modified asphalt binder were analysed using the rotational viscosity (RV) and dynamic shear rheometer (DSR) tests. In addition, the performance of modified asphalt after thin film oven (TFO) (short-term ageing) and pressure ageing vessel (PAV) test (long-term ageing) was assessed as well. Furthermore, the storage stability of modified asphalt binder was evaluated. Results showed that the addition of nano-alumina has a positive effect on rutting performance of CR-modified asphalt binders. Storage stability of the CR-modified asphalt binders improved significantly after the addition of nano-alumina. According to the Superpave rutting metric ($G/\mathrm{Sin}\delta$), the addition of 4% of NA and 12% of CR improves the performance of the binder's rutting strength. At a particular percentage of CR and NA, a slight increase in the fatigue measure was also noted. According to a storage stability test, improved asphalt binder does not experience phase separation at high temperatures. On the basis of this, it may be deduced that when a little amount of NA was added to the CR-modified binder, its physical characteristics were successfully enhanced. During rheological characterization, it was found that complex modulus increases, phase angle decreases, Superpave rutting parameter increases and failure temperature increases with increasing nano-alumina content. It was also found that Brookfield viscosity increases with an increasing nano-alumina concentration as binder becomes stiffer. All the test results confirmed the fact that crumb rubber-nano-alumina modifier is effective in enhancing the high-temperature properties (rutting resistance) of the soft grade binders and at the same time, it increases the elasticity of the binders.

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Keywords Crumb rubber · Nano-alumina · Superpave rutting parameter · Superpave fatigue parameter · Viscosity · Ageing · Storage stability

1 Introduction

The most common and highly environmental pollutant polymeric modifier is crumb rubber. It is a crosslinking polymer produced from automotive and truck scrap tyres. As crumb rubber is very hazardous for the environment and the production of crumb rubber is in tonnes every year. By using crumb rubber in road pavements, it will increase its strength and also reduce environmental pollution. The desirable properties of polymer-modified bitumen (PMB) include improved strength, cohesiveness and resistance to fatigue and deformation (Ameri et al. 2009). PMB also possesses less temperature sensitivity than virgin bitumen. PMB suffers from storage instability and phase separation at high temperatures. So, to rectify these problems, addition of nanomaterials is needed as they have high storage stability and other properties such as high specific surface area, high conductivity and large assembly. Most commonly used nanomaterials are CNTs, nano-silica, nano-clay, nano-titanium oxide, nano-alumina, etc. Nano-structured alumina have high surface area, thermal stability, conductivity, mechanical strength, stiffness, inertness to most acids and alkalis, adsorption capacity, wear resistance, oxidation and non-toxic. Nano-alumina is one of the nanomaterials which enhances the stiffness, recovery ability, ageing resistance, fatigue resistance and provides storage stability to PMB, thus making a good choice for addition to PMB. Nanotechnology is multidisciplinary science which deals with physics, chemistry, materials science and other engineering sciences. The applications of nanotechnology are spreading in almost all the branches of science and technology [13]. Nanotechnology incorporates the research and technology established at nuclear and molecular or macromolecular level to understand the nano-scale phenomenon. The macroscopic behaviour of materials is a direct reflection of microstructure and physical characteristics at micro- and nano-scale level [20]. European Union Recommendation 2011/696/EU has defined nano-material as those materials which contain particles "in an unbound state or as an aggregate or more of the particles in number size distribution, one or more external dimensions is in size range 1–100 mm," 1 nm is equal to 10–9 m. The purpose of implementing nanotechnology in pavement engineering arena is to address those specific challenges which cannot be readily solved using existing macroscopic technology [20]. Some of common nano-material which have been used in pavement engineering are carbon nanotubes (CNTs), nano-clay (NC) (Santagata et al. 2015), nano-titanium dioxide (TiO_2) [18], nano-zinc oxide (ZnO), nano-aluminium oxide (AL_2O_3), nano-phosphor [20] and graphene. They had been used either individually or in combination or in addition to polymers [18]. A swelling process of the crumb rubber by light components of the Miltonic fraction was confirmed by modulated differential scanning calorimeter (MDSC). The stability tests, performed on samples stored at 180 °C, demonstrated that the storage stability of rubber-modified bitumen was improved as rubber concentration increased [16]. Crumb rubber modification has been proven to enhance the properties of pure bitumen. The rheology of CRMB depends on internal factors such as crumb rubber quantity, particle size and pure bitumen composition and external factors such as the mixing time, temperature and also the modification technique. These factors govern the swelling process of crumb rubber particles that lead to the increase of viscosity of the modified bitumen. However, the mixing temperature and duration can cause rubber particles to depolymerize and subsequently cause loss of viscosity. Crumb rubber modification also improves the properties of bitumen by increasing the storage and loss modulus and enhancing the high- and low-temperature susceptibility [9]. Nano-alumina possesses good thermal conductivity, high surface area, high strength and stiffness, mechanical strength, wear resistance, oxidation and thermal stability [14]. Studies have shown that nano-alumina has the potential to overcome the various shortcomings of different polymers (S. Zhang, X.Y. Cao, Y.M. Ma, Y.C. Ke, J.K. Zhang, F.S. Wang). Whereas the main disadvantage of crumb rubber-modified asphalt is its poor storage stability. Crumb rubber gets separated from asphalt at high temperature. Asphalt modified with nano-alumina is found to be storage stable indicating a good compatibility between nano-alumina and asphalt. In order to compensate for each other limitations, nano-alumina and crumb rubber will be used together in the current study.

2 Objectives of the Study

1. To determine optimum mixing time for preparation of crumb rubber-nano-alumina-modified bitumen.
2. To determine the optimum content of crumb rubber-nano-alumina using softening point method and storage stability.
3. To investigate the influence of crumb rubber-nano-alumina nanocomposite on viscosity of asphalt binder.
4. To compare conventional and modified binders using conventional tests.
5. To study the effect of crumb rubber-nano-alumina content on rheological behaviour of asphalt binder based on rutting and fatigue.
6. To evaluate the effect of crumb rubber-nano-alumina on ageing and high-temperature storage stability of asphalt binder.

3 Experimental Procedure

3.1 Material Characterization

In this study, three materials were used that is asphalt binder (VG-10), nano-alumina and crumb rubber. 80/100 penetration-grade bitumen (VG-10) purchased from a local distributor is selected as base binder. The various properties of the said base binder are shown in Table 3. Physical properties of base binder were determined through conventional tests which include softening point, ductility, penetration and specific gravity tests. Nano-alumina was supplied by Platonic Nanotech private limited and properties of nano-alumina are shown in Table 1 [3]. The physical properties of crumb rubber were also provided by the manufacturer as shown in Table 2. The SEM images of crumb rubber and nano-alumina are shown in Figs. 2 and 3, respectively. In thus study, the varying concentrations (1, 2, 3, 4, 5 and 6%) of nano-alumina are considered [3]. The stability tests, performed on samples stored at 180 °C, demonstrated that the storage stability of rubber-modified bitumen was improved as rubber concentration increased [16]. Mixing temperature was kept constant (163 °C) (Shafabakhsh et al. 2019).

Table 1 Physical properties of nano-alumina

Specification	Value
Purity	99.9%
Average size of the particle	30–50 nm
Specific surface area	130–150 m^2/g
Crystallographic nature	Rhombohedral
Morphology	Spherical
Melting point	2055 °C

Table 2 Properties of crumb rubber

Specification	Value
Particle size	<0.9 mm
Relative density	1.18
Natural rubber	35–58%
Rubber hydrocarbon	45–54%
Ash	4–6.5%
Carbon black	29–35%

Table 3 Results of tests conducted on VG-10 bitumen

Test	Standard code	Value	Specification limit (minimum)
Softening point (°C)	IS:1205	46	40
Ductility value (cm)	IS:1208	100 +	75
Penetration value 0.1 mm at 25 (°C)	IS:1203	88	80
Dynamic viscosity at 60 °C	IS:1206 (Part II)	1064	800
Kinematic viscosity at 135 °C	IS:1206 (Part III)	278	250

3.2 Sample Preparation

High-speed shear mixer was used whose main aim is to cause homogenous dispersion of additives into asphalt. High-speed shear mixer was used @ 3000 rpm at 180 °C [3, 17]. After that the modifier CR was added to it and was stirred continuously for 10 min manually then NA was added in small quantities and continuously stirred manually till complete mixing was achieved [15, 16]. The base and modified asphalt binder were made to undergo short-term ageing by conditioning in a thin film oven (TFO) as per ASTM:D 1754, for 5 h at 163 °C. Long-term ageing of modified asphalt binder was carried out in a pressure ageing vessel (PAV) as per ASTM:D 6521, for 20 h at 100 °C and 2.1 MPa. The whole experimental plan is separated into two steps (Fig. 1).

4 Tests Methodology

4.1 Rheological Evaluation

Dynamic shear rheometer (DSR) that is MCR 102 is used to evaluate the linear viscoelastic properties of unaged and aged bitumen over a range of frequencies and temperature in accordance with D7175–15. The following parameters were obtained: complex modulus ($|G^*|$), phase angle (δ), Superpave rutting parameter ($|G^*|/\sin\delta$), Superpave fatigue parameter ($|G^*|.\ \sin\delta$) and ageing index (AI). The temperature is varied from 46, 52, 58, 64, 70, 76 °C and so on and angular frequency is maintained at 10 rad/s (Bhat and Mir 2020) (Figs. 2 and 3).

4.1.1 Rutting Resistance Using Superpave Rutting Parameter

It is given by $|G*|/\mathrm{Sin}\delta$. It is used to assess the rutting resistance of the asphalt binder. An asphalt binder with higher value of $G*/\mathrm{Sin}\ \delta$ will have higher rutting resistance as lesser energy will be dissipated during each cycle of loading.

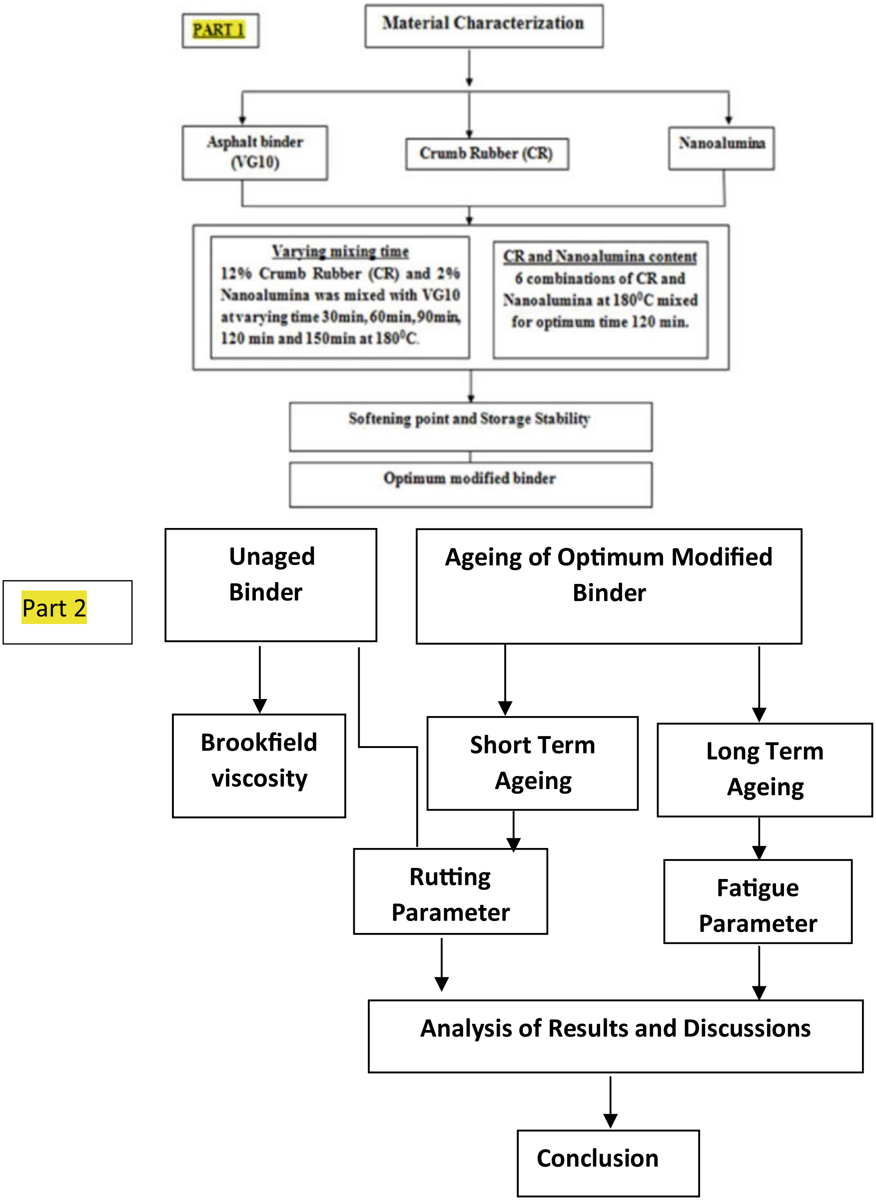


Fig. 1 Experimental flowchart

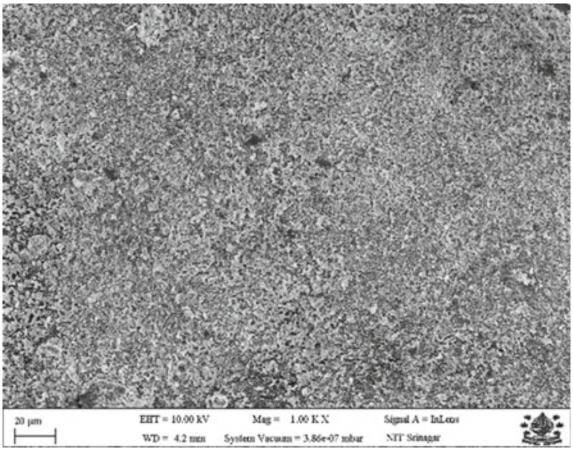


Fig. 2 SEM image of crumb rubber

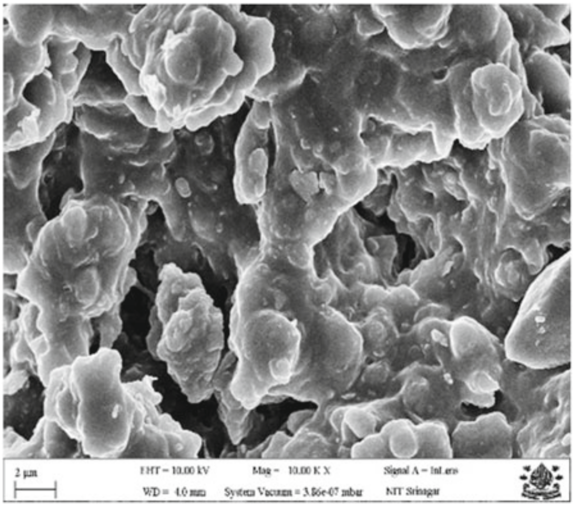


Fig. 3 SEM image of nano-alumina

4.1.2 Fatigue Resistance Using Superpave Fatigue Parameter

It is given by $|G*|.\mathrm{Sin}\delta$. It implies that if the dissipated energy per cycle is less, there will be lower stress accumulation. Thus, material with lower $|G*|.\mathrm{Sin}\delta$ has a better resistance against fatigue. As per SHRP, maximum value of $|G*|.\mathrm{Sin}\delta$ is 5000 kPa. The test temperature was selected as 25 °C.

4.2 Physical Characterization of the Binders

The basic properties of base binder and nano-alumina-crumb rubber-modified binder were evaluated, using penetration (IS: 1203), softening point (IS: 1205), viscosity (AASHTO T 316) and ductility (IS: 1208).

4.3 Viscosity Test

Rotational viscosity is used for determination of viscosity at mixing and compaction temperatures in Brookfield apparatus @ AASTHO T316-13. To determine the viscosity of the binder, torque required to rotate the vertical shaft was measured. 8–11 g of base and modified binders were tested in a chamber maintained at a temperature of 135 °C with the spindle rotating at a speed of 200 rpm.

4.4 Storage Stability

Storage stability test is used to evaluate the stability of modified asphalt binder as the modifier has a tendency to get separated from the asphalt binder at higher temperatures. This test is used to check whether the modifier has been homogenously mixed with the base binder. "The aluminium tube used in the test has a diameter of 25 mm and a height of 140 mm. It was filled with the modified sample and then stored vertically in an oven at 163 °C for 48 h in accordance with ASTM D7173-14 (Sun et al. 2006). Representative samples were taken from the top and bottom portions of the tube. The modified asphalt sample is considered to be high-temperature storage stable if the difference in the softening point of the asphalt sample from top and bottom sections is less than 2.5 °C (Galooyak et al. 2010).

4.5 Scanning Electronic Microscope

The SEM analysis was conducted to investigate the morphology of the nano-alumina powder and crumb rubber.

4.6 Ageing of Binder

The short-term ageing of the base and modified binders was done in accordance with ASTM:D 1754 using thin film oven. In this oven, a moving film of asphalt binder was heated for 5 h at 163 °C. The main aim of this method is to measure the effect of heat and air on a moving film of semi-solid asphaltic materials. To determine the effect of ageing on the binders' certain properties are tested before and after this test. "Short-term ageing determines the resistance to hardening under influence of air and heat. The conditions which occur during mixing and layout of asphalt mixtures are simulated by this instrument" (Martinho and Farinha 2019). The lone-term ageing of base and modified binders was done in accordance with ASTM:D 6521 using pressure ageing vessel (PAV). Long-term ageing was performed for 20 h at 100 °C with 2.1 MPa air pressure. It simulates the field condition for 5–7 years of service life in a pavement.

4.7 Effect of Nano-Alumina-Crumb Rubber on Ageing Resistance

Ageing refers to the oxidative ageing of the asphalt binders. Binders experience oxidation during the mixing and laying stage and also during their service life. During this process of ageing, some of the lighter and volatile components of the binder evaporate, making the binder brittle which can lead to pavement deterioration. Therefore, binders with a lower degree of oxidation are sought after. The present study evaluates the ageing resistance of the unmodified and modified binders using two parameters that is softening point incremental and ageing index.

4.7.1 Softening Point Incremental (SPI)

It is the difference between softening point of aged binders and unaged binders. Lower value of softening point incremental is required for binders having good ageing resistance.

$$\mathrm{SPI} = \mathrm{SPaged} - \mathrm{SPunaged} \tag{1}$$

where SPI is softening point increment, SPaged is softening point of short-term aged binder and SPunaged is softening point of unaged binder [7].

4.7.2 Ageing Index

Superpave rutting parameter is used to determine the ageing index which in turn gives the ageing resistance. The susceptibility to ageing increases as the value of AI increases (Ashish et al. 2016). Temperature is maintained at 60 °C at a frequency of 10 rad/s [3, 17].

$$AI = |(|G*|/\mathrm{Sin}\delta)\ \mathrm{aged}/(|G*|/\mathrm{Sin}\delta)\ \mathrm{unaged}| \tag{2}$$

where $(|G*|/\mathrm{Sin}\delta)$ aged and $(|G*|/\mathrm{Sin}\delta)$ unaged are rutting factor parameter of short-term aged and unaged asphalt binder, respectively (Martinho and Farinha 2019).

5 Results and Discussion

5.1 Determination of Optimum Mixing Time

Mixing temperature was kept constant at 180 °C [15]. Softening point was used to determine the optimum mixing time as softening point is a measure of high-temperature properties of binders. As softening point increases, high-temperature properties get enhanced. The optimum nano-alumina content in light of rheological improvements was found out to be 1–6% and the optimum CR content was found out to be 5–15% (Khasawneh et al. 2019). On basis of this, we maintained a constant concentration of 12% CR (Khasawneh et al. 2019)/2% nano-alumina and varied the mixing time from 30, 60, 90, 120 and 150 min. The optimum mixing time was obtained as 120 min using softening point method (Fig. 4).

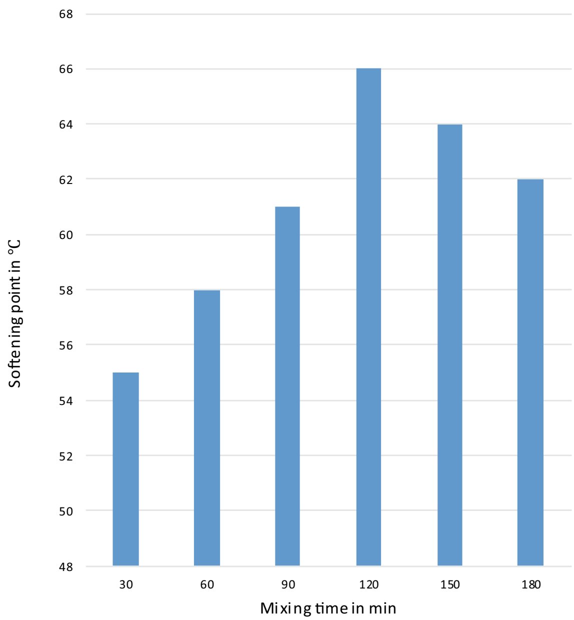


Fig. 4 Variation of softening point with mixing time for 12% CR/2% NA-modified binder

5.2 Determination of Optimum Mixing Concentration

The results of softening point test and storage stability test are represented in Fig. 5 and Table 4, respectively. The tests were carried on different combinations of CR and NA to determine the optimum mixing concentration (Fig. 6).

As it can be seen from Fig. 5, all the modified binders have higher softening points as compared to base binder. As a result, they are harder and less susceptible to permanent deformation. At constant CR concentration, as the NA content increases, softening point increases. At the constant CR concentration, the softening point initially increases slightly and later remains unaffected. This observation is consistent with other researches where it was observed that excessive number of nanomaterials in nanocomposite-modified asphalt binders may destroy the elastic nature of modified asphalt binders. 12% CR/4% NA has the highest softening point of 64.7 °C. So, economically, 12% CR/4% NA can be considered to the optimum concentration.

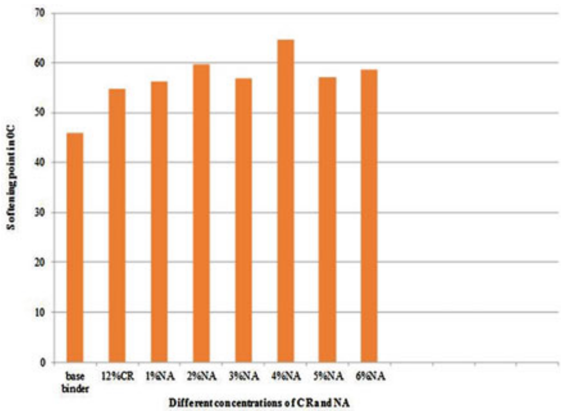


Fig. 5 Variation of softening points of binders modified with different concentrations of CR/NA mixed for 120 min

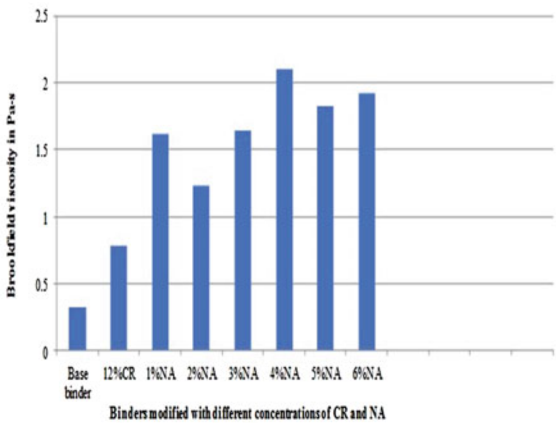


Fig. 6 Brookfield viscosities of base and modified binders at 135 °C

Table 4 Storage stability results of different combinations of CR/NA-modified binder

CR conc. %	NA conc. %	Softening point difference between top and bottom of tube	Storage stable Yes/No
12	0	9	No
12	1	2	Yes
12	2	1.8	Yes
12	3	1.2	Yes
12	4	0.74	Yes
12	5	1.21	Yes
12	6	1.01	Yes

When testing for storage stability at high temperatures, "the softening point between the top and bottom of the J samples should not be higher than 2.5 °C, indicating that there is no significant phase separation". Table 4 shows that when the NA content rises, there is a decreasing difference between the softening points of the top and bottom regions of CR-modified binders.

Because CR tends to expand in the oily fraction of asphalt as temperature rises, the differences between the densities of CR and NA become more pronounced. Furthermore, the relationship between temperature and a substance's density is inverse. As a result, with the same increase in temperature, the change in densities of CR and asphalt is different. As a result, at higher temperatures, the sedimentation velocity of modifier particles increases even more. Therefore, CR-modified binders frequently become unstable at higher temperatures. The density of the resultant nanocomposites is slightly higher with NA added, though. As a result, the total difference between the densities of the modifier particles and the asphalt reduces, which lowers the velocity of the modified particles' sedimentation. "At low polymer content, the amount of maltenes (more compatible with polymer) is probably enough to swell the macromolecules without causing the micellar structure of asphalt to become unstable, and the resins may stabilize the polymer-rich phase as they are supposed to do with asphaltenes, resulting in no phase separation. Because more polar aromatics are removed from maltenes at higher polymer concentrations, the colloidal structure of asphaltenes, maltenes, or micellar aggregates is no longer stabilized and has a tendency to collapse and settle as a non-fluorescent, high-density bottom phase.

5.3 Determination of Brookfield Viscosity

Figure 6 shows that as CR/NA concentration rises, asphalt viscosity also rises. Binders modified with 12% CR/4% NA have the highest level of viscosity. The outcomes line up with the values of the softening point. A thicker layer surrounds the aggregates when binders with a high viscosity are used. Cohesive forces between the parts grow as a result, and water and environmental deterioration is more resistant

Table 5 Physical characteristics of 12% CR/4% NA-modified binder

Test	Unit	Code	Value
Softening point	°C	ASTM D36 (2014b)	64.7
Ductility value	Mm	ASTM D113 (2007)	69
Penetration value	0.1 mm	ASTM D5 (2013)	62
Brookfield viscosity	Pa–s	AASTHO T316-13	2.1

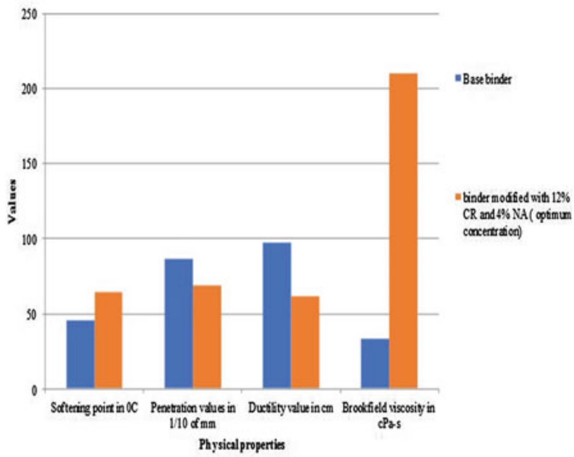


Fig. 7 Comparison of base binder and binder modified with optimum concentration of CR/NA (12% CR/ 4% NA)

to the system as a whole. However, the viscosity is not so high as to pose construction challenges, as extremely high viscous binders become hard and are challenging to compact. The improvements in asphalt binders' high-temperature qualities are shown in the results of rotational viscosity tests (Fig. 7 and Table 5).

6 Rheological Characterization

6.1 Complex Modulus

The variation of complex modulus with temperature is shown in Fig. 8. Complex modulus is a measure of stiffness that is the resistance of asphalt to deformation. In Fig. 8, it can be seen that an increment in NA concentration causes an increment in the complex modulus at a constant CR concentration. This is in accordance with other researches. The nanocomposite has a higher value of complex modulus which points out to the development of polymer network in the modified binder. The value of $|\mathbf{G}*|$ is highest for 12% CR/4% NA-modified binder.

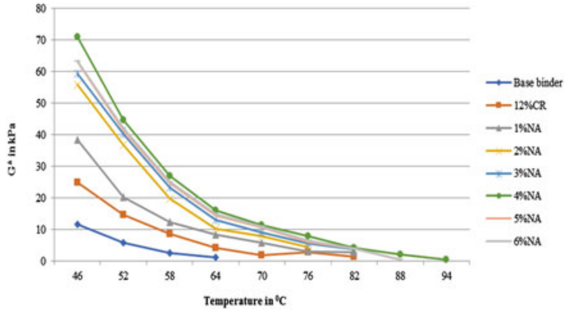


Fig. 8 Variation of |G∗| with temperature for binders modified with different concentrations of CR/NA

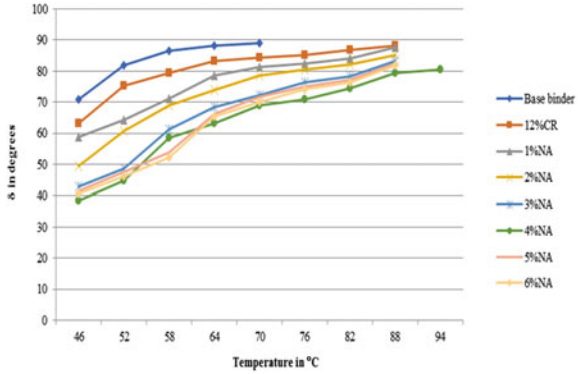


Fig. 9 Variation of phase angle with temperature for binders modified with different concentrations of CR-NA

6.2 Phase Angle

The variation of phase angle with temperature for base and modified binders are shown in Fig. 9. "Measurement of phase angle is generally considered to be more sensitive to chemical and physical structure than complex modulus for modification of asphalt". In Fig. 8, it can be clearly seen that addition of CR-NA decreases the phase angle and hence, enhances the elastic behaviour of asphalt. As the phase angle of base binder approaches 90°, it starts becoming viscous at higher temperature and makes a transition to Newtonian fluid. 12% CR/4% NA-modified binder possesses the lowest values of phase angle.

6.3 Superpave Rutting Parameter

The rutting parameter ($|G*|/\sin\delta$) versus temperature was plotted for the control and modified asphalt binders at different percentages of nano-alumina (Fig. 10). An exponential decrease in the rutting parameter was noticed with an increase in testing temperature for all tested asphalt samples.

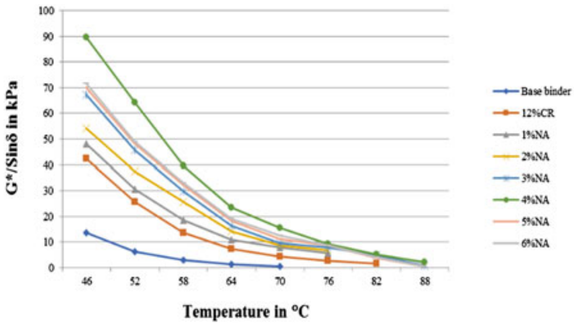


Fig. 10 Variation of superpave rutting parameters with temperature for binders modified with different concentrations of CR-NA

6.4 Failure Temperature T_{SHRP}

Strategic Highway Research Program from EEUU defined TSHRP as a temperature at which the value of rutting parameter becomes less than 1 kPa for unaged binders and 2.2 kPa for aged binders. It signifies the maximum temperature that can guarantee acceptable resistance to plastic deformations [6]. In physical terms, TSHRP in rheology is synonymous to softening point. Higher values of TSHRP signify enhanced high-temperature properties. The T_{SHRP} values are given in Fig. 11. Both CR and NA cause an increment in the T_{SHRP} values thereby improving its high-temperature properties. Thus, we can say that the interactions between CR and NA particles affect the mechanical properties of binders. The highest T_{SHRP} value is again shown by 12% CR–4%NA-modified binder.

NA can be considered to be a nanofiller in the polymer matrix of CR. The extent of improvement in the mechanical properties depends upon the degree of NA dispersion in the polymer matrix of CR. So, 12%CR–4%NA can be considered as best modifiers.

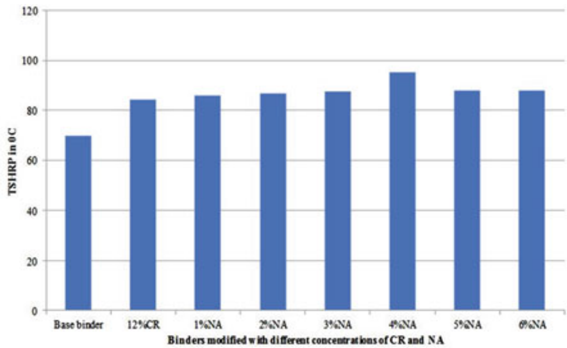


Fig. 11 T_{SHRP} values for binders modified with different concentrations of CR–NA

7 Determination of Ageing Effect

The effect of modifiers on the ageing resistance of the binders has been studied using two parameters.

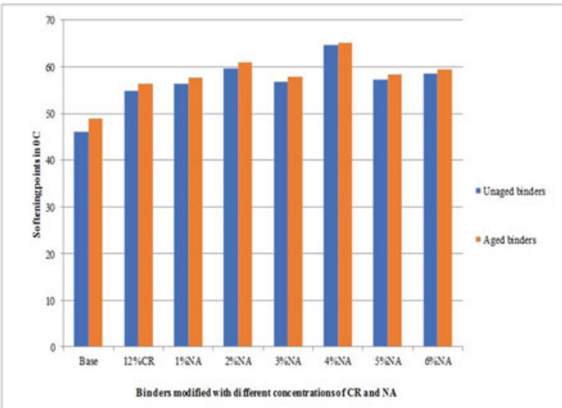
7.1 *Softening Point Incremental*

The comparison of softening points of unaged and aged binders is shown in Fig. 12. The softening point of aged binders is higher than that of unaged binders as binders become harder on ageing. The values of softening point incremental after short-term ageing are shown in Fig. 12, As the values of SPI decrease with the increasing NA concentration, the resistance to ageing increases. The least SPI value is shown by 12%CR–4% NA-modified binders (Fig. 13).

7.2 *Ageing Index*

The rutting factors of the aged base and modified binders are given in Fig. 14. The limiting value of rutting factor for the aged binders is 2.2 kPa. Aged binders have higher values of rutting factors as compared to unaged binders for all temperature ranges. This is due to the fact that binders become hard due to oxidative ageing after being subjected to TFOT. The rutting ageing index values were calculated at 60 °C at a frequency of 10 rad/s [3, 17], Fig. 14. It has been found that increased concentration of NA decreases the value of AI. The least value of AI is again showed by 12%CR–4%NA-modified binder (Fig. 15).

Fig. 12 Softening point variations of unaged and aged base and modified binders



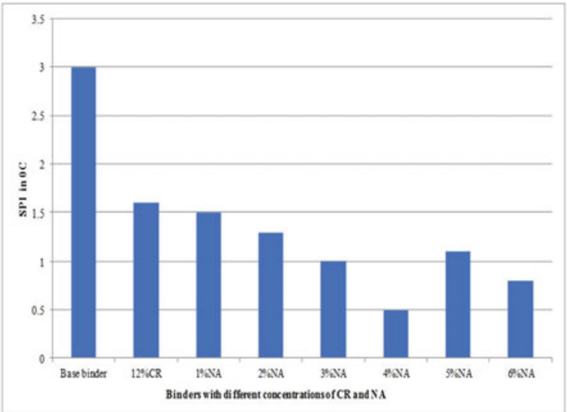


Fig. 13 SPI values for base and modified binders

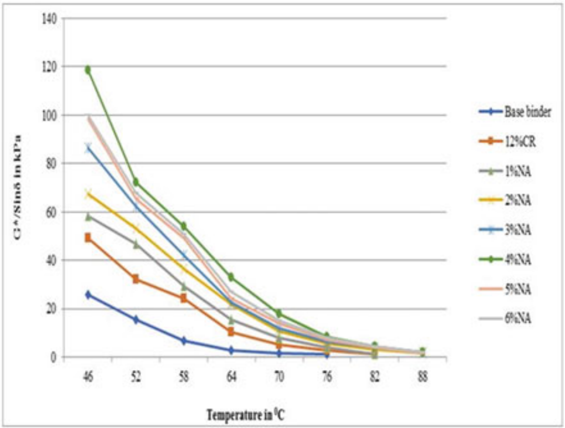


Fig. 14 Variation of superpave rutting parameters with temperature for RTFO aged binders

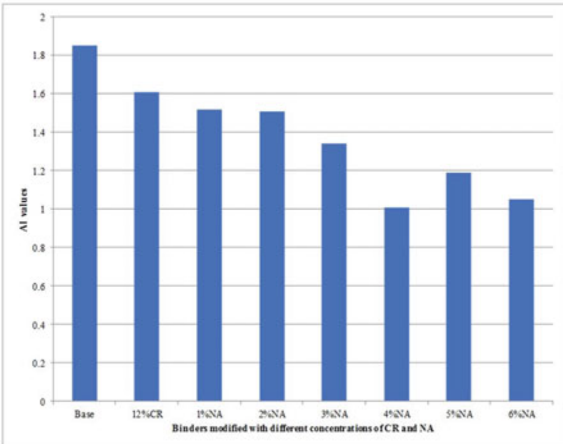


Fig. 15 AI values of base and modified binders

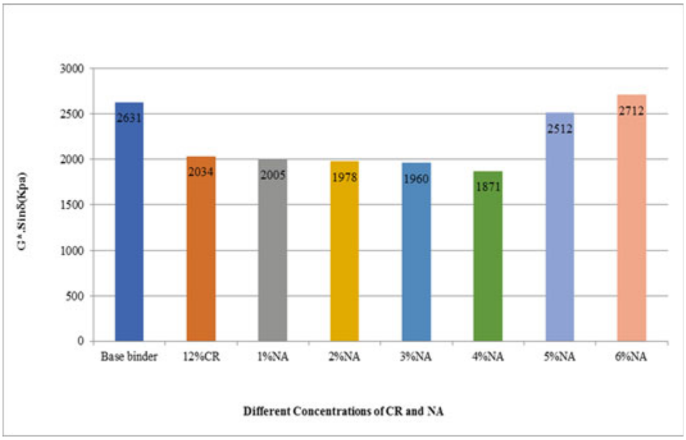


Fig. 16 Fatigue parameter values of different concentrations of CR and NS

8 Fatigue Performance

Figure 16 shows fatigue parameter (|G∗|.Sinδ) of different concentrations of CR and NA at temperature of 25 °C. Fatigue damage occurs due to repeated bending loads and outcomes in micro-damage in pavements. Such distress results in stiffness and decreases the applied load and ability to resist additional distress on the pavement, fatigue parameter of 12%CR–4%NA is decreased most as compared to other. Also fatigue parameter decreases with an increase in NA content. Thus, it can be stated that fatigue cracking resistance at intermediate temperatures can be enhanced by adding 12%CR–4%NA.

9 Conclusions

This thesis was carried out to develop CR-NA-modified asphalt binder by evaluating the optimum CR–NA content with proper mixing time. The thesis evaluates the effect of CR–NA on physical properties, storage stability, Superpave rutting parameter, Brookfield viscosity, ageing resistance and elastic behaviour of the modified binders. Six combinations of CR–NA were used by weight of control binder. The following conclusions can be drawn from the results and discussion:

1. The optimum mixing time can be considered to be 120 min on the basis of softening point. The NA was varied from 1 to 6% by weight of bitumen and CR concentration was taken as 12% by weight of bitumen. This is in accordance with softening point and storage stability. The best value was possessed by 12%CR/4%NA. The physical properties of base binders get enhanced due to CR-NA addition.

2. There is an improvement in high-temperature rutting resistance as reflected by an increment in the value of complex modulus and Superpave rutting parameter. Complex modulus increased from 11.72 kPa to 70.98 kPa and Superpave rutting parameter increased from 13.57 kPa to 89.54 kPa at 46 °C (of neat and optimum modified asphalt, respectively).
3. The failure temperature of neat asphalt was obtained as 70 °C and that of optimum modified was 95.3 °C. Thus, CR-NA addition could improve the high failure temperature in the unaged binder state.
4. CR-NA improves the elasticity of the binders as the value of phase angle decreases. The value of phase angle for neat asphalt was 71° and that of optimum modified was 38.55° indicating a decrease of 45.7% and thus an increase in elasticity.
5. CR-NA plays an important role in increasing the ageing resistance of the binders which can be derived from the fact that the softening point incremental and ageing index decreases due to the addition of CR-NA. The SPI of neat asphalt was obtained as 3 °C and minimum value was obtained for optimum modified bitumen, i.e. 0.5 °C. Also, ageing index of neat asphalt was 1.85 and minimum value was obtained for optimum modified bitumen, i.e. 1.01.
6. The enhancement in the physical and rheological properties of binders is directly related to the concentration of NA. The best result is shown by 12%CR–4%NA-modified binder. Similar trends are shown by ageing resistance.
7. CR–NA is effective in improving the viscosity of the binders as suggested by the values of Brookfield viscosity. The maximum value was obtained for 12%CR–4%NA which is 2.1 Pas.

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Development of Pavement Maintenance Management System (PMMS) for an Urban Road in New Delhi, India

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Abstract Pavements must be well maintained with proper utilization of maintenance funds as they are valuable national assets. Deferring pavement maintenance causes enormous financial losses and has a negative impact on the nation's growth. Simultaneous maintenance of existing roads is necessary in a timely way in addition to rapid construction of new road networks. Therefore, it is necessary to assess the state of pavements before deciding the type of maintenance needed, in order to make effective use of road maintenance funds. In the present study an Urban Road in New Delhi was assessed for its structural and functional condition to develop PMMS. The condition of the pavement has been evaluated in terms of pavement indices, such as the Pavement Condition Rating (PCR), and the Structural Capacity Index (SCI).

The deterioration of pavement with time is calculated using the deterioration models developed by CRRI in the year 1994, for Asphalt Concrete Roads in Northern India. A 15-year maintenance plan has been proposed for the selected Urban roads. In addition, a life cycle cost analysis has been performed to compare the costs under periodic and condition responsive maintenance strategies. It has been found that the condition responsive maintenance can be carried out at a cost of 4.7% less than the cost of periodic maintenance. Even though the difference in life cycle costs between periodic maintenance and condition responsive maintenance is minimal, the pavement can be maintained at level-1 or good condition under condition responsive maintenance.

Keywords Maintenance · Life-cycle cost · Deterioration · Pavement condition

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1 Introduction

Due to the advancement in human civilization, there is an increased need for the movement of men and materials. Passenger and freight transportation has grown into a key industry that is the backbone of a society's and country's development in terms of economy, technology, security, and growth. Road transport is possibly the oldest and, without a doubt, the most widely recognized and used means of transportation in the world. It is essential to have an efficient road transportation infrastructure for a country's development and advancement. A well-developed and well-maintained road transport industry serves as a catalyst for the country's development.

The majority of pavements in India are Flexible Pavements, (about 95% of total pavements), which deteriorate due to increasing traffic, increasing load, high tyer pressure, adverse environmental conditions, etc. Distresses like rutting, cracking, potholes, ageing are most common in flexible pavements, which greatly affects the service life. As a result, methods for maintaining pavements are required, to ensure that they perform as expected during their service life. In order to maintain stable road condition and ride quality, maintenance and rehabilitation (M&R) operations should be undertaken as a part of the maintenance plan, which results in pavement design life being attained in a cost-effective manner.

The present pavement maintenance strategy involves doing routine maintenance as necessary and restoring or strengthening pavements every 5 years without taking into account their possible economic implications. This study focuses on the Life-Cycle Cost Analysis (LCCA) of periodic and condition responsive maintenance strategies. It gives an overview of the most appropriate and cost-effective pavement maintenance strategy as well as the Net Present Value (NPV) difference between periodic and condition responsive maintenance strategies. Net present value method of LCCA is an economic tool used for evaluating the pavements, which considers both initial construction cost, and maintenance cost during the analysis period. The total life cycle costs of condition responsive maintenance and periodic maintenance are compared based on NPV, and the optimum maintenance alternative is taken into consideration.

2 Literature Review

2.1 Road Asset Management

Shah et al. [1] worked diligently on prioritization methods for effective maintenance of urban roads. According to his study, the most significant components of PMMS are determining the appropriate maintenance technique and prioritizing the pavement portions for treatment, and this prioritization can be done using a subjective ranking method. Another researcher Mandapaka et al. [2] emphasizes traditional LCCA is the fundamental tool for economic comparisons, and it may be used to assess the present costs of various pavement alternatives. Ashok et al. [3] used net present value (NPV) technique to evaluate the cost efficiency of alternatives, which determines the total cost needed over the project's life cycle. The study examined the life cycle costs of bituminous and concrete pavements, as well as bituminous and concrete overlays. According to Jain k et al. [4] financing a project in a developing country like India is challenging, and the funds provided are insufficient to satisfy the maintenance requirements, and therefore the Maintenance and Rehabilitation (M&R) plan for a specific highway stretch might be planned based on pavement conditions.

Pavement maintenance. Jain et al. [5] stated that the existing pavement maintenance program is centered on doing routine repairs as needed and renewing every five years without taking into account the costs and effects. Despite the fact that most recently developed software packages, such as HDM-4, give time-bound maintenance intervention criteria, condition responsive maintenance intervention is recommended for Indian conditions, which is more advantageous than planned maintenance. According to Hashema et al. [6] in order to avoid complicated combinations of distress levels (types, amount, and severity) and feasible M&R options, a Maintenance Unit (MU) system was developed which employs a combined index to highlight maintenance needs. The main purpose of the MU system is to decrease the number of linkages between distress data (type, severity, and density) and M&R alternatives that are necessary. Another researcher Raof et al. [7] combines both structural and functional conditions of pavement for the selection of M/R strategies and recommends that the treatment should be chosen based on the current condition and anticipated future performance.

Pavement Performance Prediction Models

Sood et al. [8] state that the pavement deterioration models are based on the traffic, environmental, and pavement conditions. Roughness, cracking, and pothole prediction models were developed based on the above distress parameters for Indian condition, which could be used to design an appropriate pavement management system that allows for maintenance planning and prioritization. Further Rohde et al. [11] developed model to determine the Structural Condition Index of the pavement directly from the deflection values of the Pavement.

3 Research Objectives

- To quantify the functional and structural condition of the pavement which includes pavement roughness, distresses like cracking, raveling, potholes, and deflections
- To propose a maintenance plan for 15 years for the selected Urban roads.
- To perform life cycle cost analysis under periodic and condition responsive maintenance strategies

Table 1 Details of selected pavement sections

Road No	Terrain	Traffic (CVPD)	Length (km)
R-1	Plain	5650	5
R-2	Plain	9257	5
R-3	Plain	7150	3
R-4	Plain	6397	7
R-5	Plain	7890	9
R-6	Plain	7538	6
R-7	Plain	7184	5
R-8	Plain	6698	8

4 Road Network

Eight heavy traffic corridors in Delhi, India, with varied lengths and an average width of 7.5 m, were chosen, for collecting pavement functional and structural data. Each road section is representative of the whole length of the road in terms of traffic and climatic conditions. The selected roads were two-lane urban roads with Bituminous concrete surfacing. The details of the study roads are tabulated in Table 1.

5 Data Collection

The functional evaluation has been carried out on road sections in which functional parameters like distresses, pavement unevenness in terms of IRI have been quantified. NSV was employed to collect the functional data of pavements, Table 2 represents the average distress on each road section. The structural evaluation of the selected road sections has been carried out using Falling Weight Deflectometer (FWD) and Pavement deflection, a structural characteristic of pavement, has been calculated.

6 Maintenance of Pavements

Pavement preservation is defined as a long-term, approach for improving pavement performance by implementing an integrated, cost-effective set of activities that extend pavement life, improve safety, and provide required serviceability. Road maintenance is routine work that is done to keep the pavement, shoulders, and other amenities provided for road users as nearer to their original construction state as feasible under normal traffic and weather conditions. Maintenance is necessary to ensure that the pavement structure provides the best possible service over its lifetime. Because of the traffic and environmental effects, all pavements require care. Maintenance

Table 2 Pavement Distress Data

Roads	Average cracking	Average raveling	Average pothole	Average patching	Average rutting	IRI
	%/km	%/km	%/km	%/km	%/km	%/km
R-1	0.30	0.19	0.00	0.03	4.03	3.684
R-2	0.27	0.03	0.00	0.14	4.48	3.980
R-3	0.37	0.03	0.00	0.03	2.99	4.547
R-4	0.28	0.01	0.00	0.05	3.52	4.848
R-5	0.04	0.01	0.00	0.00	3.32	4.454
R-6	0.47	0.01	0.01	0.02	3.29	4.355
R-7	0.55	0.00	0.00	0.02	2.93	4.832
R-8	0.94	0.00	0.00	0.03	3.32	2.409

aids in the preservation of the pavement surface and avoids the need for premature reconstruction.

Periodic Maintenance: The current pavement maintenance plan focuses on doing routine maintenance as needed and repairing or reinforcing pavements every 5 years without considering their potential economic effects. The fundamental disadvantage of this method is that certain pavements, although being in good condition, are resurfaced in accordance with a time-specific renewal cycle, while other pavements swiftly deteriorate while needing replacement but not being included in the maintenance cycle.

Condition responsive maintenance: To provide maintenance based on certain intervention criteria. The proposed criteria are based on the frequently used performance indicators, such as roughness, cracks, rutting, skid, and potholes. As per IRC 82, when the condition rating of a rural road reaches a value of 2 to 1, periodic renewal treatments may be selected. In the case of highways and urban roads, however, periodic renewal at a serviceability level of 2 may be undertaken. Before the pavement rating decreases below 2, preventive maintenance must be performed. Structural Condition Index (SCI) as per Rohde's model between 80 and 100 indicate the pavement is structurally sound [11]. The maintenance phase of a Pavement Management System's major goal is to figure out how much it costs to provide various levels of serviceability for a specific pavement. Serviceability levels of primary roads as given in Table 3 The serviceability level considered for this study is Level 1.

6.1 Intervention Criteria

A layer of 25 mm BC must be provided once every five years as part of periodic maintenance, as per MoRT&H recommendations. For the purpose of reinforcing an existing pavement, a 75 mm DBM and 40 mm BC overlay must be provided every 10 years after construction. The intervention criteria for the study roads have been

Table 3 Levels of primary roads

No	Serviceability indicator	Level 1 (Good)	Level 2 (Average)	Level 3 (Acceptable)
1	Roughness by B.I (max. permissible)	2000 mm/km	3000 mm/km	4000 mm/km
2	Potholes per km (max. number)	Nil	2–3	4–8
3	Cracking and patching area (max. permissible)	5%	10%	10–15%
4	Rutting—20 mm (Maximum permissible)	5 mm	5–10 mm	10–20 mm
5	Skid Number (Minimum desirable)	50 SN	40 SN	35 SN

Source [9]

Table 4 Intervention criteria

No	Alternatives	Work type	Intervention criteria
1	Preventive maintenance	Crack repairs	>10%
		Pothole repairs	>1%
2	Thin overlay	25 mm BC	IRI >3000 mm/km’
3	Thick overlay	40 mm BC + 75 mm DBM or	IRI >3000 mm/km’ After thin overlay is provided
		Recycling Of 40 mm bituminous layers and providing 100 mm DBM + 40 mmBC	

developed based on Table 5 and also on the opinion from the experts in the field. The preventive maintenance of pavement is carried out when cracking and pothole are greater than 10% and 1%, respectively. Thin overlay of 25 mm BC is given when the IRI of the road is more than 3000 mm/km. Thick overlay of 75DBM + 40BC is provided when the IRI increases more than 3000 mm/km even after providing a thin overlay (Table 4).

7 Deterioration of Pavements

The pavement deterioration models were developed by CRRI in 1994, under Pavement Performance Studies (PPS), for Indian roads, by considering the pavements with different wearing courses, viz., Asphalt Concrete, Semi Dense Carpet, and Premix Carpet in the states of Haryana, Gujarat and Rajasthan. 500 sections with traffic ranging between 1400 and 10,872 CVPD and length not less than 1 km were considered. Numerous time series data and correlations between pavement and pavement performance were included in the PPS. The study developed pavement performance prediction models for the major distresses, including cracking, potholes, Raveling, and roughness, which are most significant from road maintenance and road user cost considerations [10]. Since the models developed suited to the condition of selected roads of in terms of traffic, wearing course and the distresses were within the range as used in model development in the present study, these models were adopted for determining deterioration of selected roads during analysis period. The major distresses on study stretches were cracking and potholes; therefore, only cracking and pothole models were adopted to calculate the deterioration along with the roughness model.

8 Results

Both functional and structural evaluation of all the study stretches was carried out as mentioned in 5.1. The functional condition of the pavement is determined in terms of Pavement Condition Rating (PCR) as per IRC 82 2015 and structural condition in terms of Structural Condition Index (SCI) as per Rohde's model [11] (Figs. 1 and 2).

The deterioration of pavements is calculated for an analysis period of 15 years and the maintenance plan is shown in the table below. Since the study stretches were structurally adequate and the roughness of all the sections except section R-8 was more than 3000 mm/km a functional overlay of mixed seal surfacing is provided for R-1 to R-7 in the zeroth year (current Maintenance need) and then deterioration analysis is carried out and maintenance is provided based on intervention criteria. Table 5 shows the Maintenance Plan.

Economic Analysis

The life cycle cost analysis begins with the assumption of the analysis period. An analysis period of 15 years is assumed in the current study. Costs are calculated from the schedule of rates of the Public Works Department (PWD), Delhi region.

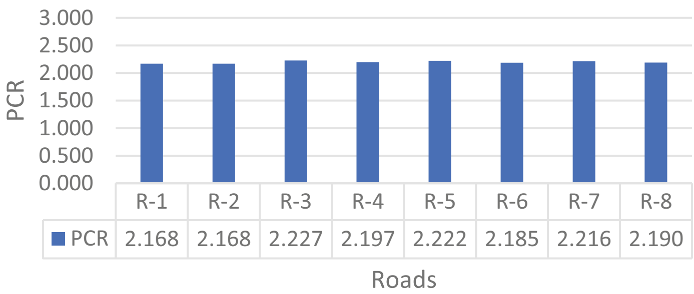

Fig. 1 Pavement condition rating

Fig. 2 Structural capacity index

Table 5 Maintenance plan

Road no year	R-1	R-2	R-3	R-4	R-5	R-6	R-7	R-8
0	FO	FO	FO	FO	FO	FO	FO	–
1	–	–	–	–	–	–	–	–
2	–	–	–	–	PM	–	–	FO
3	–	PM	PM	PM	–	PM	PM	–
4	PM	–	–	–	–	–	–	–
5	–	TO	TO	–	TO	TO	–	–
6	TO	–	–	TO	–	–	PM	PM
7	–	–	–	–	PM	–	–	–
8	–	PM	PM	–	–	PM	TO	TO
9	–	–	–	PM	–	–	–	–
10	PM	ST	ST	–	ST	ST	–	–
11	–	–	–	–	–	–	PM	–
12	ST	–	–	ST	PM	–	–	PM
13	–	PM	PM	–	–	PM	–	–
14	–	–	–	–	–	–	ST	ST
15	–	TO	TO	PM	TO	TO	–	–

FO: functional overlay of mixed seal surfacing, PM: Preventive Maintenance, TO: Thin Overlay of 25mm BC, ST: Strengthening of with 50mm BC + 75mm DBM

According to government policy, a discount rate of 12% and an inflation rate of 5.5% have been taken into account for future increases in material prices. The net Present Value of each of the alternatives is calculated. The alternative with the lowest NPV is selected as the best alternative. The comparison of NPV under periodic and condition responsive maintenance is presented in Tables 6 and 7.

Table 6 NPV comparison

Road no	Road length	NPV in crores		
	km	PM	CRM	CRMR
R-1	5	16.90	15.83	15.35
R-2	5	17.28	16.81	16.24
R-3	3	9.69	9.62	9.29
R-4	7	22.09	19.66	19.18
R-5	9	29.75	29.77	28.79
R-6	6	19.83	19.27	18.90
R-7	5	17.65	16.48	16.03
R-8	8	25.84	23.91	23.22

**PM: Periodic Maintenance; CRM: Condition Responsive Maintenance; CRMR: Condition Responsive Maintenance with Recycling

Table 7 D% Difference in NPV between PM and CRM

Road no	% Difference in NPV between PM and CRM	% Difference in NPV between PM and CRMR
R-1	6.33	9.17
R-2	2.72	6.02
R-3	0.72	4.13
R-4	11	13.17
R-5	0.07	3.23
R-6	2.82	4.69
R-7	6.63	9.18
R-8	7.47	10.14
Average	**4.7**	**7.47**

9 Pavement Performance

The pavement performance is calculated in terms of PCR as per IRC 82, 2015 under both periodic and condition responsive maintenance strategies. Figure 3 represents the performance of road R-1 under above-mentioned maintenance strategies.

10 Conclusions

The following conclusions have been drawn on the basis of this study:

- The economic analysis proves that condition responsive maintenance is more economical than periodic maintenance.

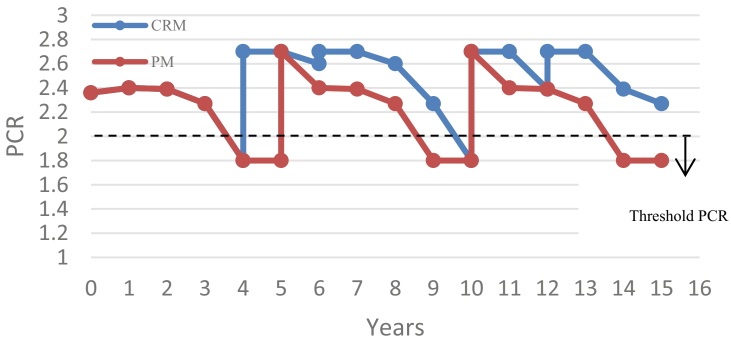

Fig. 3 Pavement Condition under periodic and condition responsive maintenance

- The condition responsive maintenance can be carried out at a cost of 4.7% less than the cost of periodic maintenance.
- Even though the difference in costs between periodic maintenance and condition responsive maintenance is minimal (around 1 crore on an average), the pavement can be maintained at level-1 or good condition under condition responsive maintenance as shown in Fig. 3.
- The condition responsive maintenance with recycling of the bituminous layer can be done at a cost of 7.47% less than the cost of periodic maintenance.

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A Study on the Design of Sustainable Bituminous Concrete Pavement with Subgrade Soil Stabilization

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Abstract The design and construction of sustainable bituminous concrete pavements are playing a critical role across countries with the rapid growth of transport infrastructure. Transporting subgrade soil having good engineering properties with more lead distances is not a sustainable option. The main aim of the research paper is to study the effect of stabilized subgrade soil in the Mechanistic-Empirical design with two cross sections of traditional aggregate bases subbase (Type-I) and cement-treated subbase layers (Type-II). The life cycle cost analysis and life cycle analysis of the bituminous concrete pavement were carried out to assess the economic viability and carbon footprint estimation using the Net Present Value method and Cradle to Grave approach respectively. From the recent literature, two stabilized subgrade soils (a) CL soil stabilized with 7.5% Flyash and Eko soil enzyme (Case-I) and (b) CL with 5% Lime stabilized (Case-II) were considered for pavement design. All other required data for pavement design and the density of materials used for construction were assumed. From the results, it is observed that the life cycle cost of Type II bituminous concrete pavement with Case-II subgrade soil is 37% less compared with Type-I pavement with Case-I subgrade soil. In the case of Type-II pavement with Case-II, subgrade soil the carbon footprint reduces up to 53% compared with Type-I pavement with Case-I subgrade soil. From this, it can be concluded the Type-II bituminous concrete pavement designed with Case-II subgrade soil is more sustainable improving the long-term life of the pavement without critical failures like fatigue cracking and wheel path rutting.

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1 Introduction

The road network across the world in terms of Expressways, National Highways, and other low-volume roads is rapidly increasing across the world. On the other hand, sustainability is the main concept in engineering projects which looks after the economy, long-term performance, and less carbon footprint from an environmental point of view. To design a pavement, soil subgrade plays a key role as it supports the entire loading coming from the above layers which should perform in a better manner. To enhance the efficiency of expansive soil subgrade, the soil is stabilized by using some additives like cement, fly ash, lime, etc. which improve the geotechnical properties of the soil. The borrowing of subgrade soil from long distances not only increases the cost of the other hand carbon footprint from transport vehicles. Further, the Mechanistic-Empirical pavement design has its importance in terms of addressing critical failures using alternative materials with long-term performance meeting sustainability requirements. The critical stresses and strains in the pavement can be evaluated by using a linear elastic analytical model using IIT Pave software (IRC-37, 2018) [1]. Further, to determine the total economic worth of the pavement the Net Present Value method is used for life cycle cost analysis. To determine the greenhouse gas emissions from the pavement throughout the Life cycle the cradle-to-grave approach is adopted. The main objective of the research paper is to understand the effect of the CBR of stabilized subgrade soil on the Life Cycle Cost analysis and Life Cycle Analysis.

2 Literature Review

Dhar and Hussain have studied the lime, which is used to stabilize the subgrade soils for improving both the strength and bearing capacity of those soils. Various laboratory tests such as linear shrinkage, Unconfined Compressive Strength (UCS), Split Tensile Strength (STS), and California Bearing Ratio (CBR) have been done at varying percentages of lime (3%,5%,7% & 9%) and different curing durations (0, 7, 14 and 28 days). He has concluded that by the addition of lime–shrinkage strains have been reduced but the UCS, STS, and CBR values of subgrade soil have been increased effectively [2]. Kumar and Harika have presented the stabilization of expansive soils like black cotton soils (BCS) with fly ash as a stabilizer at different proportions. Various laboratory tests like Atterberg's limits, CBR and UCS, etc. are done. By the inclusion of fly ash from 0 to 20%, Atterberg's limits, plasticity index, and free swell index have been decreased. The CBR value has been increased from

2.189% to 2.33% and the UCS value also has been increased from 0.1688 N/mm^2 to 0.333 N/mm^2 which are obtained at 10% fly ash. Hence, he concluded that the use of fly ash up to 10% with black cotton soil is recommendable for construction purposes related to pavement and foundation works [3]. Karama et al. have investigated the low clayey soil stabilization with primary additive first-class F fly ash, an industrial by-product, and then with secondary additives-lime, CSA cement, enzyme, and polymers. As already, the soil is first stabilized with fly ash, by adding secondary additives to that stabilized soil, which has given excellent performance. By using all primary and secondary additives, many combinations have been tried to stabilize the soil. Out of these, soil-fly ash-lime-enzyme is considered the most effective one. Then, it is split into two parts i.e., soil-fly ash-lime and soil-fly ash-enzyme, which have shown terrific excellent performance in subgrade. He concluded that when fly ash stabilized weak soils are treated with secondary additives, the thickness of the pavement layer can be reduced which also results in economics [4]. Anand et al. have presented the stabilization by using refractory castable material in subgrade soil and carried out different tests like compaction and strength at varied proportions from 15 to 50%. Also, investigated how this castable material will influence the thickness of the bituminous layer in flexible pavement. By using IIT Pave software, the design of flexible pavement has been performed. Finally, they concluded that after the stabilization of the subgrade, the thickness of the bituminous layer has been reduced. In the Bituminous wearing course and wearing course, the thickness of 3 and10 mm has been eliminated. Also, presented the economic benefit analysis for NH-48 (National highway: Jaipur-Ajmer) by using HDM-4 for various maintenance alternatives [5]. Kulkarni et al. have discussed the proposed rigid pavement and flexible perpetual pavement for the western alignment of the Pune ring road. This study had the objective to propose an alternative regarding choosing perpetual pavement over the proposed rigid pavement. They studied four combinations of flexible pavement as per IRC guidelines and perpetual pavement design criteria. They presented the rigid pavement on the grounds of sustainability, Life cycle cost analysis (LCCA), and carbon dioxide (CO_2) emissions. By using IIT pave software, the perpetual flexible pavements have been designed. Finally, they concluded that the perpetual flexible pavements have been a good choice with some factors like expected design life, economy and environment [6]. From this section, it is understood that there is a literature gap on the comparison of flexible pavement designs with soil stabilization and considering Life Cycle Cost Analysis and Life Cycle Analysis. In addition to that soil stabilization with lime and fly ash is possible can be possible as they are available in huge quantities.

3 Methodology

In this research paper from the literature two cases considered of low clayey subgrade soils which were stabilized to improve the California Bearing Ratio value and other engineering properties. The design of bituminous concrete pavement was carried out using IRC code 37–2018 and IIT Pave software [1]. The Mechanistic-Empirical design of bituminous concrete pavement has its importance in the selection of industrial products, and recycled materials to improve the sustainability of pavements. In the design process, the following CBR values of the two soil cases are considered for estimating the resilient modulus of the subgrade soil. The resilient modulus of the subgrade soil and aggregate layers was estimated using the empirical Eqs. 6.1, 6.2, and 7.1 as given in the IRC-37–2018 Code [1].

Case-I: In this case**,** Low Clayey soil having a plasticity index of 30% has been considered which has stabilized with fly ash and enzyme (Eko soil which increases the compaction) in different combinations. The combination of soil with 7.5% fly ash and enzyme (Eko soil) is considered which has a maximum CBR value of 5.33% [4].

Case-II: In this case Low Clayey soil having a plasticity index of 20.3% considered which has stabilized with lime at different proportions. The soil stabilized with lime at 5% has a maximum CBR value of 18.1% [2].

The material properties of different layers were assumed according to Indian Road Congress code IRC-37–2018 [1]. The resilient modulus of granular layers in the traditional bituminous concrete pavement was calculated according to the empirical relations given in the IRC code in Eq. 3. The Poisson's ratio of granular layers was considered 0.35. The elastic modulus of the cement-treated subbase and cement-treated base course layers were considered according to the IRC code with the maximum value of 600 MPa and 5000 MPa respectively and a poison's ratio of 0.25. The flexural strength of the cement-treated base course layer is considered as 2.5 MPa. The resilient modulus of bituminous concrete layers including dense bituminous concrete and bituminous concrete layer is considered according to the IRC-37 as 3000 MPa for VG-40 at an average annual temperature of 35˚C. The resilient modulus of the granular crack relief layer is taken as 450 MPa according to the IRC-37 code. In the bituminous concrete pavement design two different cross sections were considered. (a) Type-I bituminous concrete pavement cross-sectional designed with traditional aggregate subbase and base course layers. (b) Type -II bituminous concrete pavement designed with the cement-treated subbase and cement-treated base course layer were used. In the case of Type-I bituminous concrete pavement, the load acting on the pavement was considered as 80 kN with a tire pressure of 0. 56 MPa. In the case of the Type-II bituminous concrete pavement with cement concrete layers, the load was considered as 80 kN with a tire pressure of 0.8 MPa. The spacing between the center line of the dual tires is considered 310 mm [1]. The bituminous concrete pavements of Type-I and Type-II were designed for both the cases considered with different stabilized soil CBR values. The bituminous concrete pavement thickness design was carried out using the 90% reliability equations in fatigue cracking, rutting, and fatigue life of the cement-treated equations given in the IRC Code-37 (2018) [1].

Table 1 The Axle load group data considered for Fatigue damage analysis

Axle load group data			
Axle load (KN)	Expected repetitions	Axle load (KN)	Expected repetitions
185–195	17,500	390–410	50,000
175–185	22,500	370–390	57,500
165–175	23,000	350–370	60,000
155–165	75,000	330–350	58,750
145–155	70,000	310–330	56,250
135–145	162,500	290–310	475,000
125–135	150,000	270–290	112,500
115–125	335,000	250–270	358,750
105–115	1,300,000	230–250	312,500
95–105	375,000	210–230	296,250
85–95	1,350,000	190–210	250,000
<85	925,000	170	200,000
		<170	3,200,000

3.1 The Design Data Considered for Bituminous Concrete Pavement

The road considered is a highway with a four-lane divided carriageway with bituminous concrete pavement. The initial traffic in the year of completion is 1,500 commercial vehicles per day. The growth rate of traffic is considered as 6%. The design life of the pavement is considered as 20 years. The vehicle damage factor is assumed as 5.2. The Marshall mix design was carried out on the bituminous mix to be used as the Dense Bituminous Macadam layer (DBM). The layer has an air void content of 3% resulting in an effective bitumen content (by volume) of 11.5%. For Type II bituminous concrete pavement with cement-treated layers, the cumulative fatigue damage analysis has been carried out using the axle load data given in Table 1. The design MSA is calculated for the above data using Eq. 4.5 as per the IRC-37 code [1].

3.2 Life Cycle Cost Analysis

The Life cycle cost analysis is an analytical technique and is useful in making decisions regarding financial investments in any engineering project. The life cycle cost analysis is the sum of initial cost, maintenance, reconstruction, repairs, rehabilitation, and vehicle operation costs. The Net present value method is very much helpful to evaluate the economic viability of a project. The inflation cost is considered 8%

and discounted interest rate is considered 12%. The costs and the benefits of the independent years are discounted to the current value and compared across various alternatives in the Net Present Value method [7]. The United States Dollar was considered as 1 dollar equal to 67.78 rupees. In this analysis, we have not calculated the vehicle operation as we are comparing the life cycle cost analysis between two types of pavements and suggesting which option is better. Moreover, the vehicle operation cost of Type II bituminous concrete pavement with a cement-treated subbase layer and cement-treated base course layer will be less as the pavement failures are less and maintenance is not needed for at least 10 years [8].

Life Cycle Analysis

For carbon footprint estimation, the Life cycle analysis (LCA) is carried out using the cradle-to-grave approach at the production stage, transport stage plant stage, and laying the pavement stage at the site. This method helps to calculate the carbon footprint which is responsible for the global warming effect. The following other assumptions were made regarding the Life cycle analysis of the bituminous concrete pavement. An A14-ton truck was considered regarding the transport of materials and mixes for greenhouse gas estimation. The average distance considered for the transport of aggregate from the crusher to the plant and the transport of mixes and materials to the site and 5 km (Table 2).

Table 2 The Job mix formulae considered for different layers in the bituminous concrete pavement of Type-I and Type II

S. No	Item name	Density (kg/m^3)	Job mix formula
1	Granular sub-base	1602	40 mm-35%,20 mm-20%, 10 mm-15%, 6 mm and dust-30%
2	Wet mix macadam	2350	40 mm-32%, 20 mm-20%, 10 mm-20%, stone dust-28%)
3	Dense bituminous concrete	2450	In the case of Dense Bituminous macadam, bitumen is taken as 5%, with lime of 3% and 91.5% of aggregates, 20 mm-35%, 10 mm-20%,6 mm-20%, stone dust-30%
4	Bituminous concrete layer	2500	In the case of Bituminous concrete bitumen is taken as 6%, with lime of 2%, 92% aggregates, 12 mm-26%, 6 mm-32%, fine dust-40%
5	Cement treated base layer	2500	40 mm-32%, 20 mm-20%, 10 mm-20%, 6 mm = 14%, stone dust-4%, Cement = 6%
6	Cement treated sub-base layer	2450	40 mm-32%, 20 mm-20%, 10 mm-20%, 6 mm = 14%, stone dust-4%, Cement = 4%

The carbon emissions for different materials in the construction stage, including the transportation trucks all were taken from the references as mentioned [9–13].

4 Results and Discussion

The design of Type-I and Type-II bituminous concrete pavement with traffic of up to 79 million standard axles has been carried out for the two stabilized soil cases considered. The type I bituminous concrete pavement should be checked for fatigue cracking and wheel path rutting. In addition to these Type-II bituminous concrete pavement with cement-treated layers should be checked for fatigue cracking of the CTB layer and fatigue damage analysis (Table 3).

The above cross-sectional details were taken as input into IIT Pave software and the stress and strains were obtained. Moreover, these stress and strains were used for calculating allowable MSA in fatigue cracking, wheel path rutting, and fatigue cracking of the CTB layer using the equations given in IRC-37–2018 [1]. The cumulative fatigue damage value is less than unity for all the cases with the pavement cross sections considered.

From the life cycle cost analysis carried out for traditional bituminous concrete pavement (Type-I) and bituminous concrete pavement with cement-treated layers (Type-II) for two different cases considered the following conclusions were made. It is observed that using the net present value method the life cycle cost analysis of Type-II bituminous concrete pavements with Case-II subgrade soil reduces by 37% compared with traditional bituminous concrete pavement Type-I. In addition to that the improvement in the subgrade resilient modulus from 53.3 MPa to 100 MPa the life cycle cost of Type-I, and Type-II bituminous concrete pavement was reduced by 9.66% and 4.2%, respectively (Table 4).

Table 3 The pavement crust thickness results were as shown for Case-I and Case-II. The results were shown as follows

Thickness design	Case-I		Case-II	
Layer thickness (mm)	Type-I	Type-II	Type-I	Type-II
Bituminous concrete	50	50	50	50
Dense bituminous macadam	150	100	100	100
Granular crack relief layer	*NA	100	*NA	100
Granular base course	250	*NA	200	NA
Granular subbase layer	250	*NA	250	NA
Cement treated base	*NA	200	*NA	250
Cement treated subbase	*NA	300	*NA	250

* NA-Not Applicable

Table 4 The critical Stress and Strain values obtained in different layers and allowable MSA for fatigue cracking, and wheel path rutting for the bituminous concrete pavement

S. no	Description	Case-I		Case-II	
		Type 1	Type-II	Type 1	Type-II
1	Maximum horizontal tensile at the bottom of the bituminous layer	0.0001454	0.000106	0.0001487	0.0001064
2	Allowable MSA in fatigue cracking	179	506	136	498
3	Maximum vertical compressive strain at the top of the subgrade	0.0001064	0.00004089	0.0001239	0.00003353
4	Allowable MSA in wheel path rutting	3986	1,108,742	7279	2,726,221
5	Maximum horizontal tensile at the bottom of the CTB layer (Micro Strains)	*NA	24	*NA	27.22
6	Allowable MSA in fatigue Cracking of CTB layer	*NA	22,399,791	*NA	4,944,489
7	Maximum horizontal tensile stress at the bottom of the CTB layer (MPa)	*NA	0.2	*NA	0.167

* NA-Not Applicable

The life cycle analysis using cradle-to-grave approach has been carried out for Type-I and Type-II bituminous concrete pavements. This process includes the estimation of the carbon footprint at different stages like the production stage, transportation of raw material to the construction site, mix production at the plant, and construction stage. From the results, it is observed that the Type-II bituminous concrete pavement with cement-treated layers has less carbon footprint at all stages except the production stage compared with Type-I bituminous concrete pavement. In the production stage, the Type II bituminous concrete pavement with cement-treated layers has high carbon footprint than the Type-I bituminous concrete pavement as the cement will have more carbon dioxide emissions in the production process. With the increase in resilient modulus of the soil subgrade from 53.3 MPa to 100 MPa the carbon footprint in the bituminous concrete pavement with (Type-II) at the production stage was increased by 23% (Figs. 1, 2 and 3).

On the other hand, the transportation of raw material to the site was reduced by 24.58%, mix production at the plant was reduced by 28.41%, and at the construction stage up reduced by up to 20.77% compared with Type-I bituminous concrete pavement.

The Type-II bituminous concrete pavement designed with case II subgrade soil has 53% less carbon foot compared with Type-I bituminous concrete pavement designed with the case I subgrade soil. Further, as the resilient modulus of subgrade soil increased from 53.3 MPa to 100 MPa the total carbon footprint of Type-I, and Type-II bituminous concrete pavement reduces by 26.24% and 8.36%, respectively.

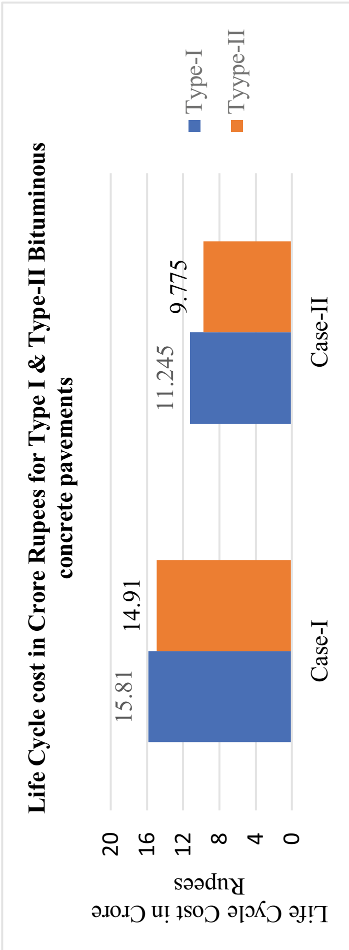
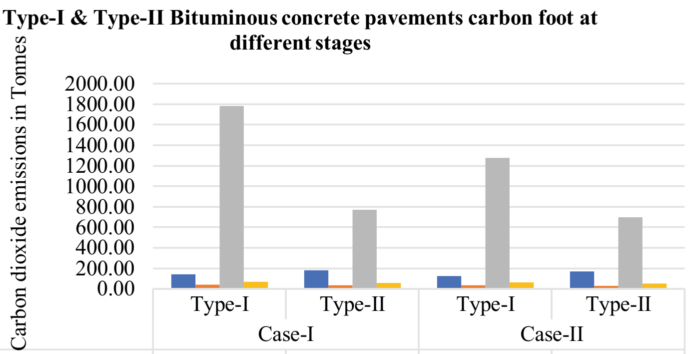


Fig. 1 Life cycle cost comparison of different cases and types of bituminous pavements



	Case-I Type-I	Case-I Type-II	Case-II Type-I	Case-II Type-II
Production Stage	139.00	180.97	122.87	171.10
Transportation of Raw material to construction site	40.60	33.39	34.10	30.18
Mix production at the plant	1784.42	770.67	1277.41	699.70
Construction	66.72	56.35	63.37	52.86

Fig. 2 Carbon dioxide emission in Tones for Traditional (Type-I) and Type-II bituminous concrete pavements for Case-I and Case-II

5 Conclusions

1. The life cycle cost of Type-II bituminous concrete pavement of case II subgrade soil is 37% less compared with Type-I bituminous concrete pavement with the case I subgrade soil. The Type-II bituminous concrete pavement with the subgrade resilient modulus of 100 MPa helps to reduce the cost of the project with less thickness of surface course layers improving the long-term performance and avoiding fatigue cracking and wheel path rutting in the pavement.
2. The Type-II bituminous concrete pavement designed with the subgrade resilient modulus of 100 MPa helped to reduce the total carbon foot print by 53% compared with Type-I bituminous concrete pavement designed with the case I subgrade soil. From this, it is very clear that Type-II bituminous concrete pavement with cement-treated subbase and base course layer is more sustainable in protecting the environment with a reduction in the production of bituminous mix quantities for pavement construction.
3. It is observed only in the production stage that Type-II bituminous concrete pavement has a more carbon footprint because of more cement production for cement-treated layers.
4. The stabilization of low clayey subgrade soils with materials like fly ash, lime, granite dust, etc. can help the improvement of subgrade resilient modulus up to 100 MPa. The Type II bituminous concrete pavement designed with this stabilized soil can make the pavement sustainable improving the pavement's long-term performance with economy and environment friendly.

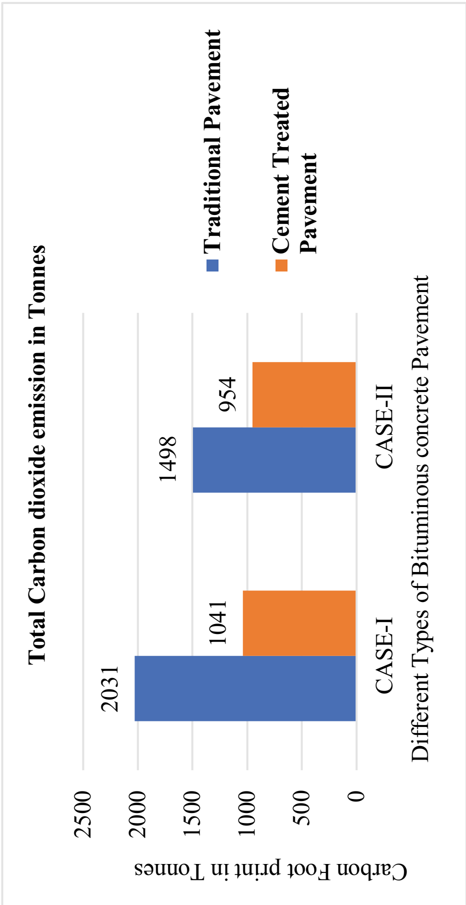


Fig. 3 The Total carbon footprint estimation for Type -I and Type-II bituminous concrete pavement in both cases

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Investigation of Effect of Aging on Reflection Cracking in Overlay

Sarfaraz Ahmed, Abhishek Tiwari, and Vijay Kakade

Abstract The use of a bituminous overlay for the extension of the service life of the pavement is the most common practice in India. Reflection cracking is a major type of distress observed in overlays constructed with the bituminous mix. Generally, the reflection cracking in a bituminous overlay is generated due to repeated traffic load, thermally induced stresses or strains, and a combination of both. The aging of the bituminous mix is also one of the causes of the formation of reflective cracks in the overlay. However, very limited studies are available on the effect of aging on the reflective cracking resistance of bituminous mix. Therefore, in this study, the effect of short-term aging and long-term aging on reflective cracking resistance of bituminous mixes prepared with viscosity grade bitumen and polymer-modified bitumen were evaluated in Texas Overlay Tester (TOT). The results of the test indicate that the resistance of bituminous mixes to reflective cracking was reduced with an increase in the severity of aging. The bituminous mix prepared with polymer-modified bitumen showed better resistance to reflective cracking caused by aging.

Keywords Reflective cracking · Short-term aging · Long-term aging · Texas overlay tester

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1 Introduction

India has the second-largest road network in the world. However, the premature failure of roads due to extreme climatic conditions and overloading is more common in most parts of India. Therefore, the provision of overlay becomes necessary to increase the life of the prematurely failed roads to its design service life. Due to the low cost of construction, the hot bituminous mix is more commonly used for the provision of overlay over prematurely failed flexible and concrete pavements. The provision of overlays will help in improving in structural and functional performance of the roads. In addition to this, the provision of the overlay will also help to prevent the intrusion of water into the pavement. However, the bituminous overlays are also susceptible to reflection cracking. Reflection cracking is the crack propagated to newly laid bituminous overlay from underlying joints/cracks [1]. The reflection cracking in a bituminous overlay is caused by (1) repeated traffic loading, (2) thermally induced stresses or strains, and (3) a combination of both [2]. The formation of reflection cracks in the bituminous overlay will result in an increase in maintenance costs and a reduction in the service life of the overlay. Apart from the above-mentioned causes, the limited attempt made by Mohammadafzali et al. [3] indicates that the aging of the bituminous mix is also one of the causes of reflection cracking in a bituminous overlay. Mohammadafzali et al. [3] evaluated the effect of aging on the resistance of bituminous mix prepared with 100% RAP to reflection cracking by using the Texas Overlay Tester (TOT). The TOT was conducted on the control mix (100% RAP-modified mix) and 100% RAP-modified mixes prepared with heavy paraffinic distilled solvent extract (HPE) and a water-based emulsion from wax-free naphthenic crude (CWE) rejuvenators. The test was conducted on the unaged and accelerated aged bituminous mixes. The accelerated aging of the bituminous mixes was conducted using the accelerated pavement weathering system where specimens were kept for a period of 1000 and 3000 h. The results of the TOT test indicate that with an increase in the accelerated aging period, the average number of cycles to failure was reduced. The RAP-modified mix prepared with a CWE rejuvenator showed the least reduction in the average number of cycles to failure among all the mixes. The performance of the RAP-modified mix prepared with the HPE rejuvenator was better than that of the control mix. From the limited attempts made in the past, it is clear that aging has a significant effect on reflection cracking in the overlay. Therefore, in this study, an attempt has been made to investigate the effect of aging on reflection cracking in a bituminous mix prepared with viscosity grade binder (VG30) and polymer-modified binder (PMB).

2 Experimental Details

2.1 Marshall Mix Design

Optimum binder contents for bituminous mixes prepared with different binders were obtained by following the Marshall mix design method. The Marshall mix design was conducted by following the Asphalt Institute Manual MS-2. Specimens of 100 mm diameter and 63 ± 2 mm height were prepared by applying 75 blows of Marshall hammer on both the faces of the specimen. Marshall stability test was conducted at 60 °C. Binder content corresponding to 4% air voids content was selected as optimum binder content (OBC).

2.2 Texas Overlay Test

The Texas overlay tester was used to evaluate the resistance of bituminous mixes to reflective cracking. To simulate the field conditions, the Texas overlay test was conducted on the short-term oven-aged and long-term oven-aged specimens. The test was conducted in accordance with the Tex-248-F. The specimens of 150 mm in diameter and 62 ± 0.2 mm in height were prepared by using the gyratory compactor. All the specimens were compacted at 7 ± 1% air voids. From top and bottom, the specimens are trimmed to get specimens of 38 ± 3 mm in height. The specimens were also trimmed from both sides to get specimens of 76 mm width. The trimmed specimen is shown in Fig. 1. Before testing the specimen was dried in an oven maintained at 40 °C for 24 h after completion of the drying period, the specimen was attached to base plate using Araldites adhesive. As shown in Fig. 2, a 4.5 kg weight was placed on the specimen to ensure proper adhesion between the specimen and the base plate. This test was conducted at 25 °C. Figure 3 shows the specimen kept in a chamber of a Texas overlay tester. The TOT conducted on specimens prepared with different aging conditions was continued until the 93% reduction in maximum load was measured in the first opening cycle. Three specimens were tested for each mix in different conditions like usage, short-term aging, and long-term aging. The resistance of bituminous mixes to reflective cracking was evaluated using the average number of cycles to failure, critical fracture energy (CFE), and crack propagation rate (CPR) obtained from the Texas overlay test. The average number of cycles to failure indicates the number of cycles at which the maximum load measured from the first opening cycle drops to 93% or more. The critical fracture energy indicates the energy required to initiate a crack in a bituminous mix [6]. The higher value of critical fracture energy indicates a better resistance to cracking [6]. The crack propagation rate is defined as the reduction in load required to propagate cracking under the cyclic loading condition of the overlay test. The higher absolute value of the crack propagation rate indicates a shorter reflective cracking life [7].

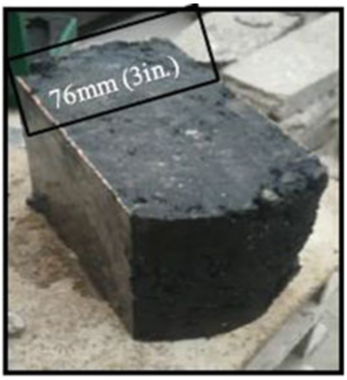

Fig. 1 Trimmed specimen

Fig. 2 Trimmed specimen glued to plate

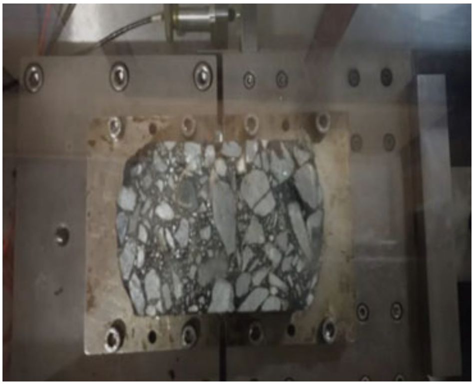

Fig. 3 Specimen placed in a TOT chamber

Table 1 Physical properties of aggregates

Property	Value	Specification (as per MoRTH, 2013)
Aggregate impact value (%)	17.71	Max. 24
Los Angeles abrasion value (%)	24.1	Max. 30
Flakiness and elongation index (%)	19.0	Max. 30
Water absorption (%)	1.15	Max. 2.0

2.3 Aging of Bituminous Mixes

The short-term oven aging (STOA) and long-term oven aging (LTOA) of bituminous mixes were performed by following the AASHTO R30. To simulate the short-term aging in the laboratory, the loose bituminous mix prepared with optimum binder content was kept in a forced draft oven maintained at 135 °C for a period of 4 h. The loose mix was stirred every 60 min to ensure uniform aging of the loose mix. After completion of the aging period of 4 h, the loose mix was then heated up to the compaction temperature of the bitumen and then compacted using a gyratory compactor. The long-term aging of the bituminous mixes was simulated by keeping the compacted bituminous mixes prepared after short-term aging in a forced draft oven maintained at 85 °C for 5 days.

3 Materials

Details of different materials used in the present study are given below.

3.1 Aggregate

The physical properties of aggregates used in the present study are given in Table 1.

3.2 Bitumen

The basic properties of viscosity grade bitumen (VG30) and SBS polymer-modified bitumen (PMB) used in the present study are given in Tables 2 and 3, respectively. The dynamic shear rheometer was used for the estimation of the complex modulus of SBS-modified bitumen. The complex modulus was estimated by following the

Table 2 Properties of viscosity grade bitumen (VG30)

Penetration @ 25 °C, 100 g, 5 s, 0.1 mm	62
Softening point, °C	55
Absolute viscosity at 60 °C , Poise	2870
Kinematic viscosity at 135 °C , cSt	520
Tests on Bitumen residue from rolling thin film oven test	
Viscosity ratio at 60 °C	2.88
Ductility at 25 °C, cm	85

Table 3 Properties of polymer-modified bitumen (PMB)

Penetration @ 25 °C, 100 g, 5 s, 0.1 mm	45
Softening point, °C	62
Elastic recovery of half thread in ductilometer at 15 °C, percent	71
Separation difference in softening point, °C	2
Complex modulus (G*/sinδ) as Min. 1.0 kPa at 10 rad/s, at a temperature, °C	86
Viscosity at 150 °C, Poises	6.7
Tests on Bitumen residue from thin film oven test	
Loss in weight, %	0.1
Increase in softening point, °C	3
Reduction in penetration of residue, at 25 °C, %	30
Complex modulus (G*/sinδ) as Min. 2.2 kPa at 10 rad/s, at a temperature, °C	87

ASTM D7175-15. The viscosity grade bitumen (VG30) and SBS-modified binder satisfy the minimum requirements provided in IS 73 (2013) and IRC SP 53: 2010, respectively.

3.3 Aggregate Gradation

The bituminous concrete 1 (BC-1) gradation given in MoRTH 2013 was used for the preparation of bituminous mixes. The aggregate gradation adopted in the present study for the production of bituminous mixes is given in Table 4.

Table 4 Aggregate gradation

IS sieve size (mm)	Cumulative % weight passing by weight of total aggregate	
	Prescribed range	Adopted value
26.5	100	100
19	90–100	91.44
13.2	59–79	68.59
9.5	52–72	55.16
4.75	35–55	39.18
2.36	28–44	35.99
1.18	20–34	26.16
0.600	15–27	20.71
0.300	10–20	14.10
0.150	5–13	7.98
0.075	2–8	2.32

4 Results and Discussions

4.1 *Marshall Mix Design*

The values of Marshall mix design parameters corresponding to 4% air voids are given in Table 5.

4.2 *Texas Overlay Test*

The average number of cycles to failure obtained for both the mixes in the different aging conditions is given in Fig. 4.

From Fig. 4 it can be seen that for both the mixes the average number of cycles to failure decreased with an increase in the severity of aging. This indicates that aging causes the reduction in resistance of bituminous mixes to reflection cracking.

Table 5 Marshall mix design parameters

Properties	Type of mix	
	VG30	PMB
Stability (kN)	14.8	16.5
Flow (mm)	3.9	3.7
VMA (%)	13.1	15.2
VFB (%)	68	70
OBC	4.75	4.85

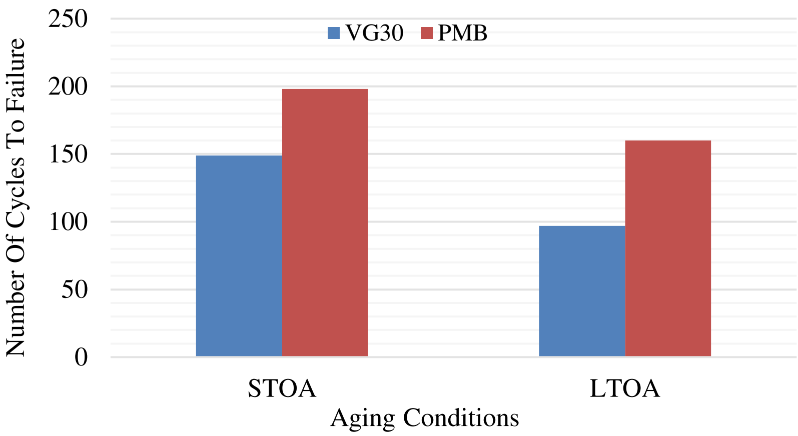

Fig. 4 Average number of cycles to failure in different aging conditions

However, the reduction in the average number of cycles to failure due to long-term aging is less in bituminous mix prepared with polymer-modified binder than that of viscosity grade binder. In the VG30 mix, the long-term aging caused a 34.9% reduction in the average number of cycles to failure. However, in the case of the PMB mix, the reduction in the average number of cycles to failure due to long-term aging was 19.2%. This indicates that VG30 bitumen was more susceptible to aging as compared to PMB. The resistance offered by SBS polymer to the formation of carbonyl and sulfoxide compounds resulted in less aging of the PMB mix than that of the VG30 mix.

The values of crack propagation rate (CPR) and critical fracture energy (CFE) of different mixes were obtained by testing the mixes after short-term and long-term age shown in Figs. 5 and 6, respectively.

From Figs. 5 and 6, it can be seen that with an increase in the severity of aging, the CPR of the mixes increases whereas CFR decreases. Significant hardening of bitumen due to aging resulted in an increase in CPR and a decrease in CFR with an increase in the severity of aging. This indicates that long-term aging causes more reduction in reflective cracking resistance of bituminous mix. After long-term aging, the CPR value of the PMB mix increased by 15.6%. However, the increase in CPR value due to the long-term aging of the VG30 mix is 36.2%. The reduction in CFR value due to long-term aging in VG30 and PMB mixes is 73.4% and 42.7%, respectively. The formation of oxidation products due to aging is less in SBS-modified binder as compared to VG30 binder. Because of that during the aging process, the SBS-modified binder becomes less stiff as compared to VG30 bitumen. Due to this, the bituminous mix prepared with PMB has a lower crack propagation rate in both aging conditions. In addition to this, the lesser stiffening of the PMB mix resulted in an increase in critical fracture energy to produce cracking in aged PMB mixes. However, the significant hardening of VG30 bitumen due to aging caused the cracking in the VG30 mix at lower critical fracture energy.

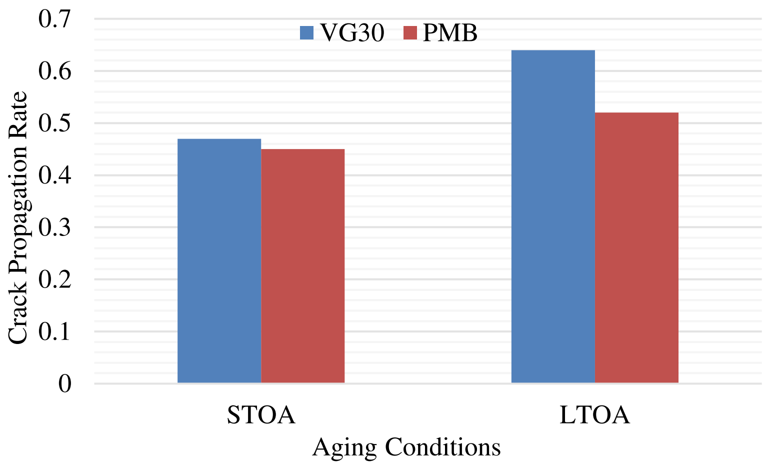

Fig. 5 Crack propagation rate of bituminous mixes at different aging conditions

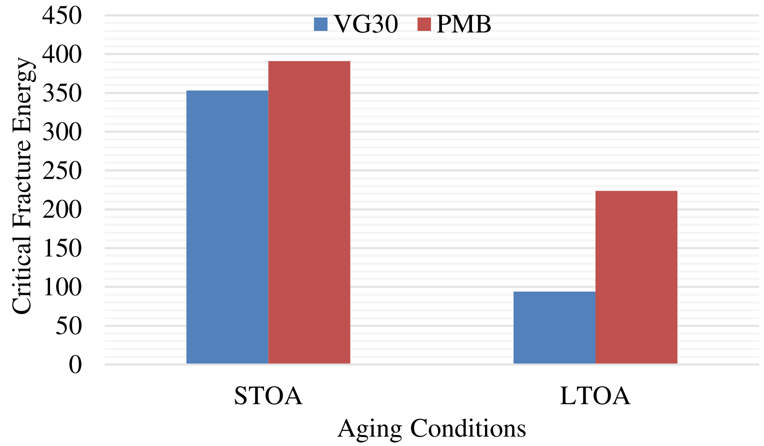

Fig. 6 Critical fracture energy of bituminous mixes at different aging conditions

Garcia et al. [6] concluded that the cracking resistance of the asphalt mix can be well understood by the design interaction plot. Thus, in this study, a design interaction plot of bituminous mixes based on CFE and CPR parameters is shown in Fig. 7.

From Fig. 7 it can be seen that short-term aged mixes require more critical fracture energy to initiate the crack as compared to long-term aged mixes. In addition, the higher crack propagation rate of short-term aged mixes compared to long-term aged mixes indicates that the flexibility of bituminous mixes is reduced with an increase in the severity of aging. Apart from this, the crack propagation rate of short-term aged mixes was less than the failure limit of 0.5 suggested by Garcia et al. [6] for the differentiation of mixes having poor and well resistance to cracking. This

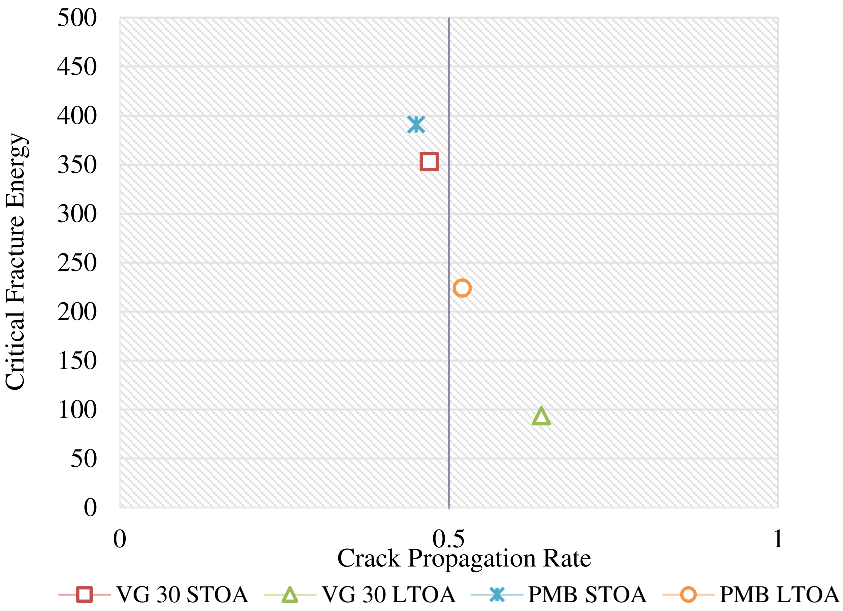

Fig. 7 Cracking performance of bituminous mixes on design interaction plot

indicates that short-term aged mixes have good resistance to cracking. However, long-term aged mixes have a crack propagation rate higher than 0.5, which indicates that long-term aging caused a significant reduction in the cracking resistance of bituminous mixes. The long-term aged PMB mix was close to the failure criteria for crack propagation rate. The higher value of crack propagation rate of long-term aged bituminous mix prepared with VG30 bitumen indicates that VG30 mix has poor resistance to cracking compared to PMB mix. In addition to this, the lower value of critical fracture energy of the VG30 mix indicates that cracks are initiated at a lower energy in the VG30 mix compared to the PMB mix. This indicates that long-term aging caused a significant reduction in the cohesive/adhesive and bonding properties of VG30 bitumen.

5 Conclusions

In the present study, an attempt has been made to investigate the effect of aging on the reflective cracking resistance of bituminous mixes prepared with VG30 and PMB bitumen. The results of the Texas overlay test conducted on the short-term aged and long-term aged bituminous mixes indicate that long-term aging causes a significant reduction in the reflective cracking resistance of bituminous mixes. The bituminous mix prepared with polymer-modified bitumen has better resistance to reflective cracking in both aging conditions. Therefore, the use of polymer-modified bitumen is essential to reduce the severity of reflective cracking in overlays constructed with bituminous overlays.

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Ultraviolet Radiation Ageing of Asphalt: A Critical Review

Suhas Pandhwale, Adyasha Mohanty, Anush K. Chandrappa, and Vijayakrishna Kari

Abstract Ageing of asphalt is an inherent characteristic of the asphalt. During mixing, transportation, and compaction, asphalt undergoes short-term ageing, while during service life it is subjected to long-term ageing. Ageing of asphalt is simulated in the laboratory as thermo-oxidative ageing using a rolling thin film oven (RTFO) and pressurized ageing vessel (PAV). However, during service life, asphalt is subjected to UV radiations and oxygen, due to which, it gets oxidized.

This ageing is referred to as photo-oxidative ageing and is not reversible. India is subjected to severe UV radiations throughout the year, but there are no test methods available to assess the UV-ageing susceptibility of binders. In this article, a critical review has been carried out on the UV ageing of asphalt considering various factors including test methods, sample preparation, ageing mechanism, and influence of anti-ageing additives. The review concludes by providing a sound scope for future research and a brief introduction of the work being carried out by the authors in the domain of UV ageing of asphalt.

Keywords UV ageing · Asphalt · Anti-ageing additive

1 Introduction

Ageing of asphalt is an inherent characteristic of the asphalt, which is known to affect its long-term performance. Due to ageing, the asphalt undergoes changes at the molecular level, which alters its physical, chemical, and rheological properties [1]. Generally, the ageing increases the asphaltene content and reduces the content of relatively low-molecular weight compounds such as aromatics, saturates, and resins [2]. As the asphaltene content increases, the asphalt tends to become hard/stiff

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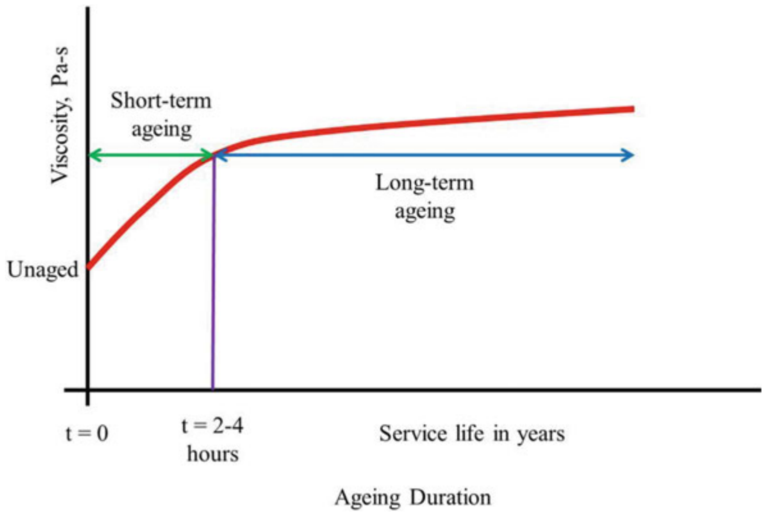

Fig. 1 Typical variation in viscosity of asphalt binder in asphalt mixture

resulting in a reduction in the strain-carrying capacity [2]. Due to this, the asphalt undergoes cracking, further leading to durability failure. In addition, the ageing of asphalt is also considered to be one of the contributory factors for the top-down cracking in asphalt pavements [3]. Asphalt binder ageing is generally considered as thermo-oxidative ageing, which occurs in two stages as shown in Fig. 1.

The first stage is referred as short-term ageing (STA), which occurs during the process of mixing, transportation, and compaction, where the temperature of the asphalt is more than 100 °C [4]. During the STA stage, the asphalt is maintained at a high temperature for 2–4 h depending on the field conditions. This stage results in excessive loss of volatile components due to high temperature as asphalt forms a thin cover on the aggregates [1]. The rate of degree of ageing is considered to be very high in this stage, as studies have shown almost a 1.5–2 times increase in viscosity compared to unaged condition [4]. The second stage is referred to as long-term ageing, which occurs during the service life of the asphalt pavement. The long-term ageing (LTA) has been considered to occur due to oxidation of the asphalt during the long-term exposure. The ageing of asphalt occurs slowly during long-term ageing owing to lower service temperature (25–65 °C). The STA and LTA are simulated in the laboratory using a rolling thin film oven (RTFO) and pressurized ageing vessel (PAV) as per ASTM D2872-19 and ASTM D6521-19 [5, 6]. Although LTA occurs due to oxidation owing to long-term exposure, it is simulated in the laboratory as thermo-oxidative ageing by applying high temperature and pressure, which does not occur during the service life of the pavement. The current ageing protocols do not give due importance to ultraviolet radiation ageing (UV ageing), which causes photo-oxidative ageing of the asphalt during its service life. Past studies have shown that the molecular composition of long-term aged asphalt in the field is better compared to the

Table 1 Global solar UVI category

Exposure category	Low	Moderate	High	Very high	Extreme
UVI	1–2	3–5	6–7	8–10	11+

UV-aged binder rather than the thermo-oxidative ageing of binder [7]. Therefore, the main objective of this review is to critically understand the mechanism of UV ageing, development in test methods, and methods to reduce the UV ageing in asphalt.

2 Importance of UV Ageing in the Indian Context

India is a tropical country, which receives abundant sunshine throughout the year, especially during the summer season. Accordingly, India is subjected to severe UV radiations, which is indicated by the UV index (UVI). UVI is a measure of the level of UV radiation present in a location. It varies from 0 to 11+ , where "0" indicates minimal exposure, while 11+ indicates severe exposure. The UVI in most parts of the country lies above 6, representing high UV exposure as shown in Table 1 and Fig. 2.

As seen in Fig. 2, most of the states in India have a UVI of more than 6, while a significantly high UVI is found in the southern part of the country. This clearly indicates that pavements are exposed to very high UV radiations and will result in photo-oxidative ageing of bitumen. Hence, there is a need to assess the UV-ageing susceptibility of commonly used binders in India and suggest corrective measures to improve the service life of the pavements.

3 UV-Ageing: Laboratory Investigation

3.1 UV-Ageing Mechanism

The UV radiation is divided into three categories as UV-A, UV-B, and UV-C based on the wavelength. The UV-C with a wavelength of 200–280 nm has severe damaging potential but is absorbed by the ozone layer. The UV-B has a wavelength between 280 and 320 nm, which is mostly absorbed by the stratosphere. Eventually, UV-A, having a wavelength of 320 to 400 nm reaches the earth's surface and is known to cause UV ageing. The main mechanism of UV ageing involves breaking the molecular bond and paving the way for the oxidation process. Each molecular compound is sensitive to a particular wavelength of light and when the energy of the absorbed photon is higher than the molecular bond strength, the bond is broken resulting in free radicals, which gets oxidized. The bond energies of commonly occurring molecular compounds found in asphalt are shown in Table 2. It can be seen from

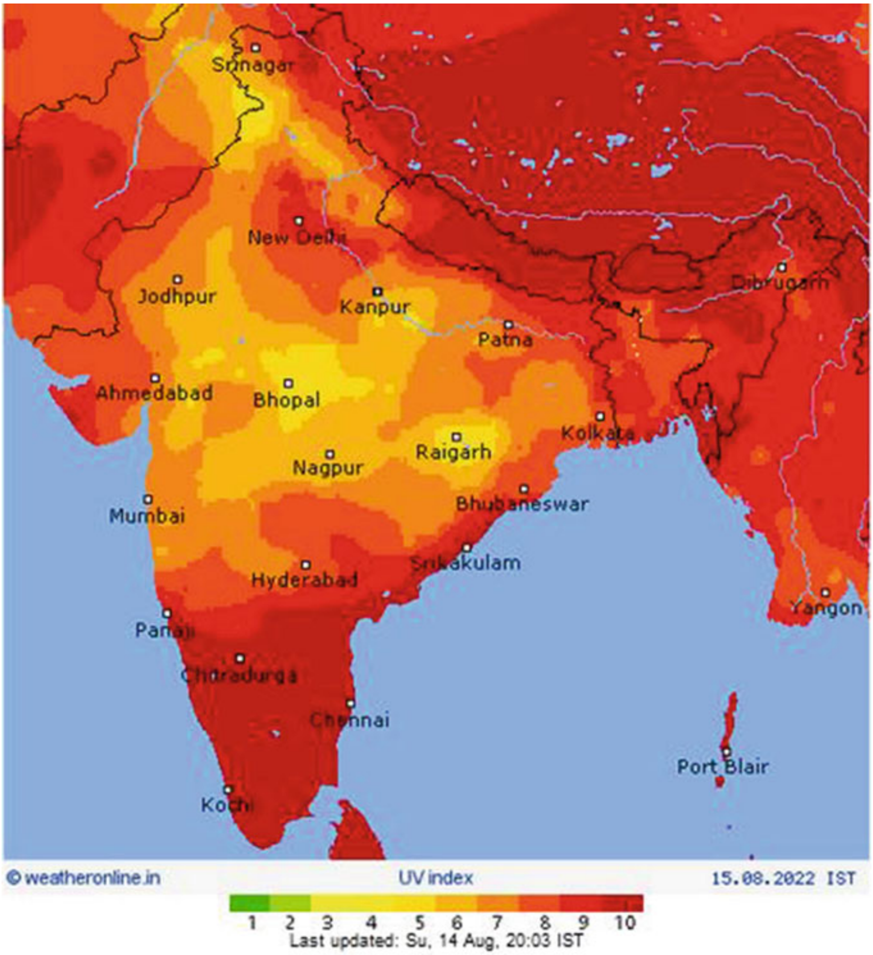

Fig. 2 UVI map of India [8]

Table 2 that the C-N and C = C compound possesses the lowest and highest bond energy, which indicates that C-N is easy to be broken, while C = C double bond requires more energy to rupture it.

The energy of the incoming photon can be calculated using Eq. 1.

Table 2 Bond Energies of Molecular Compounds in Asphalt [9]

Molecular bond	C-N	C–C	C–O	C = C	C–H	N–H	O–H
Bond energy, kJ/mol	290.9	347.9	351.6	615.3	413.6	389.3	463.0

Table 3 Energy of photon for different UV radiation wavelength

UV-A	320	340	360	380	400
Energy of photon, kJ/mol	373.84	351.78	332.29	314.73	299.08
UV-B	280	290	300	310	320
Energy of photon, kJ/mol	424.47	411.99	398.48	384.98	373.84

$$E = \frac{h * c}{\lambda} \tag{1}$$

E = Energy of photon, eV (1 eV = 96.486 kJ/mol); h = Planck's Constant; c = Speed of light, m/s; = λ Wavelength, nm.

Table 3 shows the energy of photon at various wavelengths for UV-A and UV-B radiations. It can be seen that, except C = C and O–H bond, all the other molecular bonds shown in Table 2 can be degraded by UV-A and UV-B radiations. Once the bond is ruptured, it is an irreversible process and hence gets oxidized leading to degradation of the asphalt.

3.2 UV-Ageing Test Methods

The UV-ageing test involves exposing a thin film of asphalt to UV radiations (UV-A/ UV-B) for a certain duration. The degree of ageing depends on the thickness of the asphalt film, the wavelength and intensity of the UV radiation, the temperature of the asphalt, and the duration of the ageing.

3.2.1 Asphalt Film Preparation Methods

The past studies have investigated the effect of thickness and various methods to attain thin film of asphalt. One of the simplest methods to obtain thin film in the range of 2–10 microns is the dissolution-evaporation method. Zeng et al. (2018) [10] used asphalt dissolved in carbon disulfide (CS2) at various concentrations to obtain asphalt film thickness of 2.5–15 μm. The solution was spin-coated on a quartz glass slide and the film thickness was determined using step profiler. Chen et al. (2020) [11] used an iron hoop of a height of 3 mm, where asphalt was poured on it and excess asphalt was scraped using a sharp knife. The 3 mm thick asphalt film was further divided into three 1 mm thick asphalt films to study the effect of UV-ageing gradient. Hu et al. (2018) [12] used KBr slides to prepare asphalt film of thickness around 10 microns. The asphalt was dissolved in CS2 and was poured into KBr slides, where CS2 evaporated leaving behind asphalt. Zeng et al. (2015) [13] used simple weight-volume calculations and obtained a film thickness of 1.2 mm by pouring 20 g of asphalt on an iron pan of diameter 150 mm. Sun et al. (2020) [14] proposed a new method to prepare thin asphalt samples using cryogenic slicing. The cube of asphalt conditioned in cryogenic refrigerator was sliced at a speed of 0.05 mm/second using a frozen slicer to obtain asphalt films of 3 microns thickness.

3.2.2 Effect of UV Wavelength, Temperature, and Duration on UV-Ageing

Zeng et al. (2015) [13] investigated the effect of temperature on UV ageing by considering temperatures of 30, 50, and 70 °C. Two binders, including unmodified and SBS-modified binder, were subjected to UV ageing at three temperatures and four ageing durations (24, 48, 72, and 96 h). The mass change ratio (%) before and after ageing indicated that the UV-ageing duration had a higher degrading effect than the temperature. In other words, UV-aged binder at 30 °C and 50 °C did not show a significant change in mass loss when aged for the same duration. Further, SBS-modified binder was also found to be very sensitive to UV-ageing indicating the polymer modified binder show lower resistance for UV-ageing, unlike thermos-oxidative ageing. Hu et al. (2018) [12] investigated the effect of UV wavelength considering UV-A, B, and C radiations by ageing 1.2 mm thick asphalt film for 16 days. Surprisingly, the chemical and rheological results indicated that UV-B radiation has significant damaging effects than UV-A and UV-C radiations. This can be mainly attributed to the absorption characteristics of asphalt as shown in Tables 2 and 3, where the energy of photon of UV-B was able to rupture most of the molecular bonds in asphalt. Li et al. (2021) [15] investigated the ageing gradient in the asphalt film. As only the surface of asphalt is exposed to UV radiations, the asphalt below the surface may not undergo ageing. However, this study found that UV-ageing deepens by first ageing the surface layer hardening it, which diffuses to the lower layer bringing the new asphalt on the surface. Based on the UV-ageing behavior, the study proposed two-stage ageing model, where the first 24 h are characterized by a power law, and then the ageing follows a linear model.

3.2.3 Effect of Anti-Ageing Additives on UV-Ageing Resistance

The ageing phenomenon associated with thermo-oxidative ageing and photo-oxidative ageing is different [16, 17] (Das et al. 2014; Wang et al. 2021). The thermo-oxidative ageing is reversible unlike phot-oxidative ageing. Therefore, the additives, which are known to reduce thermo-oxidative ageing, may not show a similar effect toward ageing due to photo-oxidation. The anti-UV ageing additives are generally categorized as UV-absorbents and UV-reflectors depending on their function. The UV-reflectors provide shielding effect through the optical interference phenomenon, while the UV-absorbers absorb the UV radiation and convert them into a lower energy state. Wang et al. (2021) [17] compared the anti-ageing effects of organic nano-clays modified with Cetyltrimethylammonium bromide (CTAB) and Polyethyleneimine (PEI). The study found that although PEI modified nano-clay depicted superior resistance toward thermo-oxidative ageing, its resistance toward UV-ageing was not much pronounced. Qian et al. (2020) [18] compared the anti-UV ageing effects of crumb rubber and nano-SiO_2. The results indicated that crumb rubber modified binder had less resistance against UV-ageing, while addition of nano-SiO_2 enhanced the UV-ageing resistance. This clearly indicates that although the addition of crumb rubber provides superior resistance against thermo-oxidative ageing, it is sensitive toward UV-ageing (Wang et al. 2020) [19]. Sun et al. (2022) [20] investigated the anti-UV ageing resistance of Acrylate-Styrene-Acrylonitrile (ASA) polymer. Although ASA polymer depicted adverse results compared to neat binder up to 152.5 h of ageing, after this, TGA results indicated ASA modified binder performed better than neat binder. Cheraghian and Wistuba (2020) [21] used a composite of clay and fumed silica nanoparticles in asphalt to investigate anti-UV ageing behavior. Based on the rheological, chemical, and morphological tests, it was found that composite modifier significantly enhanced the UV-ageing resistance, which can be mainly attributed to the reflective properties. The study suggested that the composite additive may function as a low-cost additive for asphalt to enhance long-term performance. Xu et al. (2019) [22] utilized layered double hydroxides (LDHs) intercalated with UV-absorbers. LDHs are layered structures, where anions are sandwiched between bi-metallic hydroxide ions. The study found that the addition of LDHs prevented the permeation of oxygen into the asphalt and also the energy of UV radiation was reduced due to multiple reflections within the structure of LDHs. A few studies have utilized nano-zinc oxide (ZnO) as an anti-UV ageing additive. Zhang et al. (2015) [23] found that the addition of nano-ZnO resulted in a decrease in the carbonyl index of asphalt highlighting its anti-ageing abilities. Similarly, Xu et al. (2019) [24] used ZnO as an anti-ageing additive in asphalt and found that the UV-absorption characteristic was highly enhanced after the addition of ZnO and recommended 3.0% by weight of asphalt as the optimum value.

4 Need for Future Research

Although the above studies have found that UV-ageing is detrimental to asphalt pavements long-term performance and recommend different anti-ageing additives, it is difficult to compare the results due to various inconsistencies in test methods. As each study uses different thicknesses of asphalt film and wavelengths for UV ageing, the results cannot be compared straightforward. Further, anti-ageing UV additives have different UV-absorption spectra and hence function differently depending on the wavelength of the incident UV light. In addition, UV-ageing simulation requires a very thin film of asphalt, which is challenging to obtain using general test methods. Moreover, thin films provide insufficient samples for rheological testing and hence several such specimens have to be produced to carry out rheological tests. Therefore, understanding the importance of UV-ageing, there is an immediate need to develop standardized test methods for preparing UV-ageing specimens and simulate UV-ageing in the laboratory, which can assist in comparing results using a common platform.

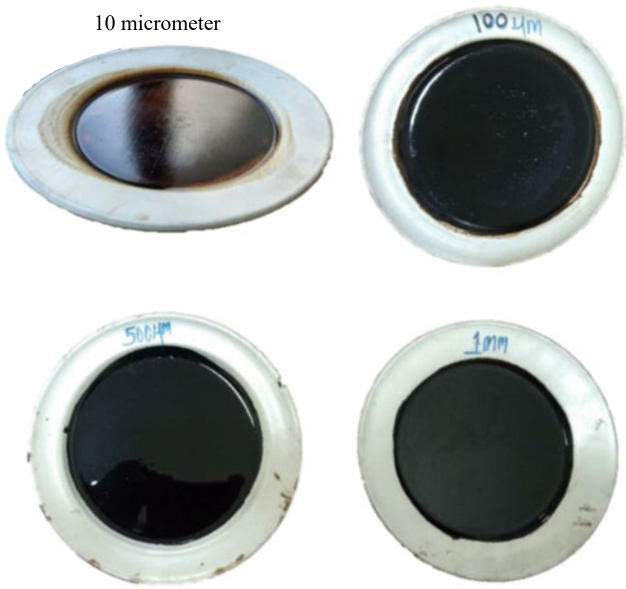

Fig. 3 Asphalt film of different film thickness

In the context of India, the authors could not find any study on UV-ageing susceptibility of commonly used binders such as VG30 and VG40. As India is subjected to severe UV radiations, it is necessary to investigate the UV-ageing susceptibility of these binders. In this regard, the authors have begun investigating the UV-ageing susceptibility of VG30 and VG40 binder by preparing test specimens of various thicknesses as shown in Fig. 3. As shown in Fig. 3, asphalt films of thicknesses 10, 100, 500, and 1000 μm were prepared. The 10- and 100-μm films were prepared using dissolution-evaporation method, while 500- and 1000-μm films were prepared using weight-volume calculations. It has to be noted that only 0.144 g of asphalt was used in preparing 10-μm film, which makes it challenging for rheological tests.

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Strength and Durability Characteristics of Stabilised Clayey Soil for Low Volume Roads

Samir Saurav and Sanjeev Sinha

Abstract Low volume roads play a crucial role in the rural road network since they link rural communities, provide trade networks, and provide access routes for a variety of functions. In this study, fly ash and cement were used to remediate an intermediate plastic clayey soil (CI) in different ratios of designed mix combinations. This soil stabilisation replaces the conventional method with lightly stabilised materials due to their rigidity, homogeneity, and impermeability, making it a practical option for usage on low-volume roads. In this study, the soil was stabilised with varying percentages of cement and fly ash in order to assess its potential for pavement layers. The UCS value increased by 35% from the initial untreated soil in some of the mix combinations. CBR strength, flexural and resilient modulus, and other important mechanical parameters for pavement design were also significantly improved. When compared to the conventional approach, the given mix combinations will save 20% on overall costs. In terms of resistance to wetting and drying cycles, mass loss is linear with durability cycles. Laboratory tests were used to assess the compaction characteristics. The findings concluded that changing existing natural materials can provide good strength and serve as a cost-effective replacement for conventional road construction methods.

Keywords Soil stabilisation · Cement-fly ash · Durability characteristics

1 Introduction

The Indian government has initiated new essential road construction programmes in order to revitalise the country's road industry, which has resulted in increased demand for road construction materials. According to Vittal et al. [1], the demand for various types of aggregates (stone chips, gravel, sand, etc.) for civil engineering construction operations in India has made the country one of the world's largest aggregates markets. Road construction is a significant user of aggregates and binder

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ingredients. If the use of locally accessible materials is to be promoted in India, specifications for alternative materials are necessary. However, it does not have to be identical to those of traditional materials, although this should be achieved as per Indian practices. While the Indian Roads Congress (IRC) [2–4] guidelines and codes for low traffic volume roads permit the use of marginal materials and stabilisation techniques for existing soil layers, the specifications for such materials in terms of gradation, plasticity, and CBR requirements have remained consistent with those for high-quality conventional construction materials. The objective of the study was to evaluate the suitability of clayey soil to be modified with cement-fly ash combination for use as sub-base materials. Soil was collected from locally available regions in the Ara district of Bihar, India. Further, the technical elements of applying stabilising techniques in road construction, current amendments made to the Ministry of Rural Road and Development (MoRD) [5] requirements, and research on locally available materials are highlighted. The IRC SP 89 [6, 7] for stabilisation and IS 4332, Part IV (1968) [8] for durability test standards, along with other IS Codes, have been sincerely followed for testing in this study.

This study aims to explore the application and mechanical properties of cement-fly ash (C-FA) stabilised materials in clayey soil. On roads with low volume, the suitability of modification to local resources in the sub-base and base layers of flexible pavements has been determined. Furthermore, the characteristics of durability were investigated in this study. The various mechanical parameters enable designers to determine pavement evaluation-appropriate stiffness/modulus values. The purpose of the investigation was to study behaviour after stabilisation in order to use it for pavement layers. Characterisation includes the evaluation of unconfined compressive strength (UCS), California Bearing Ratio (CBR) in a soaked state, and durability employing a thorough laboratory examination technique. Also, the flexural and resilient modulus of the existing mix combinations were determined.

1.1 Review of Earlier Work

In order to mitigate the problems associated with the use of the conventional approach, locally available waste materials, like fly ash, quarrying wastes, etc., can be effectively used singly or in combination with cement as an alternative to high-quality conventional materials [9]. The decision to use or reject these materials should only be made after a thorough engineering evaluation to check the effect on the strength, durability, and construction of pavement [10]. The rural roads, which have less traffic, are the right choice for using locally available materials. Mahent et al. [11] improved the granular sub-base for rural roads and found the use of existing plastic soil for other road construction materials like an embankment, subgrade, and hard shoulder by adding suitable engineering-property material. Although one of the most crucial parameters used to decide if stabilised materials are suitable for use in the sub-base or base layer of pavements is strength in terms of unconfined compressive strength (UCS) [12–14]. Often, the unconfined compressive strength is used to figure out how strong materials that are bound together are in pavements [12, 14, 15]. The main reason a cement-treated layer breaks is because of its flexural strength, so the flexural strength of specimens needs to be measured. Because flexural and tensile strengths are closely related, indirect tensile tests and flexural beam tests can be used to estimate the tensile strength of stabilised materials [16]. The combination of stone dust and fly ash addition at optimum moisture content to clayey soil was found to be suitable because it reduces swelling and increases strength [17]. Similar to other studies on additive selection, the combination of cement and fly ash in a designed ratio is more suitable for stabilisation [18–21]. The durability characteristics of stabilised materials often have a reaction accompanied by a significant increase in volume due to the expansive forces [22]. This generated stress exceeds the tensile strength of the material built up, weakening and disintegrating the stabilised material. If the tensile strength developed in the material exceeds the expansive forces, no detrimental action occurs. Because of this, the fact that carbonation isn't always bad for the material makes sense. The performance of stabilised layers is governed significantly by their resistance to the effects of durability [4, 5, 23, 24]. The durability of pavement is the ability of the material to keep its shape and stability after being exposed to the effects of weathering for a number of years [23, 25]. The wet-dry (W–D) brushing test, which was formed by the Portland Cement Association in 1992 and is used in the U.S., Australia, and India, is the standard test for durability. The mass loss after twelve W-D cycles is used as a criterion for designing the durability of stabilised materials. Also, the amount of mass loss that is allowed in the base layer may very well be less than in the sub-base [24, 26]. Although the wetting–drying or freezing–thawing test is extensively used, it takes more than a month to perform due to the 12 cycles of wetting and drying required. Kai et al. [27] evaluated the durability of silty clay soil and found that the PCA manual's (1992) durability requirement (i.e. 7% mass loss) could not be met with less than 10.5% cement. George et al. [26] performed wet-dry durability testing on cement-treated soil in accordance with AASHTO T 135 and observed that all soils satisfied the PCA durability criteria of 5% cement in the soil–cement combination.

To summarise, the studies in the past have suggested that strength is an important parameter for evaluating the stabilised soil properties, which have been used traditionally for a long time. However the durability characteristics and the type of stabiliser used are changing from period to period, considering economy and suitability. Hence, this study bridges the gap by characterising the strength and, subsequently, the pavement performance by checking its durability. Initially, stabilisation techniques were used only for improving poor soils, but now the emphasis on stabilisation is to increase pavement layer strength and reduce the usage of virgin aggregates in the upper layers. The waste from thermal power plants, like fly ash, is used as stabilisers for pavement layer soils, but their suitability is confined to certain types of soil that have been determined. The aim of this research is to improve the mechanical properties of cement-fly ash (C-FA) stabilised clayey soil materials so that they can be used as sub-base layers on low-volume roads.

2 Materials

The soil region collected falls within 25°00” to 25°30” N and 84°15” to 84°45” E, bounded by the Son River in the east, the Dharmawati-Gangi rivers in the west, the Vindhyan Hills in the south, and the river Ganga in the north. The physical and engineering properties of soil have been shown in Table 1. Fly ash is a by-product of burning pulverised coal in electric power generating plants. It has been collected from the National Thermal Power Plant (NTPC). The cement used was OPC 43 Grade, which was available on the road construction site to evaluate existing work. The use of cement-fly ash in the soil in the presence of optimum water content. According to IRC SP 89 [6, 7] the use of a small amount of cement with Class F fly ash to strengthen. This combination of cement and fly ash (C-FA) has been used successfully in sub-base course stabilisation.

Table 1 Physical and engineering properties of soil

Property	Indian standard	Soil
Soil classification (USCS)	IS:2720 Part IV	CI
Specific gravity	IS 2386 Part III	2.750
Water absorption (%)	IS 2386 Part III	16.800
Liquid limit (%)	IS 2720 Part V	35.500
Plastic limit (%)		14.300
Plasticity index (%)		21.200
Gradation		
Gravel (%)	IS 2720 Part IV	0.000
Sand (%)		73.330
Silt (%)		24.700
Clay (%)		2.000
Maximum dry density (γ_d) in g/cc	IS 2720 Part VII	1.549
Optimum moisture content (%)	IS 2720 Part VII	22.800
UCS (Un-Treated Sample after 7-Day)—MPa	IS 2720 Part X	0.050
California bearing ratios (CBR) in %	IS 2720 Part XVI	3.580
Direct shear test Cohesion (C) in kN/m^2) Φ in (°)	IS 2720 Part XIII	3.800 38°
Permeability (m/day)	IS 2720 Part XVII	9.2×10^{-3}

3 Methodology

Since the soil has an intermediate plasticity, it is stabilised using fly ash and cement. The cement variation was kept between 2 to 8% by weight of total mass. The mixing combinations of cement to fly ash were kept at 1:2 to 1:4 in order to meet the threshold limit as per IRC SP 89 Part I (2010) [6]. The pavement was designed as per the guidelines of IRC SP 72 (2015) [2]. In the present study, untreated soil and fly ash–cement treated in different combinations have been accessed, as shown in Table 2. The pavement thickness was adopted at 250 mm as per IRC SP 72 (2015) [2] based on CBR strength due to the stabilisation of the sub-base, which helps save natural resources (bitumen and aggregates) and cost for overall pavement construction.

To characterise the soils, laboratory testing was carried out. The testing was carried out at NIT Patna's Transportation Laboratory. The physical properties, compaction characteristics, unconfined compressive strength, CBR, durability, flexural, and resilient moduli of soil samples treated and untreated with stabilisers were evaluated using a variety of experiments. To ensure consistency across the prepared samples, the collected soil was sieved by gradation and then rearranged in a mix proportion that closely matched the parent material's gradation. In order to generate uniform samples for all interconnected tests, the grading of reconstituted samples remained constant throughout the testing procedure. According to the Indian Standard and MoRTH guidelines, the gradation and fluidity of these soils indicate that both cement and fly ash can be used for stabilisation. To analyse the compaction characteristics in line with IS 2720 Part VII, tests were performed on untreated soil and treated soil mixed with different combinations as shown in Table 2. Five cylindrical specimens (100 mm in diameter and 50 mm in height) at each mix combinations were fabricated for UCS testing (IS 2720 Part X) after 7 days, 14 days, 28 days, 90 days, and 120 days. Similarly, four cylindrical specimens (100 mm in diameter and 50 mm in height) at each mix combinations were fabricated for durability testing (IS 4332 Part IV (1968)) under the wetting and drying method. The CBR test was conducted according to Indian Standard 2720 (Part XVI) on different mix combinations and an untreated sample. The flexural strength tests have been conducted based on IS

Table 2 Designed combination of soil

Designed combination	Cement (%)	Fly ash (%)	Soil (%)
Un-treated	0	0	100
2C + 4FA	2	4	94
4C + 8FA	4	8	88
6C + 12FA	6	12	82
8C + 16FA	8	16	76
4C + 16FA	4	16	80
5C + 20FA	5	20	75
6C + 24FA	6	24	70
8C + 32FA	8	32	60

4332 (Part VI) 1972 [28]. The resilient modulus is determined by a tri-axial testing instrument. A cylindrical test specimen undergoes repeated axial cyclic stress and is exposed to static confining stress (air) from a tri-axial pressure chamber as per NCHRP Project 1-37A.

4 Results and Discussion

4.1 Influence of Strength and Durability on Stabilised Materials

The liquid and plastic limit variations, as well as the plasticity index, varied significantly at each mix combination, as shown in Fig. 3 and tabulated in Table 3. Similarly (Fig. 1), the variations in optimum moisture content (OMC) and maximum dry density (MDD) were at each of the mix combinations shown in Figs. 2 and 3. The decrease in optimum moisture content with an increase in cement and fly ash combination and an increase in the maximum dry density, respectively. The UCS values with respect to 7-day strength were increased from 0.050 MPa (untreated sample) to 1.869 MPa (treated sample) for the highest mix combination, as shown in Fig. 4. Also, the UCS values were increased up to 120 days further at different mix combinations with respect to 7-Day strength, as tabulated in Table 4. The CBR value of untreated soil was 3.580%, which increased to 42.080% as tabulated in Table 6. The variations in CBR for each mix combination are shown in Fig. 5. The results obtained for resilient and flexural modulus have been tabulated in Table 6. The variations obtained for each mix combination have been shown in Figs. 6 and 7. As per IRC 37 (2018) [29], the recommended UCS value for low-volume roads is 1.70 MPa for the sub-base. Similarly, as per the National Cooperative Highway Research Programme (NCHRP) 789 report, the recommended resilient modulus is 15–136 ksi (103–938 MPa) and the flexural modulus is 74–180 ksi (510–1241 MPa). Hence, the experimental results of strength as recommended were consistent.From the present study, it was found that the wetting and drying of stabilised soil samples for analysis of durability behaviour were consistent at each mix combination satisfying IRC SP 89 Parts I and II [6, 7] and resulted in a decrease of not more than 14%. This confirms that the strength is less sensitive to moisture levels even after 12 cycles of curing, although it was moulded at OMC, unlike cement concrete. Therefore, the suitability of a stabilised soil sample based on the wetting–drying cycle should be conservatively designed because the decrease in strength of a stabilised soil sample investigation is significant. As per IRC SP 89 (2010) and IRC 37 (2018), the criteria for volume change is restricted to 20% for base and 30% for sub-base layer materials. The result in Table 5 for each mix combination of treated soil specimens supports the stability of the strength values as it holds for the complete 12 cycles. The three characteristics, namely, change in volume, change in moisture, and soil–cement loss, define the specimen's stability in relation to its performance (Table 6).

Table 3 Tests result in a different combination of a mix

Notation	Designed combination	MDD in g/cc	OMC in %age	LL in %age	PL in %age	PI in %age
0	Un-treated	1.549	22.80	35.50	14.30	21.20
1	2C+4FA	1.670	15.50	35.80	18.40	17.40
2	4C+8FA	1.720	16.00	35.92	16.80	19.12
3	6C+12FA	1.770	17.00	36.30	16.40	19.90
4	8C+16FA	1.820	18.00	36.50	15.40	21.10
5	4C+16FA	1.720	16.00	35.85	18.52	17.33
6	5C+20FA	1.740	17.00	35.91	18.25	17.66
7	6C+24FA	1.780	18.00	36.80	17.48	19.32
8	8C+32FA	1.910	19.00	37.10	16.28	20.82

Fig. 1 MDD test results

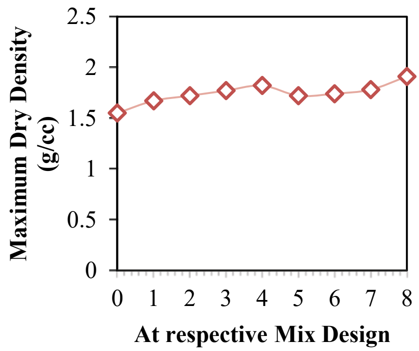
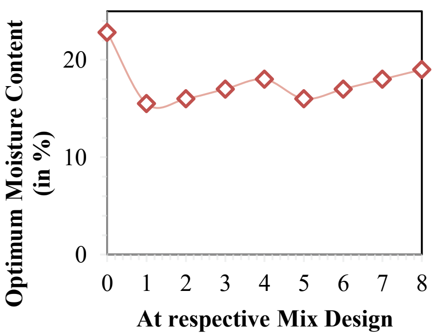


Fig. 2 OMC test results



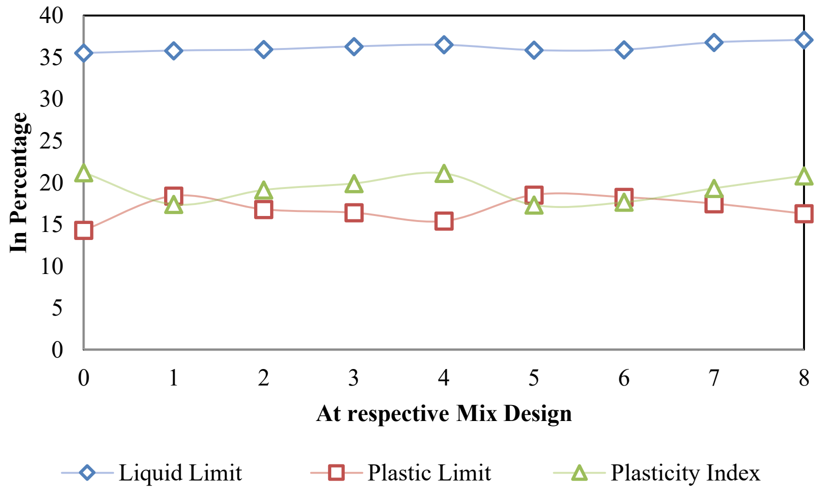


Fig. 3 The liquid limit, plastic limit, and plasticity index test results

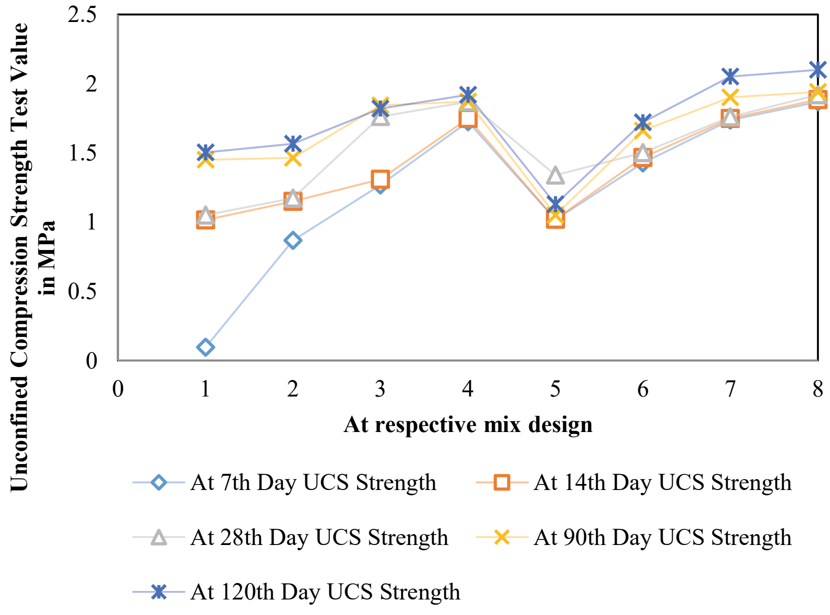


Fig. 4 UCS tests results

Table 4 Strength tests result from different combinations of a mix

Designed combination	UCS in MPa				
	7th Day	14th Day	28th Day	90th Day	120th Day
Un-Treated	0.050	NA	NA	NA	NA
2C + 4FA	0.096	1.016	1.050	1.451	1.502
4C + 8FA	0.867	1.150	1.172	1.464	1.565
6C + 12FA	1.267	1.131	1.763	1.842	1.820
8C + 16FA	1.725	1.748	1.868	1.870	1.918
4C + 16FA	1.018	1.021	1.338	1.051	1.128
5C + 20FA	1.426	1.465	1.500	1.660	1.720
6C + 24FA	1.736	1.748	1.760	1.900	2.050
8C + 32FA	1.869	1.885	1.920	1.940	2.100

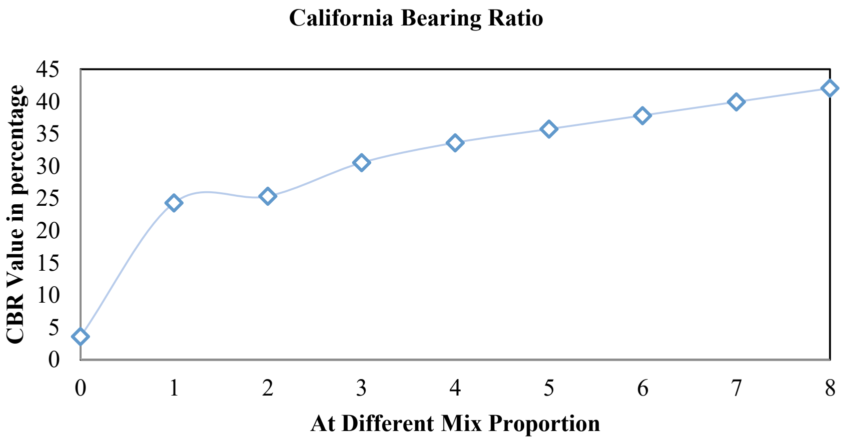


Fig. 5 The CBR test results

4.2 Rate Analysis

The comparative rate analysis for each mix combination has been shown in Table 7. Figure 8 illustrates the cost variation. The comparison was taken into consideration with the Conventional Approach to Grading I. The combination of 8% cement and 32% fly ash was more expensive than the conventional approach, whereas the remaining combinations resulted in significantly lower road construction costs. The road length was 1 km, single lane width of 3.5 m, and the thickness was 250 mm, which were both appropriate as per IRC SP 72 (2015).

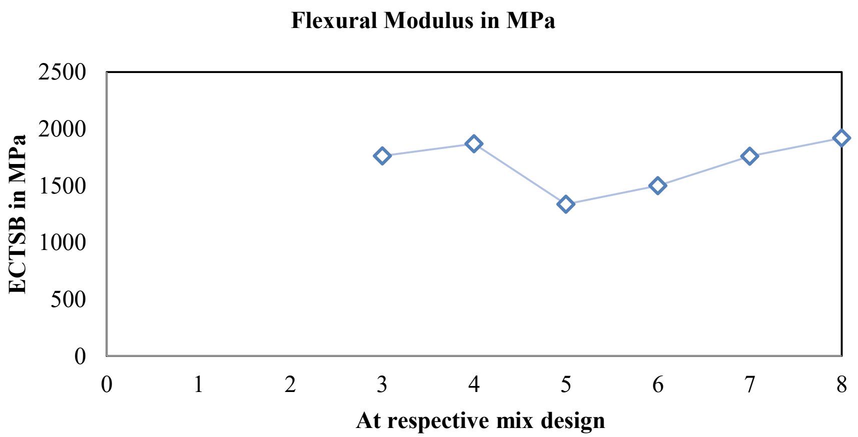


Fig. 6 The flexural modulus test results

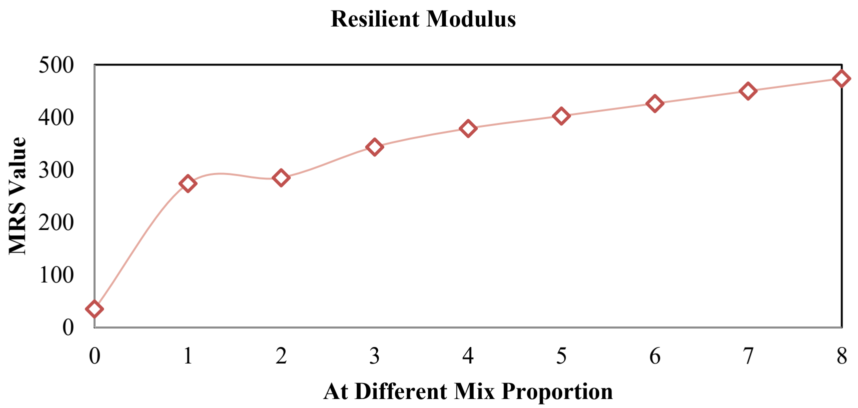


Fig. 7 Resilient modulus test results

Table 5 Durability tests result from different combinations of a mix

Designed combination	As per IS 4332 (Part IV) 1968	
	Moisture change in %age	Soil–cement loss in %age
Un-treated	NA	NA
2C + 4FA	0.096	1.016
4C + 8FA	0.867	1.150
6C + 12FA	1.267	1.131
8C + 16FA	1.725	1.748
4C + 16FA	1.018	1.021
5C + 20FA	1.426	1.465
6C + 24FA	1.736	1.748
8C + 32FA	1.869	1.885

Table 6 Strength tests result from different combinations of a mix

Designed combination	Soaked condition	After 28th day	
	CBR in %age	Flexural modulus in MPa	Resilient modulus in MPa
Un-treated	3.58	NA	35.8
2C + 4FA	24.322	NA	273.963
4C + 8FA	25.336	NA	285.385
6C + 12FA	30.534	1763	343.935
8C + 16FA	33.640	1868	378.921
4C + 16FA	35.750	1338	402.688
5C + 20FA	37.860	1500	426.455
6C + 24FA	39.870	1760	450.222
8C + 32FA	42.080	1920	473.990

Table 7 The comparative rate analysis of each mix combination adopted in this study

Type of mix	Component materials												
	labour (@Rs. / day for 1 km of pavement layer)		Machinery (@Rs. / per hour for 1km of pavement layer)		Cement (@Rs. 5573.25/tonne)		Fly Ash (@Rs. 2000/tonne)		Coarse graded GSB material (Rs. 629.05/ cum)		Water (@Rs. 201.20/kl)		Total cost (in Rs.)
	Rate per day for 1cum (in Rs.)	Cost incurred for 1km	Rate per hour for 1cum (in Rs.)	Cost incurred for 1km	Quantity (in tonne for 1km Length)	Cost incurred	Quantity (in tonne for 1km Length)	Cost incurred			Quantity (in kl for 1km length)	Cost incurred	
2% Cement + 4% FA	10.39	12,592.68	57.03	69,120.36	42.42	236,417.3	84.84	169,680	Nil	Nil	121.2	24,385.4	487,810.305
4% Cement + 8% FA	10.39	12,592.68	57.03	69,120.36	84.84	472,834.5	169.68	339,360	Nil	Nil	121.2	24,385.4	893,907.57
6% Cement + 12% FA	10.39	12,592.68	57.03	69,120.36	127.26	709,251.8	254.52	509,040	Nil	Nil	121.2	24,385.4	1,300,004.84
8% Cement + 16% FA	10.39	12,592.68	57.03	69,120.36	169.68	945,669.1	339.36	678,720	Nil	Nil	121.2	24,385.4	1,706,102.1
4% Cement + 16% FA	10.39	12,592.68	57.03	69,120.36	84.84	472,834.5	339.36	678,720	Nil	Nil	121.2	24,385.4	1,233,267.57
5% Cement + 20% FA	10.39	12,592.68	57.03	69,120.36	106.05	591,043.2	424.2	848,400	Nil	Nil	121.2	24,385.4	1,521,156.2
6% Cement + 24% FA	10.39	12,592.68	57.03	69,120.36	127.26	709,251.8	509.04	1,018,080	Nil	Nil	121.2	24,385.4	1,809,044.84

(continued)

Table 7 (continued)

Type of mix	Component materials												
	labour (@Rs. / day for 1 km of pavement layer)		Machinery (@Rs. / per hour for 1km of pavement layer)		Cement (@Rs. 5573.25/tonne)		Fly Ash (@Rs. 2000/tonne)		Coarse graded GSB material (Rs. 629.05/ cum)		Water (@Rs. 201.20/kl)		Total cost (in Rs.)
	Rate per day for 1cum (in Rs.)	Cost incurred for 1km	Rate per hour for 1cum (in Rs.)	Cost incurred for 1km	Quantity (in tonne for 1km Length)	Cost incurred	Quantity (in tonne for 1km Length)	Cost incurred			Quantity (in kl for 1km length)	Cost incurred	
8% Cement + 32% FA	10.39	12,592.68	57.03	69,120.36	169.68	945,669.1	678.72	1,357,440	Nil	Nil	121.2	24,385.4	2,384,822.1
Conventional approach (Grading I)	14.04	17,016.48	145.304	176,108.45	Nil	Nil	Nil	Nil	2981.52	1,875,525.156	72.72	14,631.3	2,068,650.08

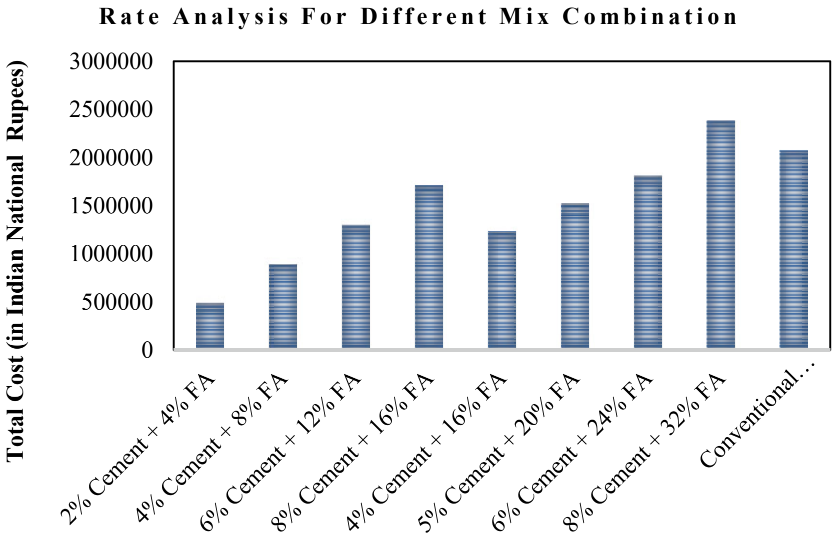


Fig. 8 Rate analysis

5 Conclusion

Infrastructure development, especially road construction, is important for the nation. Rural roads are expected to be built under the NHDP and PMGSY. Locally available soil stabilised with cement-fly ash meet the gradation and plasticity requirements of IRC SP-89 [6, 7]. The stabilised material has sufficient strength, as obtained from a combination of 6% and 8% cement, 16%, 32%, and 24% fly ash, and 60%, 72%, and 76% soil (i.e. combinations 4, 7, and 8). The CBR test also shows the highest percentage gain from 3.58 to 33.64 and 39.97 to 42.080 in this study. Further, as per IS 4332, Part IV (1968) [8], the obtained durability results for moisture change and soil–cement loss satisfy the base and sub-base layer for all the stabilised samples. The basic properties of stabilised materials like liquid, plastic, MDD, and OMC after treatment have also been improved with the current stabilisation techniques. The cost optimization has also been improved, as it reduces the cost compared to conventional methods.

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Transportation Planning, Policy and Economics

Sustainability Integration Index of Metro and Buses for Evaluation of Transport Policies

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Abstract With rapid urbanization, major Indian cities are facing issues with traffic congestion, air pollution, etc. Despite having a well-established public transportation infrastructure, including metro rail and buses, Delhi experiencing a large increase in private vehicle ownership which might be due to poor operational characteristics of public transportation and their integration. In this direction, expanding public transportation service options will encourage commuters to utilize these modes, and in that scenario, a bus might serve as a feeder to the primary transit like metro. This situation should enable commuters to travel seamlessly across public transportation modes to reach their destination. To do so, the integration between these modes needs to be studied holistically considering sustainability and improving integration accordingly. Some researchers have developed methods to estimate the sustainability integration index (SII) to study prevailing integration levels considering certain parameters. However, in the present study, an attempt has been made to consider 15 relevant parameters to estimate SII more realistically. The proposed method has been developed considering the data of user perception, expert opinion, bus and metro operation from three metro stations namely Ashram, Dwarka Sector-10 and Kashmere Gate in Delhi. The study found that integration between metro rail and buses in terms of SII (out of 100) is about 61, 50 and 55 at Dwarka Sector 10, Ashram and Kashmiri Gate, respectively. The developed method is also applied to evaluate relevant transport policies to demonstrate their suitability and utilization by authorities to implement appropriate policies to increase integration levels.

Keywords Sustainability integration index · Metro · Bus · Transport policies

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1 Introduction

1.1 Transportation Sustainability

In general, transportation sustainability or sustainable development is considered a system that is primarily used to determine the economic, environmental and social impacts on system effectiveness and efficiency [1]. The development and determination of indicator systems for sustainability assessment are largely based on methodologies that rely on the relationships of user perception and infrastructure with the economic, environmental and social parameters of the system.

Sustainable transportation must balance economic, social and environmental goals, also known as the triple bottom line. Although these indicate that each aim belongs to a distinct category, they frequently overlap. Air pollution, for example, is typically grouped under the environment domain, but it also has an influence on living conditions grouped under the social domain, as well as the economic domain.

1.2 Integration of Public Transportation Modes

Rising ridership and changing mode share contributions have been identified as a source of current benefits and the potential for future benefits [2]. A study by Jain et al. [3] found that safety is the most important criterion for considering a shift from cars, two-wheelers to buses followed by other parameters namely reliability, travel cost and comfort. This study also found that frequency and punctuality would also lead to commuter satisfaction. Under such better public transport service conditions, commuters were even willing to pay more and shift to public transport.

Integrating the public transport modes would increase the overall efficiency of the systems by enabling seamless travel by commuters across different modes. This would further enhance sustainability as commuters try to shift to bus or metro from their personal vehicles. Errampalli et al. [4] studied the need for the integration of public transport modes in order to enhance the integration level to increase ridership of public transport. The study proposes a methodology to estimate the Sustainability Integration Index (SII) considering the metro and bus in Delhi. By conducting various quantitative and qualitative surveys, a total of 12 indicators that help in determining the level of integration of metro and buses were proposed.

However, there can be some issues in one particular public transport system which can become a hindrance to improve the efficiency of the system and also affect integration with other public transport systems. For instance, improper commuter waiting behavior at bus stops and driver behavior in stopping buses, coupled with low capacity of bus stops, auto rickshaws or taxis waiting near bus stops, etc. These issues may result in inefficient operating conditions of public transport and congestion around bus stops along with safety [5]. Minimizing the discomfort while changing from one mode to another in the process of integration of the transportation system will ensure safe, smooth, and efficient and coordinated travelling. This sustainability integration of public transportation would attract commuters to use public transport [6].

In view of this, it can be seen from these studies that integration of public transportation modes can enhance the overall efficiency of the system and existing methods for estimating integration are focusing on limited parameters of public transport modes. There is a need for considering more related and suitable indicators to determine integration level in terms of SII particularly social and economic indicators like cost, bus behavior and amenities that users and commuters in the combined usage of metro and buses as public transportation indicators. These indicators help in analyzing sustainability integration between public transportation modes more realistically.

1.3 Objectives and Scope of the Study

The main objectives of the present study are:

- Develop a set of appropriate indicators for estimating the Sustainability Integration Index of public transportation modes
- Using the developed method, examine the proposed policies for optimizing metro and bus integration

The scope of this study considers three metro stations (Ashram, Dwarka Sector-10 and Kashmere Gate) in Delhi and proposes a methodology to evaluate transport policies by assessing SII.

2 Study Methodology

The main aim of this study is to analyse the current levels of integration amongst selected public transportation modes in terms of sustainability which has three domains: economic, social and environmental. As the study by Errampalli et al. [4] proposed a set of 12 indicators to assess the levels of integration between buses and metro, however, the present study identifies additional three more significant indicators in order to realistically assess the integration levels in terms of sustainability integration index (SII). These additional indicators are Transfer Cost, Seat Availability and Bus Stopping Behavior. These selected 15 indicators as given in Table 1 are collected from various surveys (Qualitative and Quantitative) using the Likert Scale concept and normalizing all of the indicators into a single utility value (U) ranging from 0 to 1. Expert opinion surveys were also conducted to assign weights to these indicators. The weighted individual value of each indicator is determined using Eq. (1). The total weighted and normalized value of all the variables makes up the sustainability integration index for public transportation modes as given in Eq. (2).

$$\text{Weighted Individual Value} = \mathrm{W_i} * \mathrm{U_i} \tag{1}$$

where $\mathrm{W_i}$ is the weight of the ith indicator$\mathrm{U_i}$ is the normalized utility value of the ith indicator

$$\mathrm{SII} = \sum_{i=1}^{n} \mathrm{W_i} * \mathrm{U_i} \tag{2}$$

where n is the total number of indicators.

3 Study Area and Data Collection

As mentioned in the scope of this study, three metro stations (Ashram as underground station, Dwarka Sector-10 as elevated station and Kashmere Gate as interchange station) in Delhi have been selected as given in Table 2. In order to assess the integration between metro and bus, the bus stops around metro stations are also appropriately considered. The user questionnaire has been designed to collect all the information on the indicators. A total of 1606 sample sizes have been collected from user perceptions surveys at these selected metro stations and bus stops. Apart from these, bus operational surveys namely reliability, speed and occupancy surveys have also been conducted at these bus stops.

An expert opinion survey was also conducted to determine the weights of each parameter by conducting a questionnaire survey among working professionals and researchers, in which they were asked to rank the indicators on the grounds of their necessity in measuring the current level of integration in terms of sustainability for the various modes of public transportation. A total of 50 sample size has been collected and these have been averaged to determine the final weights for the selected parameters as shown in Table 3. The final sustainability index value, which represents current levels of integration, is determined by aggregating all weighted individual indicators in the range of 0–100.

4 Estimation of Sustainability Indicators

Adopting the methodology described in the previous section, normalization of each indicator has been done from the collected data from the qualitative and quantitative surveys. Table 3 shows the normalized value of 15 indicators, which helps in determining the Sustainability Integration Index (SII) subsequently. The weightage of each indicator determined from the expert opinion survey is also given in Table 3. From Table 3, it can be seen that Safety and Security have been given high weightage from an integration point of view followed by productivity and accessibility.

Table 1 Indicators selected for the overall assessment of sustainability integration index

S. no	Name of the indicator	Sustainability domain
1	**Productivity:** The ratio between the number of commuters alighting the metro and changing to bus, vice-versa, and the total number of commuters alighting from metro and bus	Economic
2	**Transfer Time:** Walking/ feeder mode travel time to take the next bus/ metro, service time for security check, ticket purchase at counters time and waiting time on the platform	
3	**Transfer Cost:** Money spent to reach the bus stop from the metro station, vice-versa by travelling in any feeder mode such as cycle rickshaw, e-rickshaw and auto	
4	**Accessibility:** How accessible the bus stops are located within walking distance from the metro station and the number of bus routes available at these bus stops	
5	**Mobility:** Bus occupancy levels at bus stops near metro stations and average bus travel speeds to assess commuters' ability to take the bus after a metro ride	
6	**Additional Employment:** The number of people who utilize primarily feeder modes such as cycle rickshaw, e-rickshaw and auto for transfer between bus and metro create employment	
7	**Bus Stopping Behavior:** Combination of lateral and longitudinal deviation of bus stopping position from a designated bus stop and dwell time of the buses at the bus stop	Social
8	**Seat Availability:** Occupancy level of buses	
9	**Bus and Metro Rating:** Passenger perception on current integration levels between metro and bus	
10	**Reliability:** Variation of bus arrival time with scheduled time at the bus stop	
11	**Safety and Security:** A set of facilities required for integration include: 1. Pedestrian crossing, 2. Footpath, 3. Foot over bridge/ Underpasses, 4. Video surveillance, 5. Pedestrian signal, 6. Security personnel, 7. Emergency service and 8. Illumination/ Lighting would be determined from inventory surveys	
12	**Additional facilities:** A set of facilities provided for integration include: 1. Directional information (sign boards to guide to nearest metro/ bus station), 2. Arrival/ Departure information of buses and metro, 3. Skywalk, 4. Weather Protection and 5. Audio announcements and 6. Commercial enterprises would be determined from inventory surveys	
13	**Air Pollution:** Pollution loads by buses in the vicinity of metro stations during rush hour	Environment
14	**Non-Motorized Transport:** Locations where NMT facilities are provided for commuters to access public transport	
15	**Land Consumption:** The additional land consumed to implement infrastructure for any kind of integration between public transport modes	

Table 2 List of selected metro stations and surrounding bus stops

S. no	Name of the metro station	Sample size	S. no	Name of the bus stop	Sample size
1	Kashmere gate	180	1	Nityanand marg	597
			2	Guru Govind Sahib (G.G.S)	
			3	ISBT Kashmere gate	
			4	Ludlow castle	
			5	Mori gate	
2	Ashram	128	6	Hari nagar	388
			7	Ashram chowk	
3	Dwarka sector-10	112	8	President apartment	201
			9	NHAI building	

Table 3 Utility and weightage values determined for different indicators

S. No	Name of the parameter	Ashram	Dwarka sector-10	Kashmere gate	Weightage
1	Productivity	0.47	0.63	0.53	7.21
2	Transfer time	0.62	0.52	0.79	6.90
3	Transfer cost	0.88	0.75	0.76	5.98
4	Accessibility	0.38	0.35	0.49	7.12
5	Mobility	0.78	0.84	0.63	5.88
6	Additional employment	0.66	0.55	0.57	6.84
7	Bus stopping behavior	0.28	0.21	0.33	6.49
8	Seat availability	0.40	0.42	0.59	6.67
9	Bus and metro rating	0.69	0.65	0.65	6.34
10	Reliability	0.53	0.54	0.67	6.99
11	Safety and security	0.37	0.88	0.21	7.21
12	Additional facilities	0.59	0.42	0.75	6.64
13	Air pollution	0.76	0.96	0.22	6.60
14	Non-motorized transport	0.50	0.50	1.00	6.87
15	Land consumption	1.00	1.00	0.00	6.25

Table 4 Sustainability Integration Index of metro and buses in three dimensions

S. no	Domains of sustainability	Ashram	Dwarka sector-10	Kashmere gate
1	Economic	24.74	23.81	24.91
2	Social	19.14	21.19	21.37
3	Environment	14.73	16.00	8.35
4	Sustainability Index	58.61	61.01	54.63

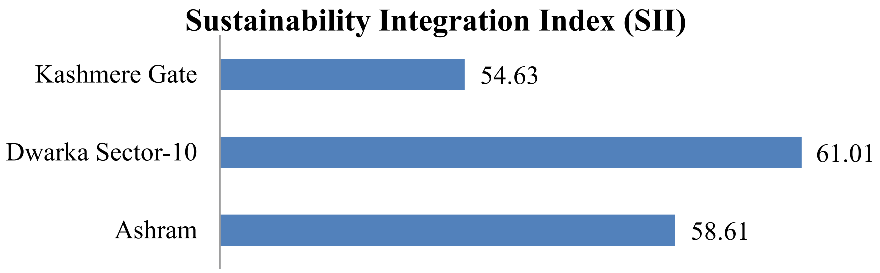

Fig. 1 Sustainability integration index (SII) of different metro stations

4.1 Calculation of Sustainability Integration Index

Utilizing the normalized utility values and weightage of indicators, the sustainability integration index for each metro station has been calculated as per Eqs. (1) and (2). The weightage indicator is multiplied by the normalized value of indicators in the dimension of Economic, Social and Environmental aspects in order to determine the SII of metro and buses. The present level of integration between metro and buses at different metro stations in Delhi is presented in Table 4 and Fig. 1. From these results, it is clear that Dwarka Sector-10 metro station has an SII value of about 61 followed by an Ashram of 59 and Kashmere Gate of 55. The reason for the high value of Dwarka Sector-10 is well-maintained footpaths with good illumination, economical transfer modes like E-rickshaw, etc.

5 Evaluation of Transport Policies

In order to demonstrate the suitability and applicability of the developed method for estimation of SII, the following transport policies relating to the integration of the metro and bus services in Delhi have been considered:

- Free shuttle service (IPT Modes) between metro station and nearest bus stops
- Common mobility card for metro and bus

- Relocation of bus stops
- Increasing bus service frequency

The first policy of free shuttles is the strategy of public transport operators in offering free shuttle (IPT modes like e-rickshaw) to reduce the cost of transfer between metro and bus. The second policy increases the ease of transfer between metro and bus by reducing waiting time at metro ticket counters and hassle-free transfers. The third policy dealt with distance and shifting the bus stops close to metro stations would reduce the distance and increase ease in transfer and building direct connections to them in order to facilitate easy transfers between buses and the metro. Last the fourth policy considered was increasing the frequencies of present bus services in order to reduce the waiting time at bus stops in turn reducing transfer time between bus and metro. The above policies would mainly influence the indicators, namely productivity, transfer time, transfer cost, accessibility, mobility, seat availability, additional employment, additional facilities and land consumption, so the values of these indicator changes under these policies. These policies have been considered to estimate the sustainability integration index (SII) at three metro stations (Ashram, Dwarka Sector-10 and Kashmere Gate) using the developed method and compared with the base case to see the impact of these policies. The positive change in SII would improve the situation hence enhancing integration levels leading to sustainability. The results estimated for these policies in terms of percentage increase in SII have been presented in Fig. 2.

From Fig. 2, it can be seen that all four transport policies have a positive effect on SII with a range of increase in SII from 2.6% to 5.8%. At Kashmere Gate metro station, the policy on Free shuttle service (IPT Modes) between the metro station and the nearest bus stops has the highest impact, whereas at Ashram and Dwarka Sector

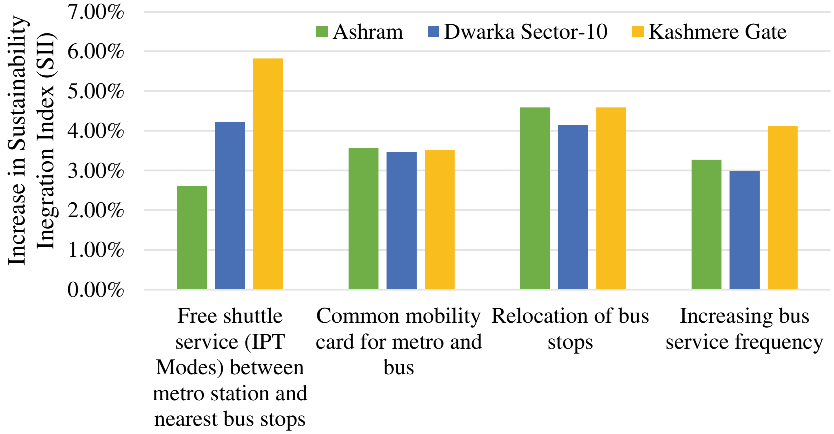

Fig. 2 Increase in sustainability integration index (SII) of different metro stations under different transport policies

10 metro stations, the policy of relocation of bus stops has the highest impact which means these policies are most beneficial in achieving sustainability in integration between metro and bus.

6 Conclusions

The present study has emphasized the need for integration between public transport modes in order to improve transit share and achieve sustainability. By collecting data from Delhi, a methodology to estimate the sustainability integration index (SII) has been proposed. In this direction, a total of 15 indicators have been identified to be utilized to estimate SII. In order to demonstrate suitability and applicability, the proposed method has been utilized to evaluate various transport policies. The four transport policies relating to the integration of the metro and bus services in Delhi have been considered to evaluate using the developed method. The policies are: Free shuttle service (IPT Modes) between the metro station and nearest bus stops, Common mobility card for metro and bus, relocation of bus stops and increasing bus service frequency. It has been found that the indicators, namely productivity, transfer time, transfer cost, accessibility, mobility, seat availability, additional employment, additional facilities and land consumption have the most influencing indicators under these policies. The policies namely free shuttle service (IPT Modes) between metro stations and nearest bus stops and relocation of bus stops have the highest impact in increasing SII.

The approach created in this study can be used to examine the current levels of integration across various public transportation modes, and the integration results will help with city-wide planning for the integration of public transportation and its viability. To boost the accuracy of the findings, more indicators may be chosen for future research.

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A Critical Review of Strategic Decisions for Travel Time Performance Improvement of Public Transport System

Narendra Dudhe, Pradeep Kumar Agarwal, and Amit Vishwakarma

Abstract The study provides a critical review of strategic decisions taken for improving the public transportation system's travel time performance (PTS). The key decision taken by various transportation systems for the improvement of travel time performance is examined and its relevance with commuter acceptance of PTS is observed. Total journey time is subdivided into Access time, in-vehicle and out-vehicle transit time, and egress time. Their percentage of total travel time influences the commuter's mode choice and hence strategies effectiveness is of prime concern to reduce travel time. Implementation of access time strategies expects to reduce the time to reach public transportation network (PTN), public transportation is more easily accessible with higher service coverage. The in-vehicle and out-vehicle travel time strategies show the key operational characteristics that influence travel time performance. In-vehicle travel time strategies emphasize the improvement and reorganization of areas responsible for enhancing travel time compatibility with personal vehicle trips. Interconnectivity and timely availability of service at transfer sites were the focus of egress time initiatives. The long-term viability of public transportation is dependent on micro-level strategy enhancement and the suitability of strategies chosen for particular transport networks.

Keywords Travel time performance · Travel time strategies · Commuters acceptance

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1 Introduction

The review of strategic decisions for improving public transportation travel time (PTTT) performance is an attempt to scrutinize identified strategies' effectiveness for PT planning and operations improvement for inefficient trips and the loss of commuter interest and reliability. "The urban population is anticipated to expand to almost 600 million (40 percent) by 2031 and 850 million (50 percent) by 2051," according to the projections [1]. To enable mobility to a large metropolitan population, an efficient PTS must be in place. PTS commuters have shifted to personal modes of transportation due to overcapacity and delays, resulting in traffic congestion and poor urban mobility. To provide excellent service and establish efficient PTS with travel time compatibility and strategic improvement is essential.

To increase commuters' willingness to switch to PT, service reliability (SR) and level of service (LOS) must be improved. PTS competency can be achieved with personal car journeys by reducing vehicle time and increasing travel speed, resulting in SR. SR is another system effectiveness parameter that may be increased by maintaining punctuality and increasing the total serviceability index [2]. Travel time performance improvement techniques are separated into three parts: Access Time Strategies (ATS), Travel Time Strategies (TTS), and Egress Time Strategies (ETS) to recommend improvement approaches at different levels of public transportation trips (PTT). Based on the facts and conclusions in the literature, Table 1 represents ATS, Table 2 shows TTS, and Table 3 indicates ETS.

This study focuses on strategies for improving travel time performance that have previously been described in the literature to aid strategic decision-making in order to achieve competent travel time performance. Figure 1 shows a schematic diagram of a public transportation system with a service access point (SAP), BRTS, or Metro as the main service.

2 Literature Review

The most important parameter for a successful PTS is travel time performance. Diagram represents a basic model of public transport services, a commuter can opt for different combinations of services between origin and destination but the travel time difference between a Private vehicle trip and PTS is high irrespective of service combination [17]. Access time, PTTT, and egress time are the three major components of travel time. The average waiting time and travel time to reach an SAP, such as a BRTS stop or a Metro station, are added together to calculate access time [3]. From the standpoint of commuters, trip planning involves everything from waiting periods to departure schedules, as well as the total route time and number of transfers. Access time, on the other hand, is frequently overlooked in PT accessibility research. To address this problem, a decision-support system was developed that allows any origin–destination pair to compute pre-trip waiting periods, travel durations, and the

Table 1 Details of strategies for access time performance improvement

Strategy ID	Strategy	Empirical findings
ATS1	Reduction of walk time to reach SAP of BRTS stop or Metro station	A strong correlation exists between PTALs and access to services. The high PTAL areas generally have good access to services and low PTAL areas have poor access to services
		"To opt bus service the max. walk time to reach SAP from the origin is standardized as 8 min of walk or a distance of 640 m. For rail, underground and light rail services the maximum walking time is standardized as 12 min or a walking distance of 960 m." [3]
ATS2	Quick Park and Ride facility	Transfer Time is the most significant aspect in activating the respondent's preference. The next important factor is the Parking Fair, and Public transport Travel Time (PTTT) has the least impact on the acceptance of P&R by travellers
		Time consumed in parking a vehicle and getting to the next facility and the parking fair is the governing factor in a commuter's decision which influences to select or reject P&R facilities [4]
ATS3	Availability of Competent Feeder Network to reach service access point	i. Comfort level of public transport systems particularly depending on the crowd density in vehicles, during peak hours ii. Commuters will adopt public transit and give priority to trip-making, when the system is properly accessible from their trip origins and service is available at preferred travel times iii. A model for optimal trip distance of each hierarchy type of route is suggested fixed on features of passengers in the PTS
		1. Commuters choose their own vehicle for its convenience and comfort, to avoid the higher level of discomfort in PTTs [5] 2. When the system is fully accessible and service is frequent, commuters will consider PT as a viable mode of transportation [6] 3. The goal of FN is to increase service offerings with the intention of improving accessibility and reducing walking distances [7]
ATS4	Increase Accessibility Levels By Providing Higher Frequency of Service	The analysis helps to visualize and assess the level of transit services in the different regions. In every region, mapped LITA scores identify the sites with limited transit services relative to those sites with higher service. The calculated LITA could be used in a Geographic Information System (GIS) to identify the points where transit service is lacking in the studied regions
		Transit must provide an appropriate level of convenience, coverage, and frequent service to peripheral locations, to compete effectively with the personal vehicle trip [8]

(continued)

Table 1 (continued)

Strategy ID	Strategy	Empirical findings
ATS5	Increase Accessibility Levels by Higher FN Service Coverage	A well-planned and systematic feeder network increases accessibility and facilitates commuters to access the public transport facility as it increases the service coverage
	LITA is regarded as a useful metric since it considers three key components of transportation system accessibility: frequency of service, capacity, and service coverage [8]	

number of required transfers [18]. PTTT can be broadly divided into two categories In-vehicle travel time (IVTT) and Out-vehicle travel time (OVTT). The level of service (LoS) is defined as the ratio of OVTT to IVTT. The OVTT/IVTT ratio is usually greater than 1. This reflects the fact that commuters spend more time outside than inside their vehicles. Longer excursions will always have a lower level of service due to the longer line-haul duration [11]. The performance evaluation procedure can help PTS improve its facilities in terms of several operational elements [19]. The performance of the urban bus system was found to be about 70% when 29 evaluation parameters were categorized into eight indicator categories. The parameters for the ‘passenger information systems’ and ‘social sustainability’ sectors were deemed to be inefficient and in need of improvement [20].

According to the study, the city coverage index and city employment index of the BCLL and Mini bus systems are both less than 0.3, indicating that the systems are lacking in terms of city coverage and city employment [19]. Another essential criterion for evaluation is the time difference between personal automobile trips and public transportation trips (PTT). The evaluation of journey time parameters serves as the foundation for strategy improvement. Travel Time Ratio (TTR), Level of Service (LoS), Interconnectivity Ratio (IR), Passenger Waiting Index (PWI), and Running Index (RI) are the parameters to evaluate for identifying inefficiencies in PTS [11]. Avg. passenger-km per vehicle-km, Annual ridership per bus station, Annual ridership per bus, Passenger trips per effective vehicle km, Operating cost per passenger-km, Revenue per passenger, Average fare per passenger-km [21]. Explanation of evaluation parameters:

- Travel Time Ratio (TTR): It is the ratio between travel time by public transport to travel time by car between the same origin and destination. It indicates the competitiveness of PTS as compared to PVT.
- Level of Service (LoS): The ratio of out-vehicle travel time (OVTT) to in-vehicle travel time VTT/(IVTT). The larger the ratio, the less attractive public transport becomes as an alternative. OVTT compared to IVTT range between 1.2 and 5.

Table 2 Details of strategies for travel time performance improvement

<table>
<tr><th>Strategy ID</th><th>Strategy</th><th>Empirical findings</th></tr>
<tr><td colspan="3">Strategies for TTP Improvement: By Minimizing Vehicle Time</td></tr>
<tr><td rowspan="2">TTS1</td><td>Minimize Waiting time by Strict Adherence to Bus Schedules</td><td>Commuters experience issues like overcrowding and travel time irregularity that need to be evaluated as accessibility indicators. Implementation of bus schedule tactics to measure travel time inconsistency in a packed full PTN helps to improve accessibility levels and minimize waiting time</td></tr>
<tr><td colspan="2">Bus schedules that are strictly adhered to increase service quality and confidence among commuters by saving commuters valuable working hours [9]</td></tr>
<tr><td rowspan="2">TTS2</td><td>Development of just-in-time approach by availability of on-time running information of fleets</td><td>GPS-enabled vehicles can transmit current location information that can be available to commuters online, to plan their trips. The real-time information about trips can help to reduce waiting times and improve TTP</td></tr>
<tr><td colspan="2">The number of transfers shows that commuters are concerned about transfers, but not exactly the number of times they need to change service [10]</td></tr>
<tr><td rowspan="2">TTS3</td><td>Minimizing waiting time by quick availability of vehicles at transfer points</td><td>PTTT is the key factor of a multimodal trip for commuters to prefer PTS for a longer trip distance ranging between 7.5 and 35 kms. PTS is preferred if the access and egress distance is not too large. TT, waiting time, and egress time are the significant and complex travel time factors. TT can be reduced by a higher frequency of service to improve efficiency and modal share</td></tr>
<tr><td colspan="2">Improved transfer facilities, P&R facilities, and card access at PTS can minimize OVTT and transfer times (TT). By facilitating diverse modes at transfer locations, TT can be reduced, resulting in efficient PTS [11]</td></tr>
<tr><td colspan="3">Strategies for Travel Time Performance Improvement: By Increasing Travel Speed</td></tr>
<tr><td>TTS4</td><td>Optimize number of bus stops</td><td>Stop locations can be placed at suitable intervals by considering walk access time so that commuters can have quick access to nearby adjacent stops. Adjacent Stops in one-kilometre vicinity can be relocated</td></tr>
</table>

(continued)

Table 2 (continued)

Strategy ID	Strategy	Empirical findings
	Bus stops can be placed in strategic locations such as institutional areas, administrative buildings, CBDs, and intersections to connect people to other places	
TTS5	The number of traffic signals	The areas with lower pedestrian traffic, actuated signals traffic control can be used along priority rapid transit corridors to increase the schedule reliability of transit service and escape unnecessary delays
	A. Ceder and I. Reshetnik [12] concluded "An investigation was conducted in Israel on 30 most important critical intersections, which have heavy traffic resulting in 15-min average delay."	
TTS6	Optimizing Stoppage Time at Bus Stop	The Total Bus Stop Time (TBST) at stops is time difference between the time the bus pulls into the bus stop and when it returns to the main traffic stream. By considering variables for a dense urban area like mode of billing, time of the day, crowd level, the time gone serving a stop, and the position of the bus stops, improvements might be made in the optimization of TBST models
	"According to a study conducted in Baghdad. Signal pre-emption for buses, on the other hand, reduces intersection delay and thus journey duration [13]"	
TTS7	Provision of a dedicated corridor	The key to successful BRTS is a dedicated corridor. The dedicated corridor provides optimum speed, safe and scheduled trips which are essential for performance
	The average operating speed of Indian BRTS is between 18 and 24 km/hr	
TTS8	Strategies for Vehicle Waiting Time Minimization at intersections	For minimizing the number of vehicles from cycle to cycle inter green signal time model can be proposed. This model also incorporates restrictions for upper and lower bounds for green signal time and cycle time allocation which provides accurate and appropriate allocation for green signal time [14]

Table 3 Details of strategies for egress time performance improvement

Strategy ID	Strategy	Empirical findings
ETS1	Strategies for minimizing transfer time via inter-connectivity improvement	A well-planned paratransit can prove a means of interconnectivity and enhance and strengthen the PTS network
ETS2	Availability of Shared Vehicle Trips for Quick Service	A shared vehicle based trip has the potential to provide door-to-door transport service with a high level of service and requires less than 10% of today's private cars and parking places [15]
ETS3	Time Table Synchronization of feeder services with BRTS and Metros	The synchronization of timetables provides a more user-oriented, system-optimal and well-connected public transport service which is more acceptable and attracts attention [16]

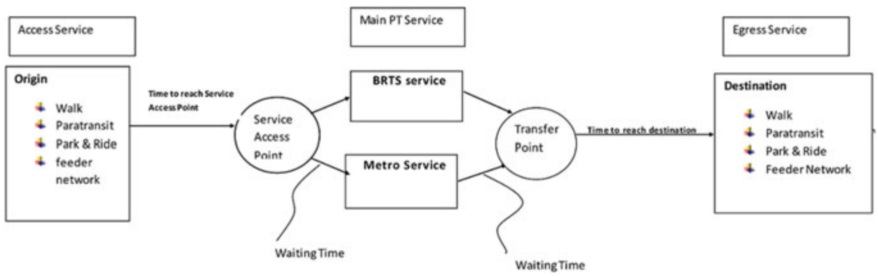

Fig. 1 Schematic diagram of public transport service

- Service Reliability: It can be defined as "the inconsistent and unpunctual service which influences traveler's opinion to opt for a PTS". It is a deviation from timetables and the average waiting time of commuters [22, 23]. Simply, it is the timely availability of service at all the stations.
- Interconnectivity Ratio (IR): It is the ratio of the aggregated access and the egress time to the total trip time. It has been ranged between 0 and 1, and found between 0.2 and 0.5 for most PT trips.
- City coverage index for PTS is the ratio of the total route length covered by PT vehicles in a city to the total length of the road network in a city both in km.
- Average passenger-km per vehicle-km is the ratio of total passenger kilometers to total vehicle kilometers. "This indicator reflects how well the operating capacity has been utilized. A higher value suggests better utilization."
- Annual ridership per bus station is the ratio of total passengers carried to the number of bus stations served.

PT provides benefits to all members of society, "those who prefer to ride as well as those who have no other option". Over 90% of the population does not possess a four-wheel vehicle and relies on public transportation. PT saves money and resources for commuters because it uses less energy per capita per kilometer travelled than private travel [24]. The majority of research does not take into account all three segments of access time, PTTT, and egress time. As a result, we haven't been able to identify the flaws that are causing the inefficient transportation service. With personal vehicle trips, present public transportation options are inefficient and time-consuming. The local Index for Transport Accessibility (LITA) index and a comprehensive spatial database on the condition of transit supply provide a practical approach to see and analyzing the level of transit services in different regions. The mapping LITA scores show which sites have little transit service compared to others that have more. The estimated LITA could be used in a Geographic Information System to identify areas with insufficient transit service.

Researchers have attempted to evaluate performance using various criteria such as the city coverage index, the LITA Index, the level of service, the interconnectivity ratio, and the passenger waiting index, but there is no sequential strategic framework for reducing travel time. This research proposes ways to improve PTS travel time performance.

3 Need of the Study

Various PTS has worked on different parameters like passenger waiting time at stops and intersections, frequency of service, and run-time information. for corresponding cities and implemented strategies to improve the efficiency. Even after a lot of efforts and actions, there are fair chances to improve travel time parameters as commuters' choice of PTS is more than 30% only when, travel speed by PT is 66% more than cars or personal vehicle trips also parameters direct routes, few transfers and high service frequency equally important to opt a PT service [25].

Travel duration, stopping periods, entrance and egress timings, and trip time element is the most significant and must be attended to first, since it reveals service reliability. According to the findings, traveling the PT takes 1.4–2.6 times longer than driving a personal vehicle [26].

The cost of time spent travelling is referred to as the Value of Travel Time (VTT). Employees who commute spend important time in PT, which can increase "costs to businesses and services." The savings from reduced travel time and vehicle running costs are referred to as VTT savings [27]. Savings in travel time have a direct influence on vehicle running costs and commuter work hours. The reduction in travel time and the level of service provided by public transportation projects are the primary criteria for evaluation. Hence, there is a need to critically examine strategic decisions taken and implemented at various levels of service by different PTS. The long-term sustainability of PTS is a difficult issue to address, as the percentage of commuters in PT does not rise at the same rate as the population. To make PT the preferred mode of transportation among commuters, significant improvements in quantitative parameters such as journey time between origin and destination are required. As a result, there is a genuine need to analyze the strategies adopted and implemented at various levels of different PTS. Also, there is a need to find a mechanism that can clearly indicate whether the set of strategies implemented in a particular PTS works together to make the system more efficient.

Hence, there must be a standard set of parameters with a strong correlation with travel time. Whose analysis can clearly define the lagging of a particular or group of parameters that have a direct impact on TTR for selecting or rejecting a PTS.

4 Strategies for Travel Time Performance Improvement

Travel time comprises three basic bifurcations access time, travel time, and egress time.

4.1 Strategy for Access Time Performance Improvement

The time it takes to go to a Service Access Point is known as access time (SAP). The whole travel and waiting time to get to SAP and choose the primary service, either BRTS or Metro, is the access time. Commuters have three primary options for getting to the SAP: walking to a BRTS stop or metro station, owning a vehicle with park-and-ride capability, or using a competent and efficient feeder network service.

4.1.1 ATS1: Reduction of Walk Time to Reach SAP of BRTS Stop or Metro Station

The density of the PTN in any urban region is similar to the density of easily available PT. The access time represents the distance walked from one's front door to the nearest public transportation SAP and has a direct impact on the PTTT between O and D. "Using PTAL criteria for mapping improves urban planning at various levels for the city, land use pattern, and boost urban transportation networks [28]."

4.1.2 ATS2: Quick Park and Ride Facility

The time it takes to drive to a parking lot, park, and walk to a bus stop or metro station must be less than the time it takes to walk to a bus stop or metro station. The amount of time spent in parking and getting to the next facility and the parking fair is the most important factor in a commuter's decision, which influences whether or not to use P&R facilities [4].

4.1.3 ATS3: Availability of Competent Feeder Network to Reach Service Access Point

Great prospects of selecting PTS as a mode of transportation rely on frequent, uncrowded, comfortable, and efficient FN that can easily transport commuters to the SAP. Commuters must have access to PTS. The amount of comfort varies depending on a multiplicity of conditions, such as crowd density in vehicles, during peak hours. A PTS is considered as efficient when seamless travel is perceived by the commuters between one mode and the other. The PTS must be easily accessible geographically and to all sections of society. Cities have different populations and travel densities in different zones and hence require the adoption of best-suited technological adoption to strengthen the feeder system and overall seamless transfer. (NUTP, MOHUA). The density of FN should be larger and the length should be longer. The goal of FN is to broaden its service offering in order to improve accessibility and reduce walking distances [7].

4.1.4 ATS4: Increase Accessibility Levels by Providing Higher Frequency of Service

Increased frequency ensures that service is always available and improves accessibility. "Simulations of actual operational data validate that up to 35% possible decrement in excessive waiting time with a slight increase of 6% in IVTT" [29].

Yet Zhang [30] observed people will become less dependent on their vehicles if commuters have more access to transit options. To maintain accessibility levels, agencies must be able to analyze the regions that require more attention in order to enhance service frequency.

4.1.5 ATS5: Increase Accessibility Levels by Higher Feeder Network Service Coverage

Increased service coverage lengthens routes per square kilometer, which has a direct impact on the number of commuters on each trip and the probability that commuters will use the service. More riders would be attracted to a well-planned and scheduled regular FN from the origin to SAP. A competent FN ensures the shortest possible travel time, increases the number of commuters, and reduces the reliance on personal vehicle trips [31].

4.2 Strategies for Travel Time Performance Improvement

Travel time performance of BRTS or Metros can be improved by controlling two basic aspects first "reducing ideal time of commuters" and second "increasing travel speed". The ideal time is OVTT or the time spent in operational activities like boarding alighting, ticketing, waiting at intersections and transfer points. The travel speed can be increased by improving Travel speed. Systematic operations can reduce OVTT and increase travel speed.

4.2.1 TTS1: Minimize Waiting Time by Strict Adherence to Bus Schedules

Punctuality in service is likely to increase commuters' confidence in PTS as it minimizes waiting time to some extent [9]. Due to crowding discomfort, researchers discovered a "population-weighted average reduction of 56.8% in accessibility to occupations in a standard workday morning peak due to crowding discomfort, as well as reductions of 6.2 percent due to travel time unreliability, and 59.2 percent when both are combined." Commuter reductions are due to user experience issues such as crowded discomfort and uncertainty about trip time. Maintaining punctuality can help to improve the predictability of travel times. It is significant to upgrade the reliability of the bus services to control the move of commuters to private vehicles and also help regulate traffic flow on routes more efficiently [32].

4.2.2 TTS2: Development of just in Time Approach by Availability of On-Time Running Information of Fleets

Commuters will be able to plan their trips more precisely if real-time fleet position information is available. It also cuts down on waiting time and, as a result, overall travel time. The public transportation system's reliability and performance will increase.

4.2.3 TTS3: Minimizing Waiting Time by Quick Availability of Vehicles at Transfer Points

To reduce transfer times, the schedules of PT modes such as BRTS, Metros, and feeder transit facilities have to be synchronized. It aids in the enhancement of travel time performance and the overall effectiveness of PTS. Improved transfer facilities, P&R facilities, and card access at public transit systems can minimize OVTT and transfer times (TT). By facilitating diverse modes at transfer locations, TT can be reduced, resulting in an Integrated Multi Modal Public Transportation System (MMTS) [11].

4.3 Strategies for Travel Time Performance Improvement: By Increasing Travel Speed

4.3.1 TTS4: Optimize Number of Bus Stop

Bus stops can be placed in strategic locations such as institutional areas, administrative buildings, CBDs, and intersections to connect people to other places. Short-distance bus stops can be combined into a single stop. A bus stop can be located within an 8-min walking distance [3].

4.3.2 TTS5: Minimize the Number of Traffic Signals or Operating Online to Reduce Crossing Time

Traffic signals (TS) control traffic while also increasing idle time and, as a result, total journey duration. Online traffic control solutions can reduce TS, and optimal time can be adjusted. Subways for pedestrian circulation, grade separators, and flyovers can be constructed in highly populated and narrow sections to reduce the need for TS. Controlling on-line traffic at signalized junctions has proven to be very effective at unsaturated intersections. A. Ceder and I. Reshetnik [12] concluded “An investigation was conducted in Israel on 30 most important critical intersections, which have heavy traffic resulting in 15-min average delay. With a congested traffic flow saturation level of 8000 vehicle/h per intersection, the developed system can improve traffic flow by reducing queue length in oversaturated intersections, saving on the order of 1000 man-hours each rush hour per intersection.”

4.3.3 TTS6: Optimize Stoppage Time at Bus Stops

Boarding and alighting (B&A) time, as well as passenger waiting time, influence bus stoppage time. Separate B&A doors, increased door width and number of doors, and low-floor buses can all help reduce B&A time. “The number of doors and B&A times have a significant effect on bus travel time to increase the level of service on the selected routes,” according to a study conducted in Baghdad. Signal pre-emption for buses, on the other hand, reduces intersection delay and thus journey duration [13].

4.3.4 TTS7: Provision of Dedicated Corridor

Because of the dedicated corridor, BRTS performs successfully. The average operating speed of Indian BRTS is between 18 and 24 km/hr. BRT stops of different systems are located before the intersection. For example, Ahmadabad BRTS has the maximum number of 127 stations in the city BRT system of India with a spacing range between 525 and 710 m [33]. Congestion pricing on Kuwait’s network, which uses a genetic algorithm to improve traffic conditions, has resulted in significant reductions in travel time: “reductions in overall delay vary from 24.4 percent to 40.58 percent, and reductions in fuel used range from 36.76 percent to 60.89 percent.“ Fuel cost savings by users had a direct impact of 27.77 percent and 43.75 percent, respectively [34].

4.4 Strategies for Egress Time Performance Improvement

The egress time (ET) is a key factor in determining service mode. After alighting from the main service, the availability of egress facilities must be frequent and convenient to reach the destination. It’s a general observation in areas with adequate accessibility; ET is also lower when the FN is strong. Quick egress service enhances the chances of PT being selected as a service.

4.4.1 ETS1: Strategies for Minimizing Transfer Time via Inter-Connectivity Improvement

Interconnectivity and, as a result, ET performance can be improved by a frequent and planned feeder service in the form of a small bus or paratransit. The length and placement of the service should be carefully planned, and feeders should be used in towns with a uniform population distribution [31].

4.4.2 ETS2: Availability of Shared Vehicle Trips for Quick Service

Localities with low population densities or suburbs in the early stages of development can benefit from shared vehicle trip service (SVT), which facilitates access and egress in the area. SVT service may prove cost-effective for users while also improving ET performance.

4.4.3 ETS3: Time Table Synchronization of Feeder Services with BRTS and Metros

A mathematical optimization model can be used to create an optimal timetable for a public transportation system that minimizes waiting times between service transfers. On the existing timetable, the weighted sum of transfer waiting times can be reduced by 11% using a heuristic technique [35].

5 Conclusion

Extensive research work has been done by researchers on various parameters like public transport accessibility levels, transfer time facility, significance of park and ride, frequency of service, network service coverage, travel time reliability, frequency of vehicle, real-time information of fleets, optimizing number of stops and stoppage time even though the efficiency of PTS is not noteworthy the reason is lack of synchronized efforts. To get remarkable results and increase commuter's acceptance of PTS we need to identify and improve the nonperforming and time-consuming parameters which make PTS inefficient. As per the review, the performance can be improved by scrutinizing implemented strategic decisions. Availability of on-time fleets, service synchronization, on-time running information of fleets, short accessibility and egress time are some of the major indicators for PTS improvement. For improvement, PTS is to be evaluated minutely at all suggested parameters and an action plan is to be proposed that can be implemented to achieve overall efficiency in the public transport system. The major part of travel time is out vehicle time due to unsynchronized public transport system. The OVT can be minimized by systematic planning and execution at certain levels, i.e. access to PT, boarding and alighting, interconnectivity at transfer points, synchronized timetable of feeder network with BRTS and metros, smart ticketing systems. To improve TTP, travel time monitoring is needed which is possible with GPS enabled and fleets. Hence, it is recommended that GPS systems must be incorporated in PT fleets to have a clear demarcation of travel time parameters. It is also recommended a committee be formed with high-rank officials who can do performance evaluations periodically.

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Trip Generation Based on Land Use Characteristics: A Review of the Techniques Used in Recent Years

Saumya Anand, Pritikana Das, and G. R. Bivina

Abstract The inadequate transportation facilities and growing population reflect the necessity of travel demand forecasting in developing nations. Forecasting travel demand serves as the foundation for planning and policy development that helps a country's economy grow. The trip generation process is the first step in the travel demand modelling process.. The accuracy of modelling trip generation is heavily reliant on the accuracy of two stages- data collection stage and generation of the model which depends on different modelling techniques used. The first and most important step in Trip Generation is data collection. Data from household trips is critical for both managing the existing transportation network and planning and designing future facilities. For many decades, household travel surveys (HTS) have been used as a time-consuming and costly method of data collection. New technologies emerge as alternatives to HTS as time passes, but HTS remains the most commonly used technique in developing countries. Data analysis techniques are the second important step in Trip generation modelling. Unlike existing modelling techniques, i.e., regression and category analysis, new modelling trip generation techniques involving machine learning have been developed in developing countries in recent years. A brief overview of data collection and modelling techniques used in developing and developed countries is provided as a comparative study.

Keywords Trip generation · HTS · ANN · ANFIS · Regression · GPS · Mobile data

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1 Introduction

Most countries treat transportation planning as an integral part of overall economic planning and have not given it any special consideration. However, both developed and developing nations have now realized the importance of separate transportation planning, both for the current system and for future growth. Transportation planning is the prediction of future travel demand and accordingly, planning for necessary facilities and services to accommodate that demand. A very important key factor in planning is traffic demand forecasting. Travel Demand forecasting is used to predict future traffic demand associated with the Traffic Analysis zone. It is essential for developing nations for the design of transportation facilities and services, and also for planning, investment, and policy development especially The first and most important stage of the conventional 'four-stage' travel demand forecasting process is the 'Trip generation'.

Trip generation is the total number of trips originating in or destined for a particular traffic analysis zone [1]. Predicting the overall number of trips that are generated from and drawn to each zone is the goal of this stage. Identifying the trip generation is crucial because it serves as the foundation for the remaining three stages. Trip generation model depends directly on the fidelity of the data collected. The precision and reliability of results obtained are increased when trip generation data is obtained using accurate and reliable methods [2]. Estimating the number of trips that will result from a proposed land use development is a critical component of preparing a transport impact assessment (TIA) which is known as "trip generation" [3]. It has been observed that a slight change in the land-use mix significantly affects the travel patterns in India [4]. A trip is defined as a one-way person movement by a mechanized mode of transportation with two trip ends, namely the origin and destination. Trips can broadly be classified as home-based trips, non-home-based trips, time-based trips and person-based trips as models based on separate purpose gives more accurate results. The home-based trips account for roughly 80 percent of all trips. Trips can also be classified as Peak and Off Peak Period Trip. Furthermore, trips are classified based on the travel behavior of the person making the trip.

To obtain data about mobility patterns, trip generation modelling has traditionally relied heavily on trip details, intercept surveys, face-to-face interviews and household interview surveys which are expensive and may be erroneous [5]. Rapid advancements in technology in the past decade have made it possible to easily collect vast amounts of mobility data at a much lower cost using global positioning system (GPS), mobile phone positioning (MPP), call record data (CDR), and web-based application data. Due to ubiquitous use of mobile phones, mobile phone data has emerged as one of the most promising sources [5]. Though low-cost and more accurate technologies are emerging for data collection, they cannot completely replace the conventional approaches and should be used in conjunction with them [6]. Limited response rates, the quality of data obtained, and expertise in such technologies are a few major barriers to the implementation of such technologies.

Various modelling techniques that are used for trip generation includes linear regression, multi-linear regression (MLR), Poisson regression (PR) and negative binomial regression (NBR) models, Logit model, multivariate analysis and hybrid models. Furthermore, over the past decade, there has been an increase in the use of soft computing methodologies for modelling trip generation because of their capacity to efficiently solve complex problems involving discrepancy, uncertainty, and imprecise information. Major soft computing techniques used for modelling such as neural networks, fuzzy logic and genetic algorithm have been found to perform better [2].

i. This paper aims to take a broader view by presenting an international and national literature synthesis of recent years, with a focus on techniques used in both stages of the trip generation process, particularly in developing countries. The objective of this review paper is to provide: Insights into the data collection techniques of developing countries
ii. Overview of modelling techniques for trip generation of developing countries
iii. An outline and comparison of developed and developing countries data collection and modelling techniques.

1.1 Literature Search

The current review study attempted to address the above three key aspects through a comprehensively updated synthesis of the selected literature studies. To identify relevant research studies, literature databases Google Scholar, Scopus, TRID and Web of Science were used. The terms trip generation, land use, regression technique, Origin destination survey, household survey, travel demand, neural network, soft computing, mobile phone data, and trip data were all searched for while looking for the relevant literature studies. This review paper restricted itself to literature studies published in English language. "Backward referencing" was used to add more relevant searches, which was then followed by "forward referencing". Second, only journal articles and conference papers were included in the review. The articles extracted from the database were inspected for duplications and eliminated if any were discovered. The remaining articles were then screened based on their titles, followed by abstracts, and finally the full text of the paper. Finally, the remaining articles were incorporated into the systematic review.

The flow diagram of the literature review structure using the PRISMA methodology is depicted in Fig. 1. A total of 25 papers were reviewed that are related to the trip generation of developing and developed countries. Out of 25 publications reviewed, 16 focused on the developing countries and 9 on the developed countries.

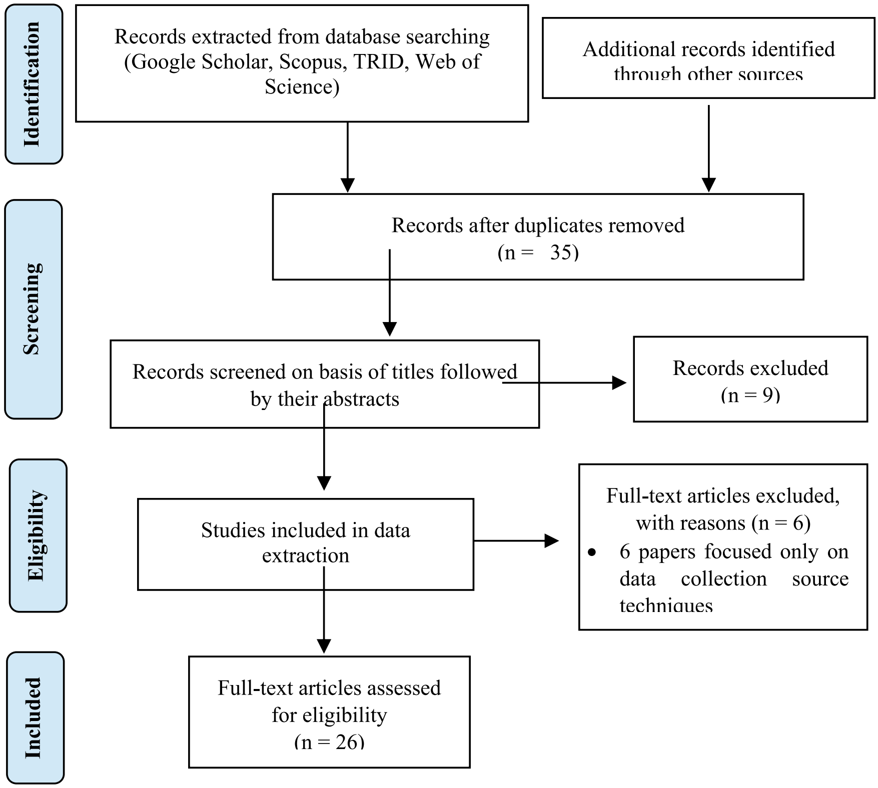

Fig. 1 Flow diagram

2 Literature Review

2.1 Traditional Method and New Technologies in Data Collection in Developing Nations

This section presented the findings of the literature review in accordance with each objective of the research. Travel data collection is the most time-consuming and important step to obtain a reliable trip generation model. Traditionally, most literature studies have developed a trip generation model based on data collected through household surveys that include face-to-face interviews [7]. This is the simplest way to collect trip data, however, it is expensive, which is a problem in developing countries. One of the most crucial aspects of the survey is the population sample size for the HTS. It is not practical to interview the entire population, as in a census [7]. A study looked into possible cost-cutting measures for Comprehensive Travel Surveys (CTS). The larger the sample size, the higher the cost and the higher the quality. They investigated whether reducing the survey sample size would result in a significant increase in error. As a result, the authors concluded that, with the exception of education trips, a household survey sample size of 6000 could be reduced to 1000 with an acceptable loss of result quality. [8]. A study of Bharatpur Metropolitan City in Nepal used a universal traditional method of household data collection and multilinear regression to develop different trip models based on the trip purpose [9]. The trip generation model for China was predicted using a household survey and modelling was done through the multinomial logit model (MNL) which was then compared with the Support Vector machine (SVM). The author concluded that SVM outperforms MNL [10]. The household interview was used to collect data for modelling activity-based trip generation using the NLOGIT model [11]. In the Indian context, a study was conducted in Bangalore to model work trip generation based on gender using a regression framework [12]. Another household survey was conducted in Chennai city to develop a model between trip frequency and determinants of vehicle ownership (2-wheelers and 4-wheelers) [13].

These traditional techniques have some limitations which include missing data, higher cost, huge requirement of manpower, low response rate. However, as technology has advanced, developing countries especially China have begun to use advanced technologies for data collection. The advancement in technology, i.e. the use of ICT is related to trip generation and trip rate [14]. The data contained in smartphone signalling, such as travel volume and geographic distribution from beginning to end, is fully utilized which was further used to obtain significant trip rates in China [15]. Another study in China created a travel survey system that collects data from individual GPS trajectories [16]. A study was conducted in Brazil city to identify high-density trip generators after detecting changes in land use using GIS and remote sensing using image classification [17].

To collect travel data speedily, accurately and cost-effectively, a joint framework combining household travel surveys and call data records should be established so as to eliminate the limitations of both approaches [6].

2.2 Conventional and Machine Learning Techniques in Modelling Trip Generation in Developing Nations

Machine learning has now permeated all aspects of life. The use of machine learning in data modelling and analysis outperformed traditional regression techniques. Few studies in the literature show that the ANN modelling technique was found to perform better than traditional approaches not only in developed countries but also in developing countries. A model was developed for three provinces in Turkey, one from each of the developed, developing, and underdeveloped regions. The models were created using regression analysis and an ANN based on the collected dataset. MLR models, PR models, and NBR models were used in the regression technique. A comparative analysis revealed that the ANN model outperformed the regression models [2].

MLR is a well-established and widely used technique for studying the relationship between variables, but with the introduction of artificial intelligence in the field of travel demand modelling, the previous approach may not perform well when modelling complex relationships. This could be seen clearly in a study showing a trip generation model for Palestine City using household survey data. The author build the relationship using the Adaptive Neuro-Fuzzy Inference System (ANFIS) and compared the result with the MLR approach and concluded ANFIS as a better performer for modelling [18]. Another similar technique known as Radial Basis Function Neural Network (RBFNN) was proved to perform better than MLR for modelling trip generation in the high-density zones of Nigeria [1]. The model has been developed using ANN in MATLAB used to study trip generation in Meerut City, India [19]. Table 1 shows the data collection methods and modelling techniques of developing countries in recent years.

2.3 Review of Techniques of Trip Generation in Developed Nations

To provide a brief overview of the techniques in developed cities mostly in the US and the UK, a total of 8 literature are reviewed. The data collection methods and modelling techniques used by developed countries in recent years are shown in Table 2. GPS-based surveys, mobile phone data, and call detail records have all gained popularity in recent years due to their ability to provide massive amounts of raw data that can be used to study the complex behaviour of trip generators. Because of the good response rate and availability of validated data in developed countries, GPS-based smartphones have shown an increase in the average trip rate. Mobile phones, GPS-based surveys, and Google's passive datasets are now widely used as a source of data collection in many studies [5, 20, 21]. With the increased use of new technologies for data collection, the incorporation of machine learning has grown significantly.

Though new innovative technologies are emerging the use of traditional approaches still used in modelling and data collection as appeared in some of the literature studies [22–25].

3 Summary

This study presents a review of data collection and modelling techniques in trip generation models used in developing countries in recent years. A brief overview of the same for developed countries has also been provided. HTS has been used for data collection for many decades, but new technologies such as GPS technology,

Table 1 Summary of literature studies on techniques for developing countries

S. No	Study	Location	Data collection techniques					Modelling techniques					
			Questionnaire survey	O-D survey	Mobile phone data	GPS	Others	Regression	Logit model	ANN	ANFIS	Comparative analysis	Others
1	Khadka and Shrestha (2019)	Nepal	–					–					
2	Yang et al. (2016)	China	–						–				
3	Eisheh and Irshaid (2020)	Palestine	–					–					
4	Kulpa and Szrata (2016)	Poland	–					–					
5	Shi and Zhu (2019)	China			–			–					
6	Kabakus and Tortum (2019)	Turkey	–					–		–		–	
7	Hedau and Sanghai (2014)	India	–						–				
8	Yin et al. (2021)	China	–					–					

(continued)

Table 1 (continued)

S. No	Study	Location	Data collection techniques					Modelling techniques					
			Questionnaire survey	O-D survey	Mobile phone data	GPS	Others	Regression	Logit model	ANN	ANFIS	Comparative analysis	Others
9	Etu and Oyedepo (2018)	Nigeria	–					–		–		–	
10	Bwambale et al. (2020)	Bangladesh	–		–				–				
11	Machado and Quintanilha (2019)	Brazil					–						–
12	Zhou et al. (2019)	China				–							–
13	Goel and Sinha (2008)	India					–			–			
14	Gopisetty and Srinivasan (2013)	India	–					–					
15	Rahul et al. (2019)	India	–					–					
16	Sarkar and Chunchu (2016)	India	–					–					

Table 2 Summary of literature studies on techniques for developed countries

S. No	Study	Location	Data collection techniques					Modelling techniques					
			Questionnaire survey	O-D survey	Mobile phone data	GPS	Others	Regression	Logit model	ANN	ANFIS	Comparative analysis	Others
1	Sana et al. (2018)	US		–								–	
2	Gruyter et al. (2021)	UK and Ireland					–						–
3	Bambwale et al. (2017)	UK			–								–
4	Moeckel et al. (2020)	Germany	–										•
5	Noland and Guo (2016)	US				–		–					
6	Zhang et al. (2019)	Portland	–					–					
7	Orvin et al. (2021)	US	–				–	–				–	
8	Huntsinger et al. (2013)	US	–					–					

mobile phone positioning, CDR, and so on have emerged as a potential and cost-effective source. It is observed that the majority of developing countries use the HTS approach for data collection except China and most of the advanced technology of data collection is used in developed countries. These technologies cannot completely replace traditional ones due to a lack of training and expertise in using the new and innovative technologies. Thus, conventional and new techniques should complement each other to produce better results..

Secondly, a brief review of modelling techniques shows that worldwide, regression analysis is one of the most used modelling techniques in developed and developing countries. With the increase in non-linearity and complexity of variables use of machine learning for modelling is increased in both developed and developing countries. Literature have shown that ML modelling outperforms the regression approach in most cases. However, it is still not feasible for every study to use ML because it is a fairly complex technique that requires a large sample size for accuracy, which affects project costs. As a result, efforts should be made to reduce the sample size without affecting the accuracy of the technique for future studies.

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Mode Choice Behaviour of Students Using Structural Equation Modelling

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Abstract Transportation is one of the most significant aspects in the overall growth of a nation, requiring serious consideration in transportation planning. The planning and management of any transportation system involve a thorough examination of the mode choice behaviour of people. It enables one to determine the demands and most preferred modes of transportation. Since, students make up a major portion of the population whose travel behavior is extremely distinctive and flexible, making them receptive to a variety of different travel modes is necessary in order to construct the travel demand model for a region. This study aims to examine the mode choice behaviour of students using the activity-travel data of Calicut city, Kerala state, India. Structural Equation Modelling was adopted to test the separate effect of household characteristics, personal characteristics and a combined effect of both personal and household characteristics and the effect of travel time, travel distance and travel cost on the mode choice of students.

Keywords Structural equation modelling · Students · Mode choice

1 Introduction

Urban regions are seeing a tremendous expansion in the number of schools and colleges due to a growing demand for education. The morning and evening peak hours of the working day in a growing nation like India cause considerable traffic congestion for all commuter groups. Student commuters make up a considerable share of the traffic jams, although they are underrepresented in the majority of travel research.

Students exhibit a distinct way of travelling and are more eager to try out different travel modes. Moreover, their travel behaviour shows some amount of uncertainty, which distinguishes them from the general population. Students' travel needs have an impact on the travel behaviour of other household members. School students in

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particular rely heavily on household adults or other adults to drive them to activities. College and university students, however, have complicated and diverse travel behaviour. They have autonomy in their decision-making regarding their activities and travel. Additionally, the daily activities of a student are highly influenced by their peers. Therefore, in order to create an appropriate travel demand model for a region, a research that takes into account how the students commute is required. This is crucial for places with large universities or lots of colleges and schools since many students and staff will be travelling during certain hours of the day, which may cause congestion in the area. Exploration of students' travel behaviour can be instructive and reveal valuable information about associations with the built environment and the extent of differences in travel (e.g., trip generation and mode choices) compared with the general population [1]. Hence, by better understanding the travel behavior of students, urban planners can propose suitable policy measures to promote a more comfortable travelling environment for other population segments as well.

The objective of this study is to investigate the impact of household and personal characteristics and other travel characteristics of students on their mode choice. This study also attempts to identify the potential policy factors that might encourage students to utilise sustainable modes of transportation.

2 Literature Review

Mode choice models are frequently used to determine the impact of socio-demographic characteristics, service features, etc. on the decision-making process for mode selection. Various socio-demographic and service factors, such as travel time, travel cost, gender, age, driving privileges, residential location, waiting times, the number of transfers required when using public transport, comfort, etc., are significant and often employed in most research [2]. Studies have indicated that adding land use or built-environment factors to the traditional parameters, such as personal and household socio-economic characteristics, and features of the activities that people travel for, can enhance the travel demand model. One of the factors influencing a commuter's choice of travel route is likely to be one of the residential site features, which are included in the category of residential location characteristics.

A study in Gainesville, Florida, examined a wide range of factors. These factors included overall density, the balance of jobs and residents, the job mix, the commercial floor area ratio, sidewalk coverage, bike lane and paved shoulder coverage, street tree coverage, and accessibility measures, in addition to household income, auto ownership, license ownership, and walk time and bike time. According to the findings, students are more likely to walk or bike to smaller schools in walkable areas than to larger schools in distant areas [3]. According to Danish surveys, kids aged 5–8 are equally likely to get to school on foot, on bicycle, by automobile or by public transport, whereas students aged 15–16 are more likely to bike. All ages could walk and use public transport equally. They discovered that whereas males were more likely to ride to school, girls were more likely to walk and take public transport. The findings showed that students' choice of transport mode is not substantially influenced by family type or financial level [4]. Another study revealed that socio-demographics, household mobility alternatives, social/cultural norms, and traffic safety may all be equally significant [5]. In Toronto, Canada, the effects of the built environment and household interactions on the school travel behaviour of 11-year-old students were examined. A Multinomial Logit model with geographic weighting was employed to investigate mode choice behaviour. The results of this study revealed that the students' decision to choose a certain mode of transportation was most significantly influenced by travel distance.

According to a study conducted in Kochi, India, school buses were chosen by elementary and secondary school students over public buses when it came to their safety and the household's monthly income. School children prefer school buses more when the distance between their location and the destination rises, whereas high school and college students like public transportation and two-wheelers, respectively. Gender was a less important factor for the mode choice decision of higher secondary students compared to the school and college students. Household size and number of employees per household were significant for the mode choice behaviour of school students but not for the other two categories [6].

3 Methodology

The data for this study comes from a 2010–2011 activity-travel survey conducted in Kozhikode city of state Kerala, India, through a home-interview survey. For data collection, random sampling scheme was adopted. The database includes household information, personal information and one-day activity-travel details of all individuals [7]. Details pertaining to 4700 students were extracted from the main database, for carrying out the study.

As the present study, the aims to examine the direct effects, indirect effects and total effects among various variables, a structural equation model for mode choice is developed. Mode choice is considered an endogenous (dependent) variable. Household characteristics and personal characteristics as latent exogenous variables. Travel time, travel distance and travel cost as observed exogenous variables. All the exogenous variables are selected on trial basis to get a stable and only the statistically significant variables. The details of endogenous and exogenous variables used in the study are shown in Table 1.

For the purpose of thorough understanding and to gain a full grasp of the impact of socio-demographic characteristics of students on mode choice, hypotheses are formulated as follows:

Hypothesis 1: Personal characteristics influence mode choice.

Hypothesis 2: Household characteristics influence mode choice.

Hypothesis 3: Both personal and household characteristics influence mode choice.

Hypothesis 4: Travel distance affects mode choice.

Hypothesis 5: Travel time affects mode choice.

Table 1 Endogenous and exogenous variables used in the study

Variable	Notation used	Description and code used in the dataset
Mode choice	MODE	Type of mode chosen (1-car, 2-two-wheeler, 3-autorickshaw, 4-bus, 5- cycle, 6-walk)
Personal characteristics		
Age	AGE	Age of student
Gender	GENDER	Gender of the individual (1-Female, 0-Male)
Education	EDUCATION	Education level (6-Post graduate and above,5-Graduate,4-Higher Secondary, 3-High School, 2-Primary school, 1-Kindergarten, 0-No education)
Personal Income	PERINC	Personal monthly income
License availability	LICAVAIL	License status (1-Yes, 0-No)
Type of vehicle	TYPVEH	Type of vehicle owned (6- Only Cycle, 5- Only HMV, 4- Both Car and TW, 3-Only auto rickshaw, 2-Only Two-wheeler, 1-Only car, 0-No vehicle)
Exclusive vehicle	EXCLUSIVE	Vehicle available for exclusive use (1-Yes, 0-No)
Type of exclusive vehicle	TYPE EXVEH	Type of vehicle available for exclusive use (3-Only cycle, 2-Only car, 1-Only 2W, 0- No vehicle)
Household Characteristics		
Household size	HHSIZE	Number of household members
Dwelling unit	DWELL	Type of dwelling unit (1-apartment, 0-independent)
House ownership	OWNER	House ownership (0-own, 1- rented, 2- Govt. quarters, 3-others)
Household income	HHINC	Household monthly income
Students	STUD	Number of students in household
Vehicles in household	HHVEHICLE	Number of vehicles in household
Employed persons in the household	HHEMPLOY	Number of employed persons in the household
Travel details		
Travel distance	TD	Travel distance (in km)
Travel time	TT	Travel time (in minutes)
Travel cost	TC	Travel cost (INR)
Travel time per km	TT per km	Travel time per km (in minutes)
Travel cost per km	TC per km	Travel cost per km (INR)

Hypothesis 6: Travel cost affects mode choice.

4 Data Summary

The average household size of the study area was found to be 4, with a minimum of 1 and a maximum of 13 members per household. Around 41% of households own at least one automobile. In the dataset, 51% were females and 49% were males. About 24% of the sample were students. Table 2 summarizes the descriptive statistics of the sample data.

Table 3 presents the summary statistics of travel details, based on various travel modes. From the analysis, it was observed that for long distance travel, bus is preferred which takes the maximum time as well.

Table 2 Descriptive statistics of the sample data

Variable	Percent	Mean	Std. Dev
Gender	Female: 51 Male: 49	–	–
Age	–	12.90	4.72
Has driving license	28	–	–
Has vehicle available for exclusive use	10	–	–
Household size	–	4	1.64
Household income (INR)	–	23,707	23,205
Number of employed persons in household	–	1.29	0.88
Number of vehicles in household	–	0.98	0.94

Table 3 Summary statistics of travel details

		Car	TW	AR	Bus	Cycle	Walk
Mode share, %		10.97	23.90	7.89	38.80	1.33	17.12
Travel Time (TT), minutes	Maximum	30	30	35	60	40	30
	Minimum	5	5	5	5	3	5
	Mean	11.57	10.76	10.89	20.36	14.87	10.59
Travel Distance (TD), kilometre	Maximum	18	18.8	17.5	21.1	11.4	9.8
	Minimum	0.3	0.3	0.3	0.3	0.3	0.3
	Mean	5.30	4.70	3.79	5.74	1.77	0.88
Travel Cost (TC), INR	Maximum	73	30	49	15	0	0
	Minimum	2	1	12	5	0	0
	Mean	21.77	7.76	24.93	5.64	0	0

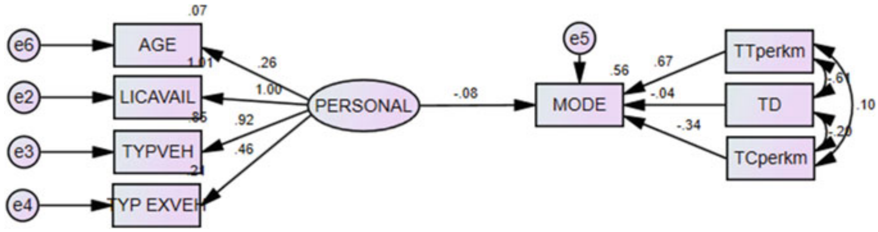

Fig. 1 Structural model based on Personal Characteristics

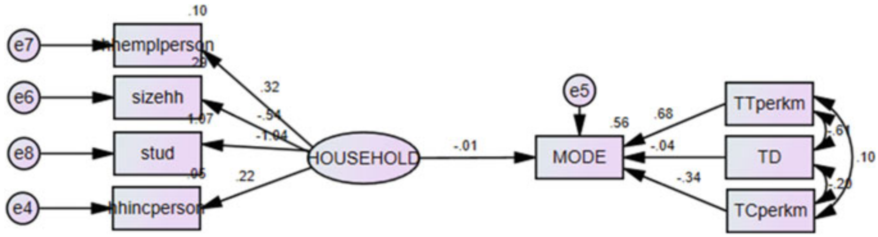

Fig. 2 Structural model based on household characteristics

5 Mode Choice Modelling

In order to determine the impact of socio-demographic characteristics of students on mode choice, structural equation models are developed. Number of samples used for model development is 4700. The structural models developed for students based on personal characteristics and household characteristics separately are shown in Figs. 1 and 2, respectively. Figure 3 shows the structural model developed for the combined effect of personal and household characteristics on mode choice. The standardized regression weight estimates, critical ratio and p-values, along with the goodness of fit measures are tabulated in Tables 4, 5 and 6. The level of significance is based on the critical ratio of the regression estimate. The statistic formed by dividing an estimate by its standard error is called the critical ratio (CR). Thus, when critical ratio values are greater than or equal to 1.96, it indicates a 95 percent level of significance.

6 Discussions

The effect of personal variables on mode choice shows that age has a positive influence. This implies that as the age of the student increases, they are more likely to prefer bus, cycle and walk. A similar finding was previously reported in a study conducted in Southern California [8]. This may be attributed to the fact that parents feel more comfortable letting older children use these modes. In addition to this, it

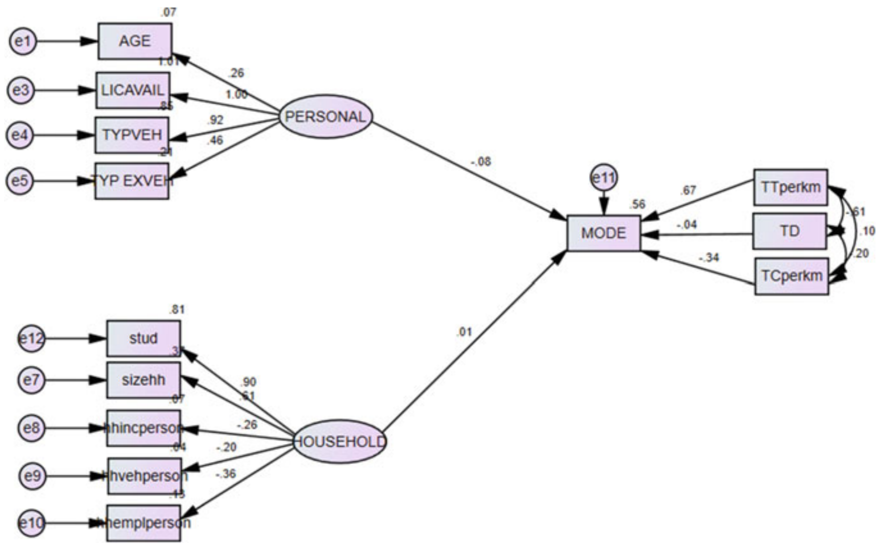

Fig. 3 Structural model based on both personal and household characteristics

Table 4 Model result (Personal characteristics)

	Estimate	CR	p
AGE ← PERSONAL	0.261	18.296	***
LICAVAIL ← PERSONAL	1.004		
TYPVEH ← PERSONAL	0.922	86.418	***
TYPEXVEH ← PERSONAL	0.463	33.725	***
MODE ← PERSONAL	−0.080	−8.282	***
MODE ← TT per km	0.675	55.278	***
MODE ← TD	−0.044	−3.533	***
MODE ← TC per km	−0.339	−34.394	***
Goodness of Fit Measures			
Comparative Fit Index (CFI)	0.975		
Tucker Lewis Index (TLI)	0.959		
Root Mean Square Error of Approximation (RMSEA)	0.073		

may be assumed that at graduation and post-graduation level, students are ready to travel more distance than they travelled for their schooling, causing them to prefer bus, cycle and walk as travel mode. Gender did not show any effect on their mode choice decision, which contradicts the result obtained by a study conducted in Iran [1]. The type of vehicle owned and the type of exclusive vehicle owned have a positive impact on the mode choice. Household size and the presence of students in a

Table 5 Model result (Household characteristics)

	Estimate	CR	p
SIZEHH ← HOUSEHOLD	−0.535	−17.992	***
HHINCPERSON ← HOUSEHOLD	0.218	11.269	***
HHEMPLPERSON ← HOUSEHOLD	0.322		
	Estimate	CR	p
STUD ← HOUSEHOLD	−1.035	−12.331	***
MODE ← HOUSEHOLD	−0.005	−0.470	0.638
MODE ← TT per km	0.678	55.389	***
MODE ← TD	−0.044	−3.569	***
MODE ← TC per km	−0.336	−34.025	***
Goodness of Fit Measures			
Comparative Fit Index (CFI)	0.996		
Tucker Lewis Index (TLI)	0.991		
Root Mean Square Error of Approximation (RMSEA)	0.021		

Table 6 Model result (Personal and Household characteristics)

	Estimate	CR	p
AGE ← PERSONAL	0.261		
LICAVAIL ← PERSONAL	1.004	18.297	***
TYPVEH ← PERSONAL	0.922	18.510	***
TYPEXVEH ← PERSONAL	0.463	16.541	***
SIZEHH ← HOUSEHOLD	0.610		
HHINCPERSON ← HOUSEHOLD	−0.256	−13.297	***
HHVEHPERSON ← HOUSEHOLD	−0.197	−10.378	***
HHEMPLPERSON ← HOUSEHOLD	−0.360	−18.248	***
STUD ← HOUSEHOLD	0.898	19.709	***
MODE ← PERSONAL	−0.080	−7.604	***
MODE ← HOUSEHOLD	0.009	0.693	0.488
MODE ← TT per km	0.675	55.302	***
MODE ← TD	−0.044	−3.534	***
MODE ← TC per km	−0.339	−34.409	
Goodness of Fit Measures			
Comparative Fit Index (CFI)	0.963		
Tucker Lewis Index (TLI)	0.944		
Root Mean Square Error of Approximation (RMSEA)	0.050		

household are having a negative effect on mode choice which indicates that as the household size and the number of students in a household increases preference for car increases. The probable reason may be getting accompanied by any other person or an adult from the household. The number of employed persons per household size is negatively influencing the mode choice of students. That means, increase in the number of employed persons in households increases the chance of choosing a car and two-wheeler as travel modes. This may be because the students are dropped off/ picked up by car and two-wheeler by household members while going to work. The household income per person has a positive impact on mode choice. This implies that as the household income per person increases, the more likely is the chance to prefer the bus. The number of employed persons per household size is negatively influencing the mode choice of students. That means, an increase in the number of employed persons in households increases the chance of choosing a car as a travel mode.

Among the travel attributes, travel distance and travel cost per kilometer have a negative value, indicating the preference for car for long-distance trips. Travel time per kilometer is positively influencing the mode choice. Results obtained from the present study and study conducted in Ahmedabad, Gujarat, India [9] simply that travel time, travel distance and travel cost were found to be significant in the mode choice of students. Private cars and two-wheelers were found to be the most preferable mode choice among students. Similar observations were also made in a study conducted in Malaysia [3]. The model results of the combined effect of personal and household characteristics show that the effect of household characteristics is not significant on mode choice. Table 7 shows the result of formulated hypotheses.

Commenting on the model fit, all the models have Comparative Fit Index (CFI) values and Tucker Lewis Index (TLI) values closer to 1. Moreover, the Root Mean Square Error of Approximation (RMSEA) values for all the models are less than 0.08 which is a quite satisfactory value and indicates a very good model fit.

Table 7 Hypothesis result

Hypothesis	Significance
H1: Personal characteristics influence mode choice	***
H2: Household characteristics influence mode choice	–
H3: Both personal and household characteristics influence mode choice	–
H4: Travel distance affects mode choice	***
H5: Travel time affects mode choice	***
H 6: Travel cost affects mode choice	***

(***Significant at 95% level of significance)

7 Policy Implications and Future Directions

This paper provides valuable insights into the mode choice behaviour of students. Mode choice models developed for students revealed that the majority of them prefer cars and two-wheelers. In order to attract more students to switch to non-motorized modes (cycling and walking) and to ensure safe and secure walking, particularly for short-distance trips, pedestrian facilities should be improved. More pedestrian-crossing facilities and more pedestrian-crossing intervals are all expected to encourage walking and the likelihood of using bicycles. The use of sustainable modes should be promoted, for an overall reduction in road congestion and improvement of air quality parameters.

The future research of this study can be expanded in several aspects. Firstly, the data collection can be done in different seasons, so that the effect of weather on the travel mode choice of students can be understood more clearly. Secondly, the location of schools, whether it is located in a rural or urban area, can be considered in students' mode choice behaviour as well.

8 Conclusions

This study has identified the impact of personal characteristics, household characteristics and travel characteristics on the mode choice of students. Data of 4700 students was used for analysis. This work aimed at understanding and modelling students' mode choice behaviour.

The structural equation models developed for the mode choice of students revealed that among the personal characteristics that showed a significant impact on the mode choice decision of students are age, license availability, type of vehicle owned and the type of exclusive vehicle owned. The number of members and number of students in a household significantly influence the preference for cars and two-wheelers. The combined effect of personal and household characteristics revealed that household characteristics have no significant impact on the mode choice decision of students. Travel distance, travel time and travel cost are the travel characteristics that showed a significant effect on students' mode choice. Bus is the least preferred mode as the travel cost per kilometer is considered. For longer travel distances, the preference for cars increases.

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Pune Metro: Preference Survey and Analysis for the Modal Split of Work Trips to Hinjewadi

Priya Hirave and Vidula Sohoni

Abstract Pune is one of the cities selected for the smart city mission, an initiative undertaken by the Government of India. Pune stands eighth amongst the largest metropolises in India and ranks in second position after Mumbai in Maharashtra. Pune has been very well known as 'Educational Hub' since ancient times. The development of the IT sector and many industries has given rise to employment opportunities. Besides this, Pune has cultural importance as well. Thus, migrants are attracted to Pune from different parts of the country as well as abroad. There is a significant increase in vehicle ownership and ultimately the number of vehicles on the road for the transportation of the huge population in Pune. Nowadays, traffic congestion problem has become very serious and needs to be solved smartly. The infrastructural development alone cannot solve the traffic problems and so there is a need to strengthen the public transport system with the appropriate modal split. Mass rapid transit system plays a vital role in such metro cities. Pune Metro is under construction and there is a need to analyze the preference of Pune Metro by the people so that Pune Metro can be considered as a major mode of transport in the modal split for work trips to Hinjewadi. This study reveals the preference of people to use metro for the work trips to Hinjewadi. The analysis of various criteria by stated preference survey shows that the majority of people are definitely willing to choose the metro.

Keywords Modal shift · Mass rapid transit system · Stated preference

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1 Introduction

Pune being one of the largest metropolises in India and well known for educational facilities, employment opportunities, and historical importance attracts many migrants. As far as traffic congestion problems in the metro cities are concerned, the challenges include a rise in population, increase in traffic congestion, increase in consumption of energy, rise in air pollution, adverse effects on health and reduce road safety. When the Peak Hour Peak Dispersal Travel rises beyond 10,000, the road transport system is inefficient to cater the traffic. Thus, there is a need to introduce the mass rapid transit system as a public mode of transport and also strengthen the public transport system. Some of the solutions to these challenges can be Prioritization for the use of public transport, motivating walking, promoting non-motorized transport, integration of various modes of public transport, transit-Oriented Development and integration of land use and transportation modes [7]. The public transport network should assure the commuter to complete his journey in any part of the city by public transport. The modal split and the capacity of modes also vary as the Travel demand varies from corridor to corridor. An efficient public transport system should be designed to suit the demand level of a particular corridor. Before doing transportation planning for any city, it is necessary to know the choice of mode that is used by the people of any particular area [1]. So modal split analysis helps to decide the mode of travel such as bus, car, auto, and railway.

2 Traffic and Transportation in Pune

The concept is to strengthen public transport by the optimum modal split of various modes of transport. The trips generated are categorized as work trips, business trips, educational trips, shopping, religious trips, recreational trips, health trips, tourist trips, etc. [4]. More than 50% of trips are contributed by work trips and out of all the work trips, traffic intensity and traffic congestion problems are found to be at Hinjewadi IT Park. Thus, work trips to Hinjewadi are taken into consideration for this research.

2.1 *Distribution of Trips by Purpose*

See Table 1.

Table 1 Distribution of trips by purpose in the study area

Sr. No	Purpose	PMC (%)	PCMC (%)
1	Work	50.05	54.18
2	Business	5.18	1.67
3	Education	31.67	41.55
4	Shopping	3.62	0.31
5	Social/Religious/Recreational	0.57	0.02
6	Health/Hospital	0.34	0.04
7	Tourism	0.23	0.01
8	Other Purpose	2.35	2.23
Total		100	100

Source CMP Report by PMRDA, Nov 2018

Table 2 Mode-wise distribution of trip purpose

Sr. No	Purpose	Car (%)	Taxi (%)	Two-wheeler (%)	Auto-Rickshaw (%)
1	Work	55.70	46.61	54.58	49.57
2	Business	20.05	25.43	18.19	17.54
3	Education	5.70	6.74	11.27	7.18
4	Shopping	6.39	8.83	7.46	15.13
5	Social/Religious/ Recreational	3.32	3.02	3.26	3.28
6	Health/Hospital	2.12	2.01	1.19	2.94
7	Tourism	3.72	5.15	1.97	2.97
8	Other Purpose	3.00	2.21	2.10	1.39
Total		100	100	100	100

Source CMP Report by PMRDA, Nov 2018

2.2 Mode-Wise Distribution of Trip Purpose

See Table 2.

2.3 Mode-Wise Traffic Composition

See Fig. 1.

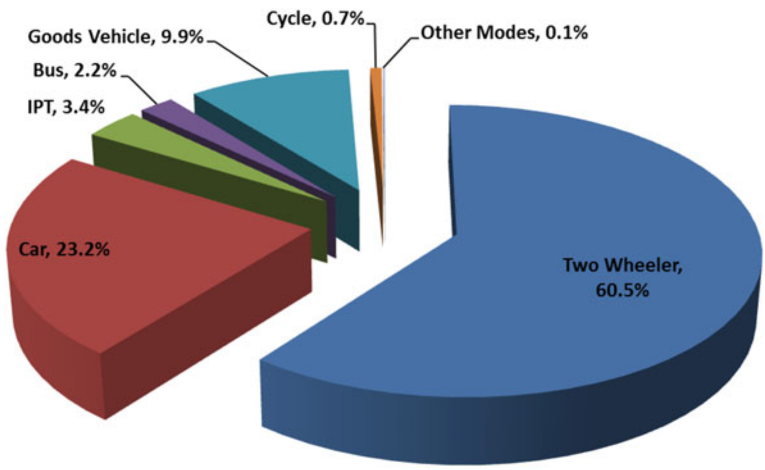

Fig. 1 Average traffic composition. *Source* CMP Report by PMRDA

3 Modal Shift

The study area considered is Pune city, and the upcoming rapid mass transit system "Pune Metro" is under progress. There are very high expectations to reduce the traffic problems once the metro project is completed. However, the success of the Metro Project depends on the user characteristics and the modal split of various modes of transport on a particular corridor to suit the demand.

The focus of this research is to achieve the modal shift from private vehicles to public transport. The existing public transport systems can be evaluated on the current conditions like travel time, travel fare, waiting time, daily passengers, etc. [6]. This evaluation can help in improving these public transport modes. As the Metro Project is upcoming, there is no evidence of its success. Stated Preference Survey on a particular metro corridor will enable the perception of passengers about the use of the Metro as a rapid transit mode. The analysis of this data will help in making the Metro Project an important mode of transport in the modal split. This will enhance the probability of achieving the modal shift and thus solving the traffic congestion problems in the metro cities.

The traffic problems in metro cities can be solved by achieving a modal shift of passengers from private vehicles to the public transport system [5]. Due to the expanding cities and increase in travel time, people will shift from private vehicles to public transport only if they get benefitted from public transport over private vehicles like:

(a) Reduced Travel Time
(b) Reduced waiting time
(c) Door to door Connectivity
(d) Seamless Journey

(e) Reduced driving stress (Comfortable journey)
(f) Reduced Travel Cost

This can be achieved by the Integrated Public Transport system having the appropriate modal split to suit the demand levels. This research will give insights to make this mass rapid transit system more reliable in achieving the modal shift and a perspective to solve the traffic problems in metro cities.

4 Data Collection for Stated Preference Survey

The survey was designed in two phases:

(a) Roadside Interview
(b) Online Questionnaire

Road Side Interview: The first phase of the survey was conducted at Hinjewadi Phase I, II & III by the volunteers The people of various age groups were interviewed roadside by requesting them to answer the questions. A total of 129 commuters were interviewed including students, business professionals, servicemen, retired professionals and homemakers. The responses were collected from the volunteers [2].

Online Questionnaire: The second phase was conducted online in parallel with the first phase using Google Sheets. Online Questionnaire the survey was conducted by sharing the Google Sheets link through email, Facebook and WhatsApp. The survey was responded by 50 participants.

The data collected from both sources was collectively analyzed. A total of 129 responses were collected.

5 Data Analysis and Interpretation

The sample size was calculated by using the Random Sampling Technique.

$$\mathrm{n} = \frac{\mathrm{z}^2 \times \mathrm{p}(1 - \mathrm{p})}{\varepsilon^2 \mathrm{N}}$$

where Z = Z Score
ε = Margin of Error
N = Population Size
p = Population Proportion

The population size is 1,15,248 which is daily trips to Hinjewadi. The confidence level is 85%, so Z = 1.44. Margin of error is 5%. Considering the 50% population proportion the sample size is 207.

5.1 Data Analysis

Data Analysis was conducted by various criteria like gender, monthly income, travel slab, age group, frequency of travel, purpose of trip, travel time saved and metro fare hike [3].

5.2 Interpretation & Results

(a) A total of 207 questionnaires were filled in the Roadside Interview Survey and online interview survey. The respondent consisted of 61% men and 39% women. Out of the total respondents, the age range of respondents was 33.3% belonging to 21–25 years, 47.5% belonging to 26–35 years, 14.2% belonging to 36–45 years and 5% belonging to 46–60 years.
(b) 10.4% of the respondents were students; 33.5% civil servants; 42.7% worked in the private sector; 9.2% worked at home and 4.2% were unemployed.
(c) The majority of respondents had access to a private vehicle (83.3%). Many respondents commuted frequently (92.1%). The primary transport mode of choice was motor motorcycle (60.4%), bus (17%), car (8.7%), auto (3.6%), and cab (2.5%).
(d) 7.7% were using a combination of different travel modes.
(e) Out of the total respondents, 51.8% used public bus transport rarely, 20.5% used public bus transport on a daily basis, 15.5% used public bus transport 2–3 times a week and 5% never used public bus transport.

Gender Preference Analysis It is observed that 77% of the respondents were male and 23% of the respondents were female. 84% of the male respondents will definitely choose the metro. 93% of the female respondents will definitely choose the metro (Fig. 2).

Monthly Family Income Preference Analysis It is observed that 21% of the respondents belong to the 10–20 k monthly family income group out of which 99% of the respondents will definitely choose the metro. 27% of the respondents belong to the 20–30 k monthly family income group out of which 68% of the respondents will definitely choose the metro. 12% of the respondents belong to the 30–40 k monthly family income group out of which 98% of the respondents will definitely choose the metro. 19% of the respondents belong to the 40–50 k monthly family income group out of which 96% of the respondents will definitely choose the metro. 21% of the respondents belong to the above 50 k monthly family income group out of which 90% of the respondents will definitely choose the metro (Fig. 3).

Travel Slab Preference Analysis It is observed that 8% of the respondents belong to 1–5 km. travel slab out of which 80% of the respondents will definitely choose the metro. 23% of the respondents belong to 6–10 km. travel slab out of which 67% of the respondents will definitely choose the metro. 30% of the respondents belong to 11–15 km. travel slab out of which 97% of the respondents will definitely choose

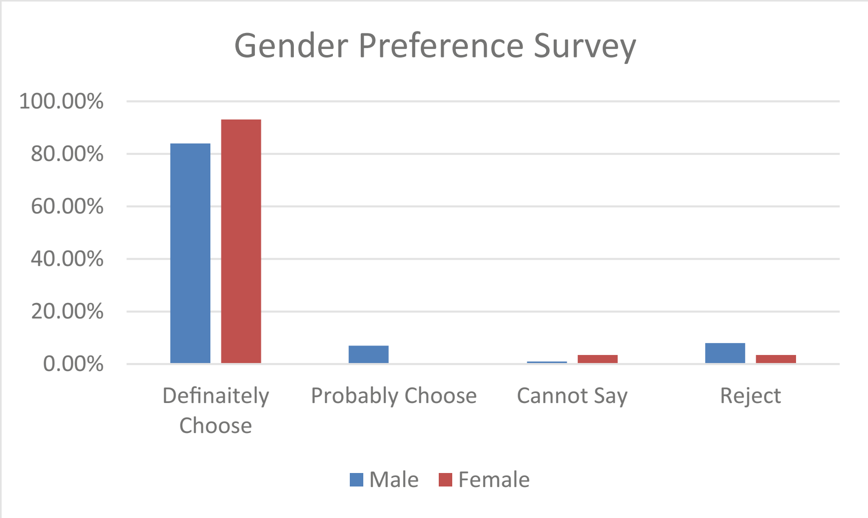

Fig. 2 Gender preference analysis

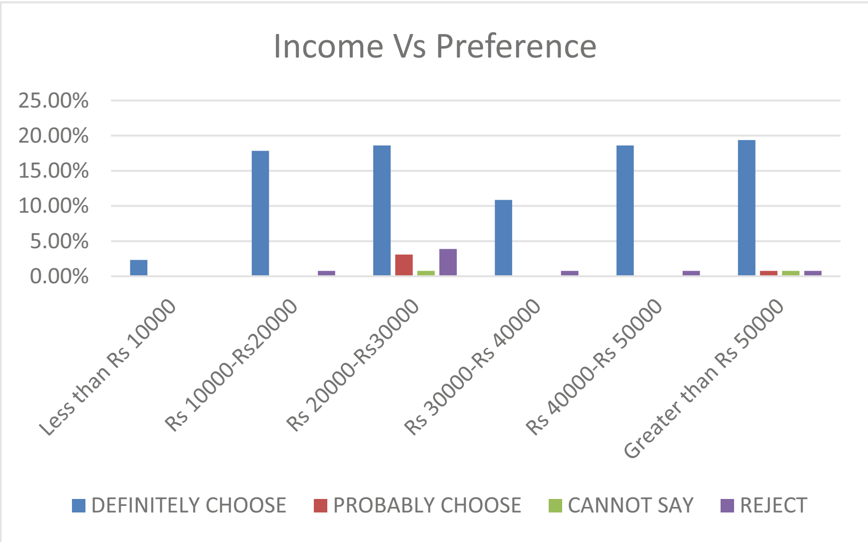

Fig. 3 Monthly family income preference analysis

the metro. 14% of the respondents belong to 16–20 km. travel slab out of which 83% of the respondents will definitely choose the metro. 24% of the respondents belong to 21–25 km. travel slab out of which 96% of the respondents will definitely choose the metro (Fig. 4).

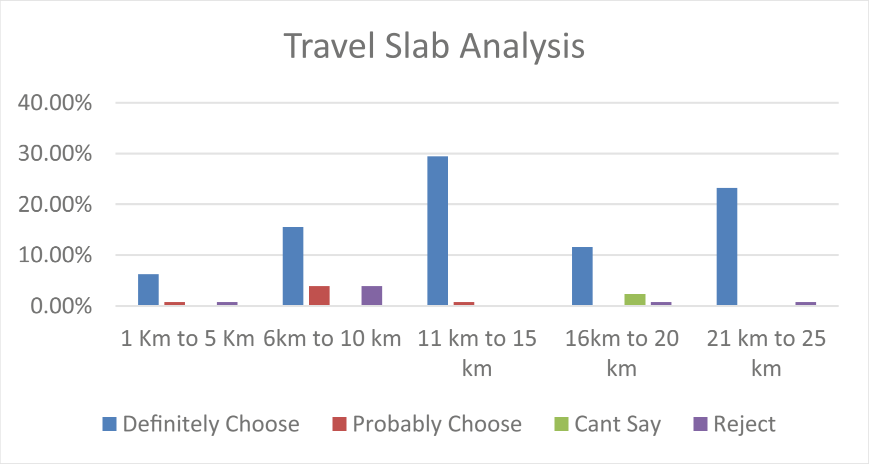

Fig. 4 Travel slab preference analysis

Age Group Preference Analysis It is observed that 79% of the respondents belong to the age group of 21–30 years out of which 85% of the respondents will definitely choose the metro. 16% of the respondents belong to the age group of 31–40 years out of which 90% of the respondents will definitely choose the metro. 3% of the respondents belong to the age group of 41–50 years out of which 100% of the respondents will definitely choose the metro (Fig. 5).

Frequency Preference Analysis It is observed that 83% of the respondents will travel 5 days a week out of which 87% will definitely choose the metro. 6% of the respondents will travel 6 days a week out of which 75% will definitely choose

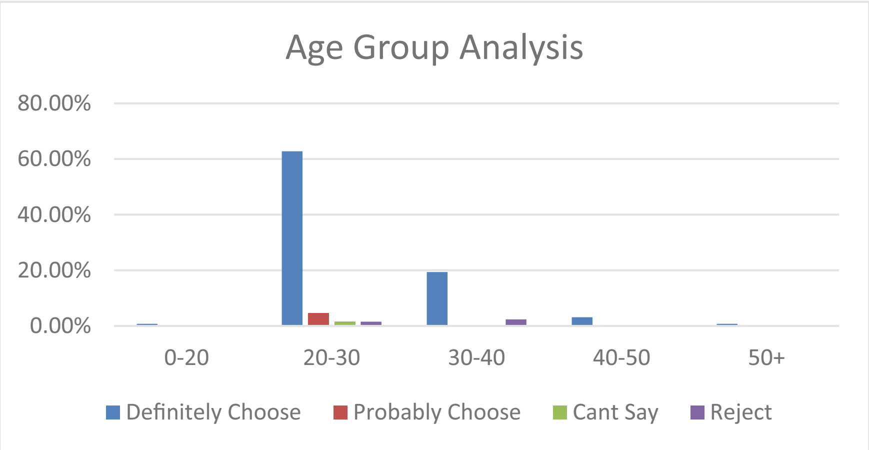

Fig. 5 Age group preference analysis

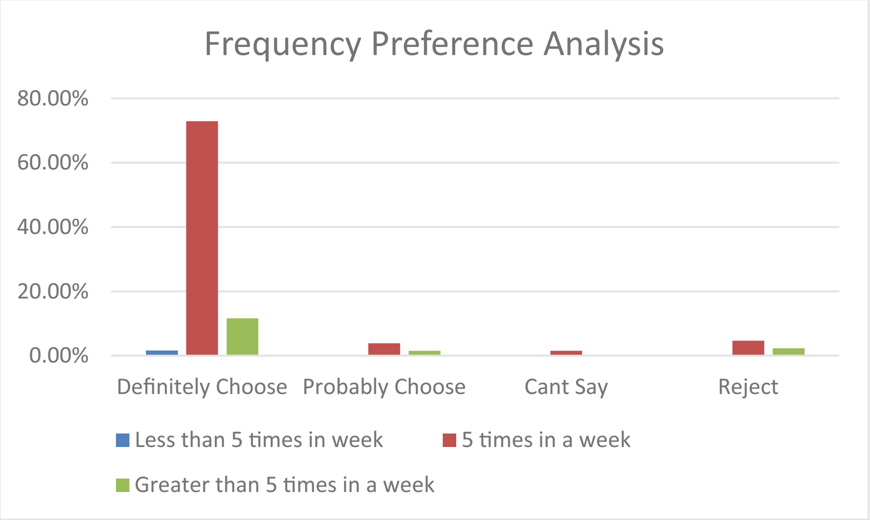

Fig. 6 Frequency preference analysis

the metro. 9% of the respondents will travel 7 days a week out of which 83% will definitely choose the metro (Fig. 6).

Purpose of Trip Preference Analysis It is observed that 86% of the respondents will travel for the purpose of work out of which 87% of the respondents will definitely choose the metro. 9% of the respondents will travel for the purpose of returning home, out of which 83% of the respondents will definitely choose the metro. 3% of the respondents will travel for the purpose of business out of which 75% of the respondents will definitely choose the metro (Fig. 7).

Travel Time Saved Preference Analysis It is observed that 85% of the respondents will definitely choose the metro if their travel time is saved. 6% of the respondents will probably choose the metro if their travel time is saved. 3% of the respondents cannot say whether they will choose the metro or not and 6% of the respondents will reject the metro (Fig. 8).

Metro Fare Hike Preference Analysis from the above Bar Chart, it is observed that 86% of the respondents will definitely choose the metro even if the metro fare price is hiked as compared to the public bus transport. 5% of the respondents will probably choose the metro at a hiked fare price. 3% of the respondents cannot say whether they will choose or not and 6% of the respondents will reject the metro if the prices are hiked (Fig. 9).

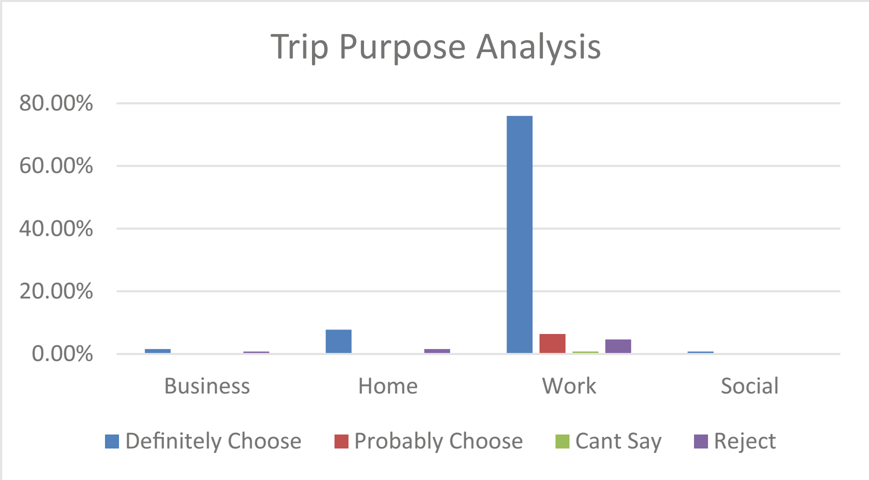

Fig. 7 Purpose of trip preference analysis

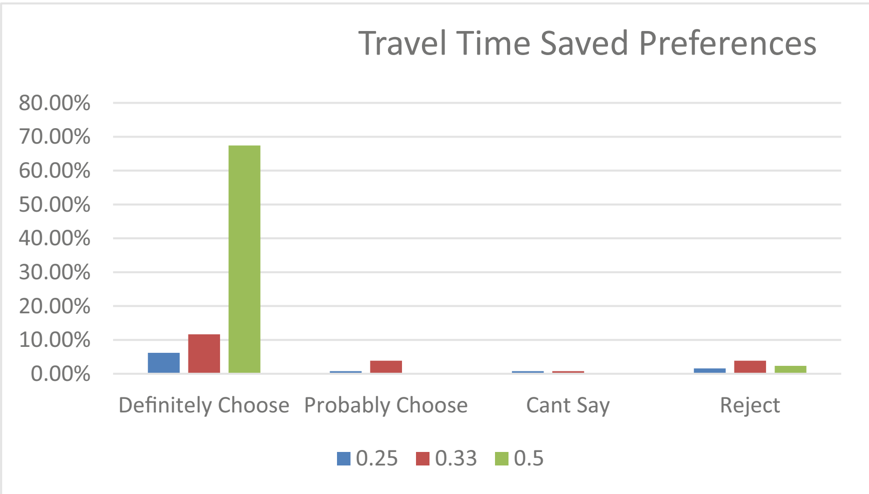

Fig. 8 Travel time saved preference analysis

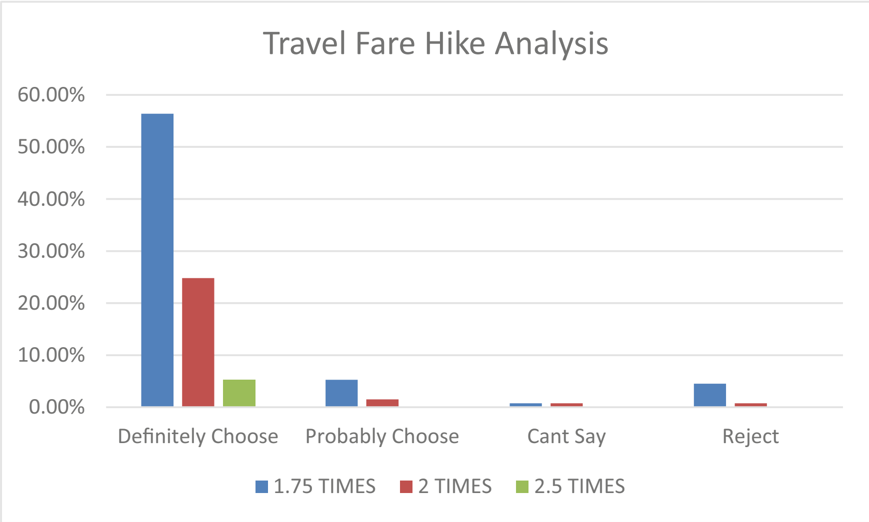

Fig. 9 Metro fare hike preference analysis

6 Conclusion

Male commuters and female commuters willing to switch to the metro are 84% and 93%, respectively. Commuters willing to switch to the metro for traveling 1–5 km, 6–10 km, 11–15 km, 16–20 km and 21–25 km distance are 80%, 67%, 97%, 83% and 96%, respectively. It was noticed that for distances below 10 km, people prefer their previous means of transport. People are interested in saving time and hence if more time is saved then people will prefer the metro for that trip. 83% of commuters will definitely choose the metro if travel time is reduced to 50%. It was also noted that people are choosing metro generally for their work thus frequency at peak hours must be high compared to non-peak hours. 87% of commuters traveling for work will definitely choose the metro. Commuters willing to prefer the metro belong to the age group 21–30 are 85%, age group 31–40 are 90% and age group 41–50 are 100%.

Mass Rapid Transit will be a major mode of transport in the modal split for work trips to Hinjewadi where the commuters are traveling long distance, above 10 km. Other commuters travelling distances of less than 10 km shall be provided with a modal split of other modes of transport such as Bus, Non-motorized Transport, bicycle, and walking.

This study shows that the metro will be preferred by commuters for work trips to Hinjewadi. In order to achieve the modal shift of private vehicles to public transport, the metro will play a major role as a mass rapid transit system. Thus, metro will be a successful project and play a major role in achieving a modal shift as it reduces a lot of travel time. Policies are to be made related to the cost of travel so as to minimize as much as possible to enhance the use by lower income groups as well. Multimodal transport system shall be enhanced to improve the efficiency of the metro system.

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Passengers Perception and Satisfaction Level Towards Public Transport: A Review

Arjun Singh Lodhi and Anuj Jaiswal

Abstract Transportation is an essential component of human life as well as any country's economic development, such as commercial, industrial, and service sectors. It facilitates the transfer of people and objects from one place to another. A well-functioning transportation network is essential for a developing country's future economic success, such as India. People pick, organize, and interpret information to construct a meaningful image of the world through perception. The same stimulus can be perceived differently by various people. The aim of the paper is to understand the association between passenger's perception and satisfaction using a comprehensive review approach. The literature on the study's core sections, perception, expectation, service quality, and passenger happiness, is the focus of this work. The study covers the literature on customer perception and satisfaction, as well as measures to improve public transportation service quality. The literature will help in identifying the governing parameters further for survey data collection.

Keywords Perception · Satisfaction · Public transport · Service quality · Passenger

1 Introduction

Transportation is an essential component of human life as well as any country's economic life. Commerce, agriculture, and the service industry all rely on it to survive. It aids in the movement of people and objects from one location to another. A well-functioning transportation network is a must for a rising country like India's future economic progress. Public services can be described as the provision of services or meeting the needs of individuals or groups with an interest in the organization while adhering to the organization's rules and procedures. Customer perception is a user's assessment of a service after utilizing it and comparing it to what he

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expected and experienced previously. People pick, organize, and interpret information to construct a meaningful image of the world through perception. Consumer perception of service quality has been studied by many scholars and researchers who have shown that they are positively related to satisfaction and brand image [4, 16, 30]. "Satisfaction is the consumer's fulfilling response," according to one definition. It's a determination that a product or service feature, or the service product itself, provided (or continues to give) a pleasurable level of consumption-related satisfaction, including levels of under- or over-fulfillment" [12]. This paper looked at the literature on the study's three main components: service quality, customer perception, and expectation. The following major points were highlighted in particular: 1. Service Quality, 2. Customer Expectations, 3. Customer Satisfaction, 4. SERVQUAL Model and related consumer satisfaction and service quality. Service Quality: Service quality is considered to be a key factor in service sector performance, that is, profitability. Service quality not only entices new customers away from competitive firms but also induces customers" repurchase intentions [32, 33].

Satisfaction can be defined as a post-consumption evaluation of quality of services [2]. It is the expression of the mental state that arises after encountering a service and contrasting that with previously held experiences [19]. Satisfaction is positively related to perception of the performance of any service. Besides the managerial measures, satisfaction is also used as a standard of service performance [12, 26]. Citizen satisfaction plays a crucial role in deciding public services like public transport services. Many scholars explored the relationship between demographic traits (age, income, gender, race, etc.) and satisfaction. For instance, Brown & Coulter [5] found that the Blacks were less satisfied with government services.

While customer satisfaction is a short-term service-specific experience, service quality is a long-term cognitive process that encompasses both the way services are delivered and the results achieved. To assess service quality for customer satisfaction, various conceptual models have been developed by researchers over the years. Therefore, service quality and customer satisfaction are interrelated concepts, but they are not the same. Parasuraman, Zeithaml, and Berry [20] define service quality as the expectations and perceptions of customers regarding a service. Lehtinen and Lehtinen [15] suggest that service quality can be evaluated by assessing the way services are delivered and the outcomes of those services. Government services can be assessed by the experience and satisfaction of the citizens [11].

After 1970, many transportation planners and researchers began experimenting with different approaches and techniques for developing different models to estimate transit system performance in terms of user perception. Because service quality is a qualitative parameter, modeling qualitative parameters is more difficult. The following sections discuss service quality measurement, and models proposed by various researchers for various systems. There are different types of methods for performance evaluation models such as SERVQUAL, Impact Score Technique (IST), Important Ordered Logit Model, Structural Equation Modeling (SEM), Soft Computing Techniques, RECSA model, Multivariate regression analysis, Factor Analysis, Likert scale. SERVQUAL and SEM both are the most popular in the review literature. According to the literature study both methods have given good results that is why the methods most popular.

2 Findings and Discussion

Public transportation plays an essential role in many people's daily lives, providing affordable and convenient transportation options. Passengers' perception and satisfaction with public transportation are crucial to ensuring that these services meet their needs and expectations. However, it can be challenging to maintain high levels of satisfaction among all passengers, as perceptions of quality and satisfaction can vary widely depending on individual experiences and circumstances. Jun [8], Diana [10], and Lierop [31]: Jun has investigated public transit. In his study, he examines consumer satisfaction with reference to the caliber of the public transportation service. SERVQUAL was applied across five dimensions as a measure of customer satisfaction. The most crucial elements that contribute to exceptional service quality are identified by this study. Diana [10] even beyond more immediate marketing objectives, it is clear that a public transportation provider needs to evaluate customer satisfaction. This study aims to show how satisfaction indicators may be used to understand how individual attitudes, transportation use, and the urban environment interact. Users of both private automobiles and public transportation reported high levels of satisfaction with urban transportation networks, according to a representative sample of Italian multimodal travellers. As per Lierop [31], retaining users of public transit is a challenge for many municipalities. It is essential to comprehend the elements of public transportation that affect user's adherence to the system in order to create complete policies aimed at retaining riders. This research summarizes the literature on the variables that affect patronage and satisfaction with public transportation. According to research findings, the service factors most closely associated with satisfaction include on-board comfort and cleanliness, operator courtesy and helpfulness, safety, punctuality, and frequency of service.

Oña [9]: The findings of this study can be utilized to generate policies and suggestions to encourage more people to use public transportation instead of driving their own cars. This modal shift can be accomplished by imposing limits on private vehicles or enacting policies that improve people's contentment with public transport. According to Cheng [7], bus transit is one of the most common ways of transferring passengers at high-speed railway stations in China. This article focuses on the transfer experience of passengers and presents a methodology for assessing existing bus traffic transfer services. The associations between bus transfer service, passenger perceived value, and passenger satisfaction are investigated using a structural equation model. Results of the analysis suggest that cost and convenience are the most important elements influencing customer happiness. Morton [17], a quality of service indicator used by the Scottish Government to gauge passenger attitudes regarding bus transportation is an 11-item opinion scale. The latent constructs included in this scale are identified via factor analysis. According to the findings, attitudes toward bus service quality vary significantly across passenger groups. Perceived bus service convenience appears to have a strong positive explanatory power over perceived bus service satisfaction. Increased service frequency, availability, reliability, and stability will likely raise perceived contentment among existing passengers.

Rahim et al. [22] employed a quantitative technique to investigate how users' perceptions affected the choice of buses as public transportation versus trains or airplanes. Users' opinions were gathered from a sample of 384 respondents who travelled to their destinations via express bus. Their research found that encouraging people to take public transportation is significantly influenced by accessibility, convenience and safety. Additionally, public transportation policies had a big impact on how public buses were modified. Raj et al. [23], Customer Perceptions on Using Public Transportation was the subject of a study. 100 people who use public transportation were given a standardized questionnaire. The investigation concludes that the majority of individuals prefer public transportation to owning vehicles. It has been discovered that public transportation consumer's discontent reduces their desire to use it.

Sinhaa [28], the report examines public transportation in city buses and the Bus Rapid Transit System in Ahmedabad, India. Findings show that important service quality characteristics are consistent across public transportation modes. Passenger expectations, according to the report, are linked to current service quality levels. Users are more worried about basic service qualities and less attentive to 'higher order' quality indicators when service quality is low. Singh [27], this research has three key objectives. It begins by attempting to gauge customer satisfaction with public bus transportation in the Indian city of Lucknow. Second, it aims to investigate the factors that influence passenger satisfaction with.

According to the findings of studies by Ramu and Gurtoo [24], commuters in Lucknow are generally unsatisfied with public bus services. The importance of road transit as a means of transportation cannot be overstated. As a result, ensuring that this service is of high quality is vital. Researchers look at the characteristics of intercity bus passenger transportation in Europe and India. Factors such as women's friendliness and ticket price affordability have a significant impact on commuters' perceived service value. In Europe, good drinking water and clean bus stops are more important, while in India, luggage handling is more relevant and tangible. According to supplementary studies, in both cases, technology has a major impact on satisfaction. Chaudhary [6], in today's society, urban public transportation is becoming increasingly important. Surat was determined to be the best of the three cities in terms of service quality, followed by Rajkot and Ahmedabad. For such initiatives, it is critical to capture the demand side quality of services, hence an attempt has been made to see if commuters' impression of the quality of services provided by Bus Rapid Transit systems varies across demographic cohorts. This is demonstrated by its inclusion in the United Nations Development Program's Sustainable Development Goals as the eleventh aim, 'Sustainable cities and communities.' Cities' competitiveness is primarily determined by the reliability of their transportation systems. The SERVPERF model was used to investigate the performance of the city's BRT networks.

Linda Too and Earl [29], the purpose of this study is to introduce and use a SERVQUAL framework to measure public transport services within a master-planned community in Australia. The findings have been useful in shedding broad light on the areas where improvements are needed most, i.e. responsiveness and reliability of services, to encourage greater use of public transport within the community level. Kumar [13], Informal public transport modes like Tata Magics, Vikrams and mini-buses cater to the mobility needs of the population in Indian cities. This sector is not sufficiently acknowledged for the important contribution that it makes to mobility supply. The paper highlights that these systems bridge a large transport supply gap and play an important role in India's cities. Barabino [3] Purpose of the paper is based on a modified version of EN 13816, a European standard on service quality in public transport. The results illustrate a high degree of importance placed on attributes such as on-board security, bus reliability and cleanliness. Moreover, the SERVQUAL framework might be improved with the inclusion of additional attributes.

Amponsah and Adam [1] found that transport study of Vancouver Lower Mainland in the Province of British Columbia, Canada. A judgment sample of 205 was collected from an urban population of the Translink system for the study. The main findings of the study showed a significant relationship between service quality and customer satisfaction. Late-night bus services had a significantly negative effect on overall satisfaction. The SERVQUAL model, noted for its robustness in measuring customer satisfaction, was adapted for the study. Mounica [18], the overall satisfaction of the customers on Tirupati public bus transport is below average. Correlation analysis was performed on the data obtained and it shows that certain attributes show a strong correlation with the overall satisfaction with the bus service in Andhra Pradesh. Based on the findings some suggestions are proposed for policy recommendations.

Ponrahono [21], this research highlights the urban–rural bus services passenger satisfaction level in the selected settlements in Peninsular Malaysia. The main objectives are to evaluate the bus service quality through passengers' satisfaction surveys. The result shows socio-demographic and trip characteristics influenced the satisfaction level and passengers' expectations of future bus service improvements. Roberto et al. [25], study of public transport in Sweden identifies and characterizes current and potential travellers' satisfaction with public transport services. An analysis is based on a dataset of almost half a million records from public transport authorities and operators. Three key attributes that should be prioritized by stakeholders are identified: customer interface, operation and network and length of trip time.

Kumar et al. [14], this study measured the level of customer satisfaction with the quality of services offered by the Uttar Pradesh State Public Transport Corporation (UPSRTC). The study was conducted on over 2,000 passengers between June 2015 and October 2015. A high occupancy public bus transport service in some countries has gradually dropped due to its less competitive service performance. Using two approaches, a sample of 816 public bus users completed a questionnaire on perception and expectation measures at terminals in six regional capitals. The result of the analysis shows that there is a wide gap between commuters' expectations and public bus transport quality attributes. This explains that passengers' satisfaction would be enhanced when transport operators accommodate their expectations of service. The coefficient between satisfaction and usage of bus transport, and the variance explained in these variables (satisfaction and usage) by the expectation model is stronger compared to the perception model. Due to the scarcity of previous research on the subject, many of the findings from this analysis are indicative rather than conclusive. The fact that the methods utilized and factors controlled for in the analyzed studies vary further limits the ability to draw firm conclusions. Nonetheless, the review gives an overview of quality studies in regional public transportation and highlights several quality traits that have been shown to be essential in these studies on a regular basis. It's important to note that the way quality attributes are defined and applied differs amongst the studies evaluated. This makes the evaluation more difficult, but by adapting the methodology described in Section Service quality attributes, we were able to compare research and uncover some general trends. One critical issue that affects passengers' perception and satisfaction levels towards public transport is the reliability and punctuality of the service. Passengers often rely on public transportation to reach their destination on time, whether it be for work or personal reasons. If the service is consistently late or unreliable, it can lead to frustration, inconvenience, and dissatisfaction among passengers.

3 Conclusion

By reviewing several literature, the authors concluded that due to rise in fuel prices and rise in air pollution people prefer to travel by public transport system in urban and rural part of the countries in the world. Proving more accessible services, and public transportation appealing to users. This review study has analyzed satisfaction levels regarding public transport in general and has not differentiated between modes of transport. However functional aspect significant impact on customer satisfaction and requires more attention in order to improve it. As per prior studies concluded that pricing, punctuality, and trip time affect a better degree of satisfaction. Moreover, observation of passengers is a typical and effective method of determining the customer's needs and how to meet them. Although customer satisfaction has been thoroughly analyzed in developed countries that follow lane-based traffic conditions. Additionally, it has observed limited research conducted in non-lane traffic conditions like India. Existing research has been focused to develop a marketable and appealing public transportation system. Moreover, research requires proposing to generalize the results and broaden the study by analyzing other areas with different characteristics (e.g. countries, network size, etc.). While increasing the sample size and the number of questions in the survey comparisons could be made between how the public perceives different modes of transport.

According to the structure of quality attribute categories, the most commonly investigated categories are cost, availability, time, and comfort, which are covered by more than half of the analyzed studies. The most extensively discussed category is availability, with a focus on modalities (bus vs. rail) and network (coordination, transfers, access and egress). However, there are no details on the operational element of availability, such as working time and frequency. The examined studies have focused on reliability in the time category. There are no elaborations on trip time, such as studies on travel time improvements in regional public transportation. Accessibility, information, customer service, security, and environmental impact are all mentioned in earlier research, but they don't go into detail. Environmental impact and its impact on mode choice, ridership, and customer satisfaction is a category that has received little attention.

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Effect of Land Use Pattern on Bus Blockage Duration at Curb Side Bus Stops

Remya K. Padinjarapat, Darshana Othayoth, K. V. Krishna Rao, and Tom V. Mathew

Abstract A bus stop is a strategic location considering congestion and delay in an urban road network. The influence of a bus stop on the traffic stream is generally characterised by the average dwell time. However, dwell time alone cannot be a complete measure of the influence of a stopping bus on the traffic stream. The duration of the bus to decelerate and accelerate to the required stream speed needs to be taken into consideration. The bus blockage duration that takes these factors into consideration gives a better picture of the effect of a bus on the stream. Again, the dwell time is usually correlated to the number of boarding/alighting. However, the other factor that influence passenger behaviour is the activity around the vicinity of the bus stop. These activities influence the passenger behaviour and the bus frequency and are a good measure of a combination of these various factors. Thus, in this study, the various factors influencing bus stops are studied in detail based on bus arrivals, departures and dwell time data collected from 1651 buses along 11 bus stops in the Mumbai region. The land use activity near the bus stop and its influence on the bus blockage duration is the prime contribution of this paper.

Keywords Land use · Bus blockage · Curb side bus stops

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1 Introduction

The heterogenous traffic is characterised by a mix of vehicles with varying static and dynamic characteristics. The non-lane following traffic stream resulting from this mix is easily affected by minor blockages, bottlenecks, etc. Bus stops which are a prominent feature in a road network are predominant locations of bottlenecks. Most of the bus stops seen on the road network are curb-side bus stops. A bus stopping at the curb not only reduces the width of the roadway for the remaining traffic, but also affects the following vehicles. These bus stops thus become locations of congestion and hence reduce the capacity of the urban arterial. Thus, it is necessary to study the effect of bus stoppage duration on the traffic stream. An important variable that defines this effect is the bus blockage time. This is the time during which a stopping bus blocks the free flow of the stream. The effect of the bus stops will be different according to the frequency of buses, number of boarding/alighting, etc. These factors depend on the land use type of the area in which the bus stop is located. The present study aims to identify the effect of bus stops in terms of bus stoppage duration for various land use types.

2 Literature Review

Among the various bus stops generally seen, Koshy and Arasan [1] found that the curb side bus stops most affect the traffic stream with a nearly 25% reduction in stream speed. The curb side bus stops create bottlenecks on the urban streets with buses blocking a certain portion of the roadway. They also tend to block the following vehicles causing them to stop until the bus starts moving after the dwell period. The dwell period is thus seen to affect the traffic speed and impact the overall travel time.

Various studies have been done to estimate the relationship between boarding/alighting and dwell time. Sun et al. [2] studied smart card data to find the impact of boarding/alighting on dwell time. Zang and Teng [3] studied the dwell time patterns using automatic vehicle locations and automatic passenger counts. These studies and many others [4–8] have associated the impact of the number of boarding/alighting with the dwell time of the buses. The number of boarding/alighting is proportionally related to the dwell time. Other factors that are found to influence dwell time are vehicle type, time of day, service frequency, stop location, waiting time, passenger behaviour, etc. A majority of these factors like stop location, service frequency, waiting time, etc. depend on the activity in the vicinity of the bus stop locations. Gokasar and Cetinel [4] have suggested clustering of dwelling patterns based on the properties of the area around bus stops. Thus, land use type near the stop location is a good indicator of these parameters combined.

In order to study the complete effect of a stopped bus on the traffic stream, the dwell time alone is not sufficient. The bus starts to influence the stream, and the movement starts to decelerate. The duration of time the bus affects the traffic stream includes the dwell time and the acceleration and deceleration time [9]. Thus, to estimate the impact on the traffic stream, the duration for which the bus blocks the free flow speed is to be taken into consideration. The Highway Capacity Manual [10] suggests a blockage duration of 14.4 s at signalised intersections. The bus blockage duration will give a complete picture of the effect a single bus has on stopping at a bus stop. Thus, this study is focused on illustrating the influence of land use patterns on bus blockage duration.

3 Field Data Collection

The bus stops were selected such that they were located in different land use types. The land use types considered for the study are—Office/Commercial spaces, Educational spaces, Residential areas, Transit change point locations and Recreational/Shopping areas. Transit point location indicates bus stops located near any other public transit mode. The blockage time survey was conducted at 11 bus stops in Mumbai city in India. Figure 1 shows the location of the bus stops at which the surveys were conducted. All the bus stops were curb side bus stops and did not have separate bus bays or bus lane markings.

Fig. 1 Locations of bus stops selected for data collection

At each location, the survey was conducted on a typical day for a duration of 3 h including peak hours, either during morning—8:00 AM to 11:00 AM—or evening—5:00 PM to 8:00 PM. Two surveyors were located at each bus stop for the conduct of the survey. The surveyors would note down the following details at each bus stop:

1. Bus number and bus type
2. Number of doors for boarding and alighting
3. Clock time of arrival of the bus at the bus stop including deceleration time
4. Clock time of departure of the bus from the stop including acceleration time
5. Number of commuters boarding and alighting
6. Approximate number of persons standing on the bus.

The data of 1651 buses were collected from the 11 bus stops. Among all the buses surveyed, it was seen that there were two types of buses plying the study area—air-conditioned and non-air-conditioned buses. Among the two, 89% of the buses plying in the study area were non-air-conditioned buses, while the rest 11% were air-conditioned buses. The non-air-conditioned buses have two doors for the commuters to board/alight the vehicle. Both the doors were always open. It is observed that the front door is used by passengers for alighting and the rear door for boarding. The air-conditioned buses had a single door for boarding and alighting, which was switch-controlled. The approximate number of people standing in the bus was also noted to get the approximate measure of crowding inside the bus.

While noting the clock time for the arrival of a bus, the time was noted from the point when the bus started to decelerate. Again, the clock time for departure was noted when the bus accelerates to the stream speed. The difference between the clock times of arrival and departure gives the total time a bus blocks the portion of the traffic stream. The difference in arrival clock times of two buses gives the headway between the buses at the bus stop. Based on the details collected at the survey locations, the blockage time analysis was done.

4 Blockage Time Survey Analysis

The bus stop localities pertaining to similar land use patterns were grouped together for analysis. Table 1 shows the details of the buses collected at the different bus stops aggregated based on land use. The bus stop locations were evenly selected to cover a majority of the land use type seen in the study area. It is observed from Table 1 that the mean headway at transit point locations is low since they are one of the most frequented places by commuters to shift from one mode to another. Again, in residential areas the headway of buses is the highest, indicating a lower frequency of buses since the ridership in such areas will be less. This is reflected in the smaller number of samples obtained from such locations.

The blockage time for each bus is noted from the field data collected. The blockage time includes the time for deceleration, time taken by commuters to board and/or alight the bus and time taken for acceleration. It is assumed in the study that the time

Table 1 Details of buses aggregated based on land use at the bus stop location

Land use	Number of bus stops	Sample size	Mean headway (sec)
Commercial/Offices	2	232	79.88
Educational	2	511	41.68
Residential	2	75	267.19
Transit change point	2	346	59.41
Recreational/Shopping	3	487	62.74

taken for a commuter to board or alight is the same. Thus, the total time taken for all boarding/alighting in a bus will depend on the number of commuters and the number of doors. In buses with two doors, boarding and alighting take place simultaneously, unlike the buses with single door.

In order to investigate the relationship between the blockage time and the number of boarding and/or alighting, the actual number of boarding/alighting that affect the dwell time needs to be estimated. In buses with two doors one each for boarding and alighting, the time taken for all the boarding and alighting will be the time taken for either boarding or alighting whichever is larger. Hence, the larger of the two is taken to affect the dwell time in a case. However, in buses with single doors, the total time taken for boarding and alighting will be affected by both the number of boarding as well as number of alighting, as the commuters will use the same door for both purposes. Thus, in such a case the number affecting is the sum of the number of boarding and alighting.

Figure 2 shows the plot between the number of boarding/alighting and the blockage time for various land uses. A linear regression equation is fit to the data points. Linear regression is used as this has been widely used in previous studies. The value of the regression coefficient, however, is low in all the cases. This is due to the spread in the scatter points along the y-axis which is the blockage time. As stated earlier, blockage time includes the acceleration as well as deceleration time apart from dwell time which results in a larger spread of the points along the y-axis. In order to estimate the exact relationship between the dwell time and the number of boarding/alighting, the analysis needs to consider various other factors associated with dwell time like frequency of buses along similar route, commuter arrival patterns, waiting duration, etc. This is however beyond the scope of this paper. Although Fig. 2 is not able to convey the exact relation between the blockage time and the number of boarding/alighting, it is able to establish a linear relationship between the two. Even though, linear regression models may not give a complete picture, they are widely accepted in establishing the relation between boarding/alighting and dwell time. Thus, the relationship described in Fig. 2 helps in validating the data collected by establishing the linear relation between the two.

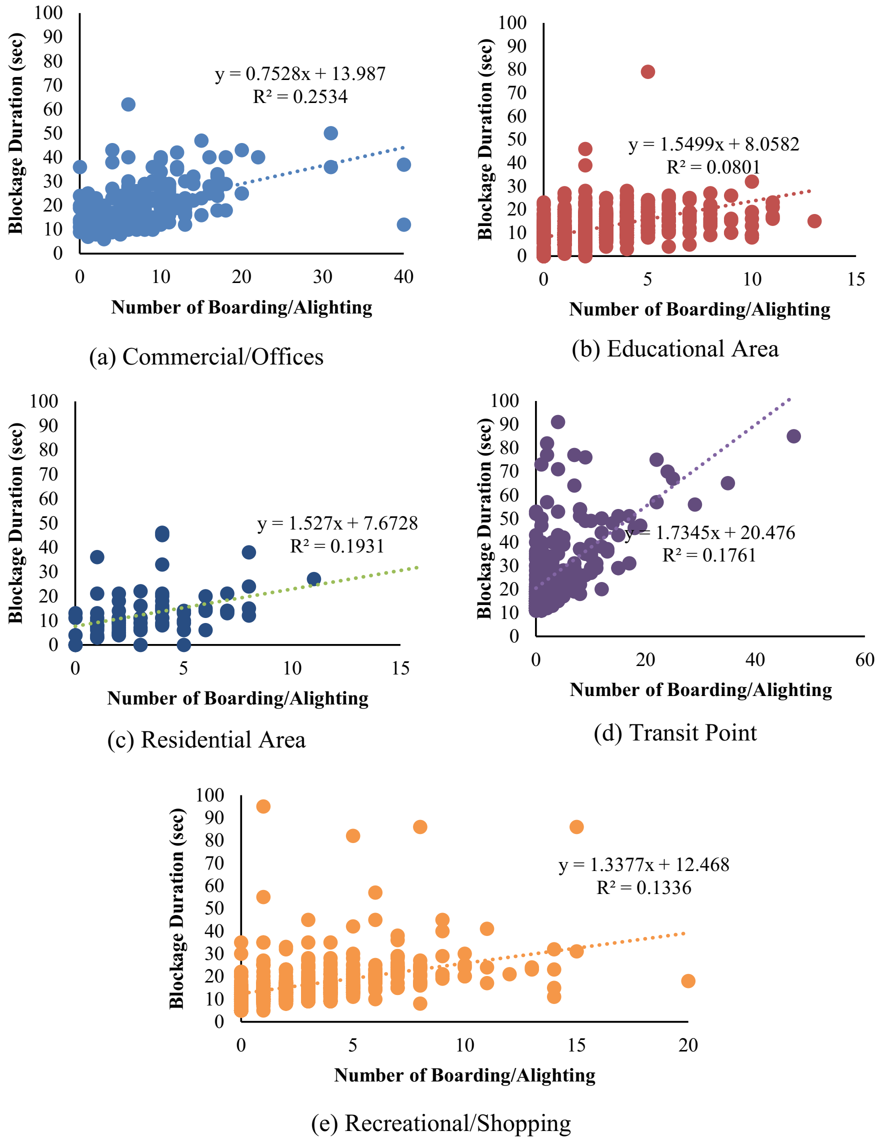

Fig. 2 Number of boarding/alighting versus bus blockage duration

4.1 Bus Blockage Duration

The duration for which a bus blocks the free flow of the traffic stream is analyzed for various land use types. The mean value of blockage time seen for various land uses is shown in Table 2. The blockage time at transit point locations is due to a

Table 2 Mean blockage duration for various land uses

Land use	Mean blockage (sec)
Commercial/offices	19.44
Educational	13.00
Residential	11.73
Transit change point	29.08
Recreational/shopping	16.77

larger number of people boarding/alighting the bus. The average blockage time is almost twice as large for these areas as compared to other land use types. Similarly, the blockage time is lesser for residential areas, as the number of commuters in the region is less. However, the mean values alone do not give the complete picture.

The frequency of blockage time at various land uses is described in Fig. 3. A visual interpretation of the plots indicates that the frequency distribution follows a normal distribution for all land use types. This is further confirmed using the KS, test which failed to reject the null hypothesis that the data comes from a normal distribution with a mean and a standard deviation. At transit point locations although the mean duration of the bus blockage is high, the spread of the blockage time is also larger than other land use types. This indicates that the probability of the bus stopping for a longer duration is higher in such areas. These locations are thus critical and are frequent locations of congestion and delays due to buses stopping at curb side. It is thus important to design separate bus bays at these locations so as to reduce the congestion caused due to bus stopping.

Although the blockage duration is found to be normally distributed, for practical purposes and for use in various models, a specific value is required. It is suggested to use the mean value for various land uses. In case the land use pattern is not known, or for general application, a weighted mean of 18 s is suggested.

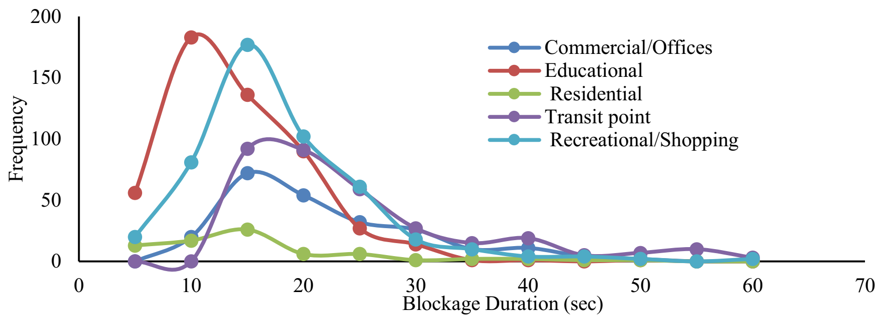

Fig. 3 Frequency distribution of bus blockage duration

5 Conclusions

The curb side bus stops on urban streets tend to create disturbance in traffic streams and reduce the capacity of the streets. In order to estimate the effect of these bus stops, it is important to determine the blockage duration due to a single bus stopping. This study was conducted to estimate the bus blockage duration in different land use types. The study was conducted in the Mumbai area with peak hour data from 11 curb side bus stop locations. The blockage duration was found to be starkly varying depending on different land uses. The bus stops located at transit points were found to be very high due to the higher number of commuters boarding and alighting. Thus, these locations are critical points of congestion and delay to the traffic streams. Providing bus bays at such locations can highly help in improving the traffic conditions in these land use types.

The blockage time due to a single bus stopping at a bus stop is used to estimate the reduction in capacity, as an adjustment factor for saturation flow estimation, etc. The value of this blockage time is found to be land use dependent and varies drastically for different land use types. Thus, it is important to incorporate the land use type in the estimation of blockage duration. However, if the type of land use is known for general applications a weighted mean of 18 s is suggested as blockage duration.

The study has its limitations in not considering bus arrival pattern, congestion in upstream section, passenger arrivals, waiting time, traffic flow characteristics, road encroachments, mixed land use, etc. as they were beyond the scope of the study. The effect of these factors on bus blockage duration remains to be studied extensively.

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Spatio-Temporal Factors Affecting Short-Term Public Transit Passenger Demand Prediction: A Review

K. Shanthappa Nithin and Raviraj H. Mulangi

Abstract In public transit (PT) planning, passenger demand forecasting is an important process to periodically update operation management and planning infrastructure in the future. In the past, many researchers considered passenger demand forecasting a fundamental need for transportation planning and developed forecasting models based on statistical methods and Artificial Intelligence (AI). To increase the precision of the model, spatial and temporal attributes that influence the passenger movement at the station level, corridor level, and network level, are need to be considered. Hence, in this study, a detailed literature review is carried out to understand the pros and cons of various methods used in passenger demand forecasting and how distinctively spatial and temporal attributes are used in the development of the models. External factors like weather and events are also considered by the researchers in the development of the model. In the end, what are the challenges in the PT passenger demand forecasting are discussed and directions for future research are given.

Keywords Passenger demand forecasting · Statistical and artificial intelligence (AI) methods · Temporal and spatial attributes

1 Introduction

Public transit (PT) system can be called efficient if it provides comfort and safety to users, meanwhile reducing the operators' costs. However, most of the PT systems' performance in India is not satisfactory due to improper planning and operation. For optimal planning and management of PT, initially, there is a need to understand the passenger mobility pattern and passenger demand distribution. Passenger demand is a key factor for transit network design, frequency setting, and scheduling of services.

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To frequently track the changes in passenger mobility patterns, there is a need to study and understand the characteristics that influence the variation in passenger demand. Using those factors, we should be able to predict current and future passenger demand with high precision. Passenger forecasting can be macro or micro level, where macro-level involves forecasting passenger demand for the long-term which helps transport planners to build permanent infrastructures like roads, design new routes, stations, terminals, management buildings, buying vehicles, etc., whereas micro level involves forecasting passenger demand into weekly, daily or even hourly demand, which helps planners to change schedules, adding new services, increase or decrease frequencies and so on.

Implementation of the intelligent transportation system (ITS) in PT systems all over the world has helped to get automatic passenger count (APC) with the usage of smart cards and electronic ticketing machines (ETM), which has eased the job of planners of collecting data from tedious questionnaire survey which is time-consuming and costly as well. The availability of big data in recent times has increased the research on passenger demand forecasting using various forecasting models. The passenger demand data is a time series that can be analysed using numerous tools.

The models used for forecasting vary in a wide range starting from statistical tools to recent artificial intelligence (AI). The statistical tools involve collecting and interpreting numerical data based on the inference of a sample from the population [1]. Examples of such statistical tools used for forecasting are time-series models like auto-regressive integrated moving average (ARIMA), seasonal ARIMA (SARIMA), generalised autoregressive conditional heteroscedasticity (GARCH), etc., [2–5]. But these statistical tools fail to predict precisely when the data is complex and highly non-linear and also in the case of time-series models, prediction is done based on previous time frames, hence it considers only temporal attributes.

The progress in the field of computer science in recent times has helped to develop a complex AI model. AI models have the ability to learn, adapt, and evolve non-linearity in the data and increase the accuracy of prediction. The most commonly used AI models are machine learning and deep learning. Widely used machine learning models are XGBoost; support vector machines (SVM); k-nearest neighbours (KNN); etc., [6–8] and most commonly deep learning models are artificial neural networks (ANN); recurrent neural networks (RNN); convolutional neural networks (CNN); Bayesian neural networks; etc. [9–14].

To develop a model whose precision is high various temporal and spatial attributes need to be considered. Passenger demand varies at different periods like an hour of a day; day of a week; month of a year. There are various spatial attributes like network topography, land use, and point of interest (POI), geographical which affect passenger demand. Other than spatial and temporal attributes, various external factors are also considered in the literature which has increased the accuracy of the model like population density, weather, events, and so on. The models should be capable enough to incorporate these complex attributes to increase accuracy. Hence, there is a need for a detailed literature review on passenger forecasting models, which helps researchers to understand the recent developments in the prediction of PT passenger demand [15].

The aim of this paper is to present a detailed literature review of passenger demand forecasting models. Analysis of literature has been to get an answer for, (i) how different methods i.e., statistical methods and artificial intelligence (AI) affect the prediction of passenger demand? and how to prioritize them? (ii) what are the different types of input data representations of spatial and temporal attributes? and how it affects the prediction model? (iii) how different external factors have influenced the development of the model? and (iv) what is the scope for future work?

2 Literature Analysis

In this section, the literature is analysed with respect to passenger demand forecasting, with Sect. 2.1 explaining methods used for forecasting, Sects. 2.2 and 2.3 explaining spatial and temporal attributes and external factors influencing passenger demand forecasting respectively.

2.1 Methods Used for Forecasting

As mentioned in Sect. 1, forecasting methods were classified as statistical methods and AI methods and numerous studies have considered these methods to build forecasting models in the past. This section briefly introduces the types of statistical and AI methods that have been used.

Statistical Methods. Statistical methods are the parametric models, which use historical data to forecast the future based on statistics. Time-series models are the most commonly used statistical models for forecasting. Time series is a sequential measure of a variable over time [16]. Historical data is collected and processed and then the model is trained by considering the characteristics of the data and used for future predictions. The most commonly used time-series models are exponential smoothening techniques; a family of Autoregressive (AR) models; Auto-Regressive Integrated Moving Average (ARIMA); Seasonal Auto-Regressive Integrated Moving Average (SARIMA); soft computing techniques; Autoregressive Conditional Heteroscedasticity (ARCH) etc., [17].

Artificial Intelligence (AI). AI or machine learning (ML) models are computationally intensive non-parametric models with self-learning ability. The self-learning algorithm is the one that reduces the error margin by evaluating the results of the explanatory model and fine-tuning it iteratively. Deep learning (DL) networks, i.e. neural networks (NN), which are an advanced version of machine learning have better learning capabilities than other networks due to their nature of nonlinearity, adaptability, and arbitrary function mapping ability [18]. Without prior knowledge of input and output variables, NN has the ability to deal with the nonlinearity of the problem. The most commonly used deep network models are artificial neural network (ANN); recurrent neural network (RNN); convolutional neural network (CNN); long-short-term memory (LSTM).

Statistical methods consider spatial and temporal factors independently without considering the mutual relationship between them. However, PT passenger flow is strongly influenced by the non-linear and complex correlation between spatial and temporal attributes. These hidden complex relationships between input data points can be learned by AI models, hence statistical models fail to predict precisely. The comparison between statistical models and AI models is shown in Table 1.

2.2 Temporal and Spatial Attributes

Passenger demand varies temporally and spatially across the network. The influence of these attributes on passenger flow should be keenly observed and its characteristics should be incorporated into the prediction model to increase the prediction accuracy. Hence in this section, we will discuss how these influential factors have been considered in the literature with examples.

Temporal Attributes. A time series data will usually have seasonality and variations during different periods such as hourly, daily, weekly, and yearly. The season does not mean, the seasons of the year, it represents a constant cycle of rise and fall in the data [17]. The two key attributes of time-series data are cycle and trend. The data usually have a periodic pattern, for example, a bus passenger demand during this time of a day may correlate with the same time on the previous day, which can be called a 24-h cycle, similarly, it may be similar to that of the same day of the previous week or similar to that of same day and the same month of previous year and trend refers to long-term decrease or increase in data [23]. Hence it is essential to consider temporal dependencies while developing the forecasting models.

Temporal dependencies are usually represented in two ways, *sequentially* and *periodically*. In the case of *sequential* representation, passenger counts pertaining to recent temporal intervals are stacked in a vector sequentially. In the case of *periodicity*, depending upon the dimension (daily, weekly or monthly), passenger counts corresponding to temporal units are stacked consecutively for each dimension.

Both statistical and AI methods consider these temporal attributes. For example, [2] considered a 15-min time sequence, daily, and weekly time series and developed ARMA, ARIMA, and SARIMA to capture different features of it. Similarly, [24] decomposed the time-series data into epoch and seasonal components and employed a combination model to forecast future demand. When AI-based studies were considered, [23] incorporated individual layers of LSTM for embedding recent time interval, cyclic, and trend variations into the forecasting models and [21] considered the correlation of adjacent time sequence with the daily cycle and weekly cycle i.e., periodicity. When results were compared, the inclusion of periodicity into the model increased the efficiency of the model when compared to sequential data of recent time intervals. But to incorporate cyclic and trend characteristics models demand a high data

Table 1 Analysis of literature (SM-Statistical Model, PD-Periodicity, TS-Temporal Sequence)

References	Study Area	Model	Temporal	Spatial	External	Remarks
[2]	Shenzhen bus transit	SM	PD and recent TS	–	–	Periodicity increased the accuracy of the model
[26]	Brisbane Metro	SM	Hourly TS	Models for spatial clusters	Weather	Weather had a significant effect on different spatial clusters
[17]	KSRTC Kerala	SM	Daily TS	–	–	–
[24]	Chongqing rail transit	SM	PD	–	–	Decomposition and combination models produced less variance
[9]	Hubli-Dharwad BRTS	SM and LSTM	TS	Stacked vector	–	Accuracy of the LSTM model was higher than SARIMA
[23]	Nanjing Metro	LSTM	Recent TS, PD	Stacked vector	Weather	Daily cycle and weekly trend increased the accuracy
[27]	Singapore Metro	LSTM	Recent TS	Stacked vector	-	Model had better accuracy than the baseline models
[28]	Taipei BRT	LSTM	Recent TS	Stacked vectors having different load profiles	Weather	Model behaves differently with different load profiles
[19]	Qingdao Metro	ANN and LSTM	Recent TS	Land use	–	Land use increased the accuracy of ANN-LSTM compared to regression
[20]	Nanjing PT	Light GB and CNN	Recent TS	Stacked vector	–	CNN with 1D time series is better than LSTM having multi-D

(continued)

Table 1 (continued)

References	Study Area	Model	Temporal	Spatial	External	Remarks
[29]	KSRTC Mysore	LSTM	Yearly TS	Stacked vector	–	Model had better accuracy than the baseline models
[30]	Kochi Metro	LSTM	Daily TS	Stacked vector with station encoding	–	Station-encoded stacked vector increased accuracy
[31]	Beijing bus transit	XGBOOST	TS	POI	–	Model with POI data had better accuracy

volume of several days or weeks, respectively. Different studies considering different temporal influences are summarized in Table 1.

Spatial Attributes. The distribution of demand along the PT corridor is dependent on the spatial characteristics of that corridor. There will be a possibility of correlation between different lines of a network and different stations of a line. Hence, the inclusion of spatial characteristics into the prediction model has seen an increase in prediction accuracy [21, 25].

In the case of PT flow prediction, the most general way of incorporating spatial characteristics is by stacking the passenger values of a particular stop or corridor in a single vector similar to sequential representation in temporal attributes. For example, [30] did statistical encoding of a station by merging the statistical attributes of 3 neighbouring stations into an input variable space. For example, [30] encoded the time-series passenger count of 3 neighbouring stations into an input variable space considering that there is a correlation between adjacent stations.

Another way of incorporating spatial characteristics is to divide the study area into a grid-like structure representing it as two-dimensional Euclidean data. This representation is done on the basis that the outflow from nearby zones as well as from distant zones affects the inflow of a particular zone [21].

The stacked vector and grid-like representations work in traffic flow predictions where adjacent zones are interconnected, but this theory does not hold good for PT passenger flow, because spatial features at PT stations don't have Euclidean information, owing to distinct regional functions and POI around the station [23, 25, 32]. Hence to embed spatial features into the model, different studies have found different ways. Studies that considered spatial characteristics in developing prediction are summarized in Table 1.

2.3 External Factors

Passenger demand can be influenced by external factors like weather and events. There is a need to compare passenger flows under distinct circumstances to analyse the effects of these external factors. For example, if rain is observed on a specific day, the passenger flow of that day is compared with the same day of the previous week [22, 23, 33]. Studies considered external factors are summarised in Table 1.

3 Conclusion

In this paper, the literature was critically reviewed to compare how statistical and AI methods differ from each other when it is used for PT passenger demand forecasting. The necessity of the inclusion of temporal and spatial attributes in the model and different ways of representing them was discussed. The influence of external factors was also analysed.

With the availability of numerous techniques for forecasting, the question that arises first is which method to choose as each method has its own pros and cons. Statistical methods consider the spatial and temporal attributes independently and the relationship between them is not considered. However, AI models learn all the hidden patterns in the input data and give the best result. As seen in [19], when spatial data was used, ANN-LSTM gave better results when compared to the regression model. Also, when the nonlinearity and complexity of the data increase the statistical models result in overfitting. Hence AI models have a better ability to understand the data and predict with precision. But in the case of AI models, it is not possible to analyse the influence of different variables on the model because AI models are referred to as "black box" [34] and also the inclusion of more data into the model will increase its complexity, and to manage it high effort and time is required along with skills and expertise. Hence the selection of the model must depend on how the researcher needs to approach the problem. If the objective of the researcher is to do an explanatory analysis, i.e. to check how different variables affect the model, then statistical methods are best suited because AI models have less explanatory power. If the data is complex and the goal is to develop a model of high precision, then AI models can be chosen.

The influence of spatial and temporal attributes is very high and it cannot be neglected while developing a prediction model. Data representation is also one of the important factors that needs to be focused on because as seen in Table 1 different representations have different effects on the model. Regarding temporal attributes when periodicity was considered model has shown better accuracy when compared to the recent temporal sequences because people follow the routine in their travelling behaviour, hence daily cycle and the weekly trend have an influence on the prediction accuracy, but to consider periodicity sufficient length of data is necessary. In the case of spatial attributes, stacked vector and grid representations are most commonly used but the inclusion of Euclidean information is not sufficient as the spatial characteristics between the corridors and within the corridors are different. Hence there is a need to consider non-Euclidean information like regional functions, POI, distance, connectivity, etc. Hence while selecting input data representation various factors need to be taken into consideration like availability of different data types, size of the data, domain knowledge, type of model, etc.

In the future, there is a scope to further extend these works. Firstly, there is a need to integrate different models relating to different input characteristics so that accuracy increases [10] and advanced deep architectures like Graph Neural Network (GNN) [35], Graph Convolutional Neural Network (GCN) [25, 36] can be developed to incorporate non-Euclidean information. Second, as observed from the literature different kinds of data representations were used, so there is a need to compare different input data representations and prioritize the factors that need to be considered while selecting these representations. Also, developing new data representations based on domain knowledge, for example, developing an adjacency matrix for non-Euclidean information like regional functions, POI, distance, connectivity, etc. Third, as seen in [28], different load profiles have a different impact on the model, hence there is a need to develop a generalized model which can fit all the data sets, but developing such a model is very challenging. Further, more factors influencing travel demand like demographic attributes, events, resident distribution, population density, etc., can also be considered for model development.

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Assessment of Satisfaction Level for Bus Transit Systems in Bhopal

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Abstract Most people are now highly dependent on private motorized travel because of the low satisfaction level of public transit modes. This leads to poor public transit share in most of the cities in India. This study is intended to identify the influencing parameters; that affect the Satisfaction level of Bhopal Bus Rapid Transit System (BRTS) and other Bus Transport services, and also develop a model for assessing the Satisfaction Level of these transit systems in Bhopal, India. For conducting this study an On-board survey was done for these public transport modes. Based on the literature, Delphi survey and opinion survey, some parameters have been identified that influence the Satisfaction level of these public transport modes. Further by correlation matrix, the five most influencing (critical) parameters were considered amongst them for evaluating the Satisfaction Level. The adopted methodology in this study can help decision-makers to improve public transport services especially various BRTS systems in India so that transit ridership can be improved in BRTS mode.

Keywords Satisfaction level · Public transport system · Demand assessment · User satisfaction · Service quality · Bhopal

1 Introduction

Public transportation plays a key role to fulfill the transport needs, reduce traffic volume and solve the problems of traffic congestion in urban areas [3]. Public transport has been identified as a sustainable solution for all major transport problems all over the world [11]. An efficient public transport system provides opportunity, access, choice, and freedom; all of which contribute to an improved quality of life [17]. Public

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transport is an important service that people patronize to fulfill their travel needs all over the world and the most important issue for this service is the user's satisfaction [10]. Transit service quality has a critical impact on ridership. User satisfaction is a common tool to evaluate transit service quality from the perspective of riders and is also an important key to the success of any public transport system in developing countries [9]. Poor satisfaction with public transport is a major cause of low ridership in Indian cities [17]. Satisfaction is defined as the customer's perception (in terms of comfort and convenience)towards facilities provided by public transport. It is a judgment of a service feature, or service itself, which provided a pleasurable level of consumption-related fulfillment, including levels of under or over-fulfillment [7].

Due to enhancement in social status and paying-capacity, most of the people are willing to buy a car and other personalized modes for their travel requirements. Therefore, these commuters compare the comfort and convenience of BRTS and other public transport modes with car and other private modes. Compared with the users who don't own a private vehicle, car owners prefer higher expectations for comfort and economy; therefore, improving the quality of services of these two aspects is a feasible way to attract more private car owners to use public transport [4]. This imposes more pressure on service providers and concerned agencies to provide better comfort and convenience in existing Bus transit systems in Indian cities. However, for maintaining higher transit ridership, the public transport system must have a high Satisfaction Level for the users.

Public transportation is undoubtedly considered an important solution for traffic congestion, air pollution and traffic fatalities. As per the Ministry of Urban Transport (MoUD) report, the modal share of public transportation is declining day by day in Indian cities particularly in category-IV cities [31]. As per the Indian Road Congress (IRC), category-IV cities are those cities having a population between 2 to 4 million. Inefficiency and poor Satisfaction levels of the public transport system are the major causes of decreasing transit ridership in this category of cities, especially in India [24]. To increase the satisfaction level of PT modes, the satisfaction of service qualities has to be improved as users' satisfaction with the PT system depends on the service quality [5]. An important characteristic of Indian urbanization has been the growth of the informal sector as an integral part of the urban system. They are very strong captive users of the public transportation system as they do not have other alternatives.

2 Significance and Objective of the Study

The rapid growth of the urban population in Indian cities requires an enormous need for efficient public transport services to provide access to high volumes of passengers through dense and congested urban areas [23]. Bus rapid transit System may be a good solution for this high demand. Most of the BRT systems are underperforming owing to dissatisfaction from the road users. In developing nations like India, most of the transit systems run at lower capacity and face predominant ridership issues. The Working Group of Urban Transport (2012) has suggested a desirable modal share of public transport will be 60% of all motorized trips to reduce energy needs and address all transport issues and crises. The poor satisfaction level of these bus transport systems is one of the foremost causes of it. Both scenarios represent the inefficiency and ineffectiveness of these systems. Hence, there is a need for an efficient public transport system that is capable of carrying a significant amount of ridership during peak hours and improving overall transportation capacity by releasing the burden of excess demand on dense and congested road networks [2]. It can only be possible when the satisfaction with the public transport system is increased, especially in Indian cities.

An efficient public transportation system is generally dependent on the level of satisfaction provided by the mode to the user. In order to attract more passengers, public transport must have a high satisfaction level for steady users and also to attract new users. Poor satisfaction level of high-capacity public transport system (Metro, LRTS, and BRTS) does not surely generate high demand in most of the Indian cities [5]. Based on the studies it has revealed that for designing an efficient public transit system, agencies must ensure their performance and level of satisfaction [6]. For improving the satisfaction of the user, the service provider has to improve the service quality of the transit system. For identifying the service quality and facilities which might be improved to increase user satisfaction, the public transport service provider must quantify the existing and desired value of User Satisfaction, which is associated with the satisfaction of service quality of a particular transit system.

This study is an attempt to evaluate the satisfaction level of Bus transit systems pertaining to the service quality so that the service provider can judge which service needs to be improved for increasing the satisfaction in BRTS in Bhopal. The study is also focused on identifying those factors that influence the satisfaction level of the user of Bus transit in Indian cities. It gives the opportunity to all researchers and service providers to adopt this methodology for assessing and enhancing the Satisfaction level of transit users so that more people can be attracted towards public transport, therefore, the transit ridership can be increased.

3 Public Transport in Bhopal

Bhopal is the capital of the Indian state of Madhya Pradesh and the administrative headquarters of Bhopal District and Division. Bhopal is also known as the lake city of India as there are many natural and artificial lakes and also one of the greenest cities in India. Bhopal city was planned on a ring radial pattern with a hierarchical road network. Although this form and pattern do not support public transportation networks effectively especially high-capacity public transport, i.e. BRTS, LRTS, etc., there is a scope for an efficient public transport system for making the city sustainable. Increasing urbanization, urban sprawl, and haphazard growth of the city; present many constraints for proposing the ideal transportation system for catering to the continuously increasing demands for the public transport system for the city

[17]. Despite this, the rapid growth of the urban population in the city has generated an enormous need for high-capacity public transport services to carry high volumes of passengers.

Bhopal City already has a well-developed public transport system with the Bus Rapid Transit System (BRTS) operational at a few identified corridors maintained by *My Bus* (Fig. 1). The Bhopal BRTS is a combination of partially closed & open systems. The majority of BRTS network has a dedicated lane but where the road width is not available, it runs in a mixed lane. In *My Bus* system, the buses running in exclusive lanes are called BRT buses and the buses that are running in mixed lane are called standard buses. In both the system the transit modes are the same. Hence, the public transport services in Bhopal currently consist of BRTS, Standard bus services, Mini buses and Magic-Van. In this study, we have considered both BRT Buses and standard buses for assessing the satisfaction level as poor ridership in these buses is a major challenge for the service provider.

There are mainly three modes of public transport system available in Bhopal city namely, standard bus services (also known as BRTS), Mini-bus services and Van services (Fig. 2).

Bhopal BRT system (including standard buses) was funded by the central government of India under the Jawaharlal Nehru National Urban Renewal Mission scheme. Bhopal BRTS is operated by Bhopal City Link Limited (BCLL) and has an average capacity of 45 persons per bus. This low-floor bus service is available on sixteen routes only with 251 buses with an estimated ridership of about 160,000 passengers per day (Fig. 2). Mini-bus stands second in the hierarchy system, with an average

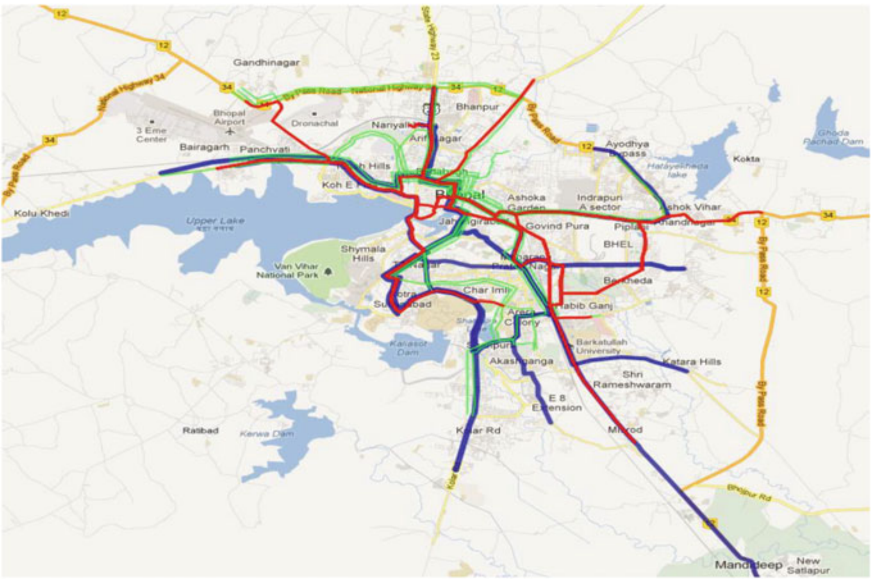

Fig. 1 Road network and bus transport coverage in Bhopal [4]. *Source* CMP

Fig. 2 Types of public transport modes in Bhopal

mode capacity of 25 persons per vehicle, and provides services on various routes based on demand. Mini-buses are run by private operators with assigned routes by the Regional Transport Office (RTO), Bhopal. For many BRTS routes, mini-buses are good feeder alternatives. Van services (also called Magic), with an average capacity of eight persons, are an important public transport system for catering to smaller routes and shorter trips. Magic services are also run by private operators and act as a feeder service for BRTS and other standard buses in Bhopal. They have 43 routes in total with 488 Magic with an estimated ridership of 100,000 passengers per day (RTO and Bhopal Municipal Corporation, 2018). A total of 78 routes of all public transport are available in Bhopal. Figure 3 presents the share of passenger trips in different modes of transport in Bhopal.

The percentage of trips covered by public transit mode is very low in the city, i.e. 49%. Figure 3 presents the current trend of ridership (projected for 2018) in Bhopal. According to IRC, for category-IV cities (population between 2.0 million to 4.0 million) like Bhopal, the desirable public transit ridership must be 60% to 70% for an efficient transportation system for any city [17]. It shows that public transport systems are unable to satisfy people's need for Bhopal. IPT and non-motorized modes have shared about 6% and 8% respectively (Fig. 4). There is an urgent need for a modal shift from private vehicles to public transit to achieve desired modal split as given by IRC (i.e. 60–70%) for smooth maneuver of traffic on urban roads. BRTS Bhopal can be a good solution to increase the desired ridership. It is only possible when BRT mode provides good quality services for achieving a high Satisfaction Level. For improving the service quality of the BRT system, the service provider requires a user satisfaction assessment of these transit systems. Further for enhancing the satisfaction level of these systems, a relative assessment of the services is needed.

4 Literature Review

Barabino and Deiana [4] highlighted the characteristic of transport services, it was clarified that the need for a new perspective on public transport management in urban areas with the help of two methods, namely the modified SERVQUAL method and

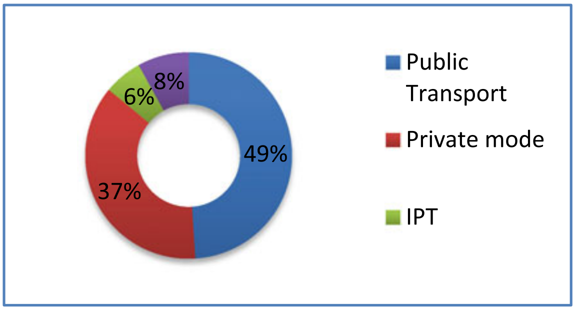

Fig. 3 Projected Ridership (2018) of public transport. *Source* CMP

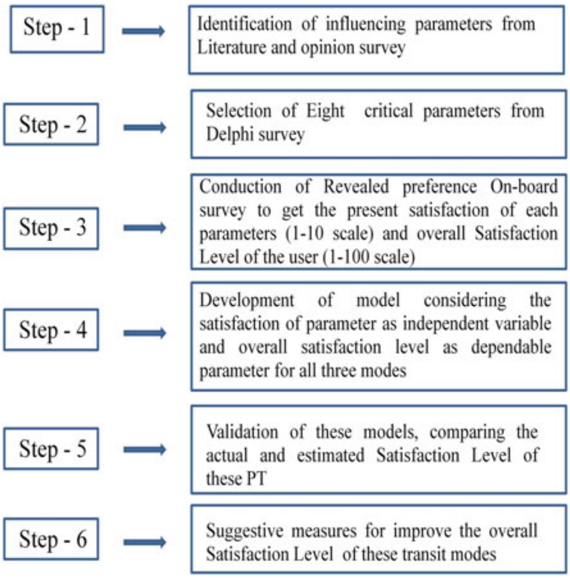

Fig. 4 Conceptual approach of proposed methodology

Multiple Linear Regression (MLR) model. The study came across the factors related to 'on-stop' and 'on-board', affecting the usage of public transportation facilities. Along with this, onboard factors like bus interior cleanliness, temperature, noise, seats comfort affect travelers' willingness to use these transit modes. Punctuality in service and the attitude of transport operator's employees towards bus users were also influential factors.

Ladhari [22] discussed four distinctive features of services i.e. intangibility, heterogeneity, perishability, and inseparability which have been recognized as significant in developing a construct of service quality. The study has shown that good service quality holds the existing customers and also attracts new users. In this study, he focused on the development of reliable instruments for measuring service quality, i.e. "SERVQUAL" scale, which was developed by Parasuraman et al. [23] and is commonly used to measure it.

Too and Earl [30] use a SERVQUAL framework to measure public transport services within a master-planned community in Australia. The survey results find a wide gap between community expectations of public transport services and the actual service quality provided. This study provides a glimpse of the travel behaviours, Public transport needs and assessment of public transport service quality of four main groups of commuters within Varsity Lakes – education, business, general community and visitors. The study attempted to measure the service quality of public transport (bus and train) within Varsity Lakes, in Australia, with the key dimensions of service quality such as tangibles, reliability, responsiveness and assurance. For most commuters, frequency, connectivity and integration between the different modes of transport are important.

Passengers always prefer a well-coordinated, safe and reliable environment in public transport [22]. To fulfill the expectations of passengers, a strategy is required to analyze the performance and improve the service quality of public transport [25]. The performance of the public transport system is mostly analyzed in terms of user's satisfaction which is actually the perception of the PT users towards the services delivered. So passenger's point of view reflects the passenger's perception of the service quality [14]. Level of Satisfaction can also represent effective ways to measure the service quality or the performance of a public transit [7].

Services refer to the elements needed to make the journey easy and more pleasant, like ease to pay fares, traveler's information, journey time and bus stop facilities, etc. The perception of passengers about the services and facilities provided by public transit agencies is expressed with the help of experimental surveys known as "**Customer Satisfaction Surveys**" [14]. These Satisfaction surveys combine the science of measurement of importance and satisfaction of selected parameters. These parameters are primarily the services provided by public transit modes like comfort, safety, reliability, etc. Numerous recent noteworthy studies discuss various parameters to find out customer satisfaction (Table 1).

Kaparias et al. [20] said that safety is still a key issue in transport planning, as many people are involved in road accidents every day. Another important parameter is customer-service which refers to the elements that are needed to make the journey easier and more pleasant like the easiness of purchasing tickets, paying fare, ticket integration, bus stop facilities, traveler's information, usage of ITS (Intelligent Transport System), etc.

Le-Klahn et al. [17] said that public transport demands responses are most sensitive to travel time, travel cost, accessibility, comfort, and convenience for Indian cities. To attract more ridership, travel time should be reduced and the enhancement of accessibility is required especially in Indian cities.

Juan [8] identified that comfort and flexibility are perceived as important factors by the majority of private vehicle users,those are willing to use public transport. Comfort is an important aspect of enhancing ridership of public transit. Le-Klahn et al. [17] emphasized two types of comforts, i.e. inside-comfort and outside-comfort. Inside comfort implies availability of seats; cleanliness and sitting comfort (proper spacing between seats) inside the bus whereas outside comfort means the designing of waiting areas, parking areas, informative services, cleanliness, etc. at bus stops [16].

Table 1 Overview of former literature on satisfaction of PT system

Authors	Parameters selected
Racca and Ratledge [26]	**Travel time, fare**, **income**, **captive user**, parking availability and costs, **age, accessibility, frequency**, trip distance, service
Eboli and Mazulla [5]	Bus stop availability, route characteristics, **frequency, reliability**, bus stop furniture, bus overcrowding, cleanliness, cost, information, promotion, **safety on board**, personal security, personnel, complaints, environmental protection
Tiwari [5]	**Accessibility**, mobility and socio-economic well-being
Kaparias and Bell [7]	**Travel time**, punctuality price, information, cleanliness, staff behavior, seat availability, bus stop security, **safe from accident**, information in bus stop, **frequency**
Budino [20]	Mobility, **Reliability**, Operational Efficiency, **Safety, Accessibility**, Total service area, No. of Motor vehicles and electric vehicles
Jaiswal and Sharma et al. [5]	**Comfort level**, **frequency**, headway, feeder service and **fare, travel time** and more convenience with proper **accessibility**
Shah [3]	Affordability and **accessibility**, mobility, operational efficiency, environmental and resource conservation
Chee and Fernandez [5]	**Age, sex**, income, regular access to private vehicle**, comfort**, availability of parking facility, essentiality of flexibility
Ibrahim et al. [5]	**Fare, travel time,** waiting facilities**, accessibility,** routes suitability**, trip schedule,** waiting time, drivers attitude**, safety, comfort,** passengers' discipline, electronic ticketing system
Putra et al. [4]	**Accessibility,** integrated, capacity, regular, fast and quick, easy, **reliability, comfort, fare,** orderly**, safety,** low pollution, efficient
Le-Klähn et al. [5]	Punctuality**, reliability,** network connection, **frequency,** convenience**, accessibility, safety on board,** ease-of-use**, information, cleanliness** and **space on vehicle, comfort, fare, overall satisfaction in general**
Christopher et al. [5]	**Accessibility**, service **reliability**, security and **comfort**
Jiancheng et al. [20]	Timeliness, **safety**, convenience, **comfort, reliability** and economy
Cyril et al. [11]	Inadequate **frequency**, increased **travel time**, poor **service quality**, and overcrowding

Le-Klahn et al. [12] identified four service dimensions i.e. traveling comfort, service quality, accessibility, and additional features for the understanding of tourist satisfaction with public transport. It is found that commuters are most satisfied with system punctuality, reliability, network connection and service frequency. Reliability of service is defined as 'the ability of the transit system to adhere to a schedule or maintain regular headways and a consistent travel time' [7]. Unreliable service results in longer travel time and longer waiting time for passengers that lead to a loss of passenger ridership,while improvement in reliability can attract more passengers in public transits.

Juan et al. [10] state that the Service quality of any public transport system is an important factor in influencing traveler's behavior, as it encourages public transport users towards selecting transport modes. Reference [25] concluded that accessibility, integration of different modes, capacity, on time and comfort are the main priority expectations of public transport users. Reference [7] said that frequency, price, punctuality and travel time are the crucial factors that are responsible for bringing a higher level of satisfaction.

Frequency can be defined as the number of runs of the service per unit time, i.e. per hour or per day [5]. Fare also matters a lot for the users in mode choice. If the public transport system is inefficient, then it will provide less service than desirable which results in hike in the fare. In other words, if the system is inefficient then users have to pay more than required [6]. Past studies also show that fare is the deciding factor for mode choice in most of the Indian cities. Fare includes the cost of a one-way trip, discounted fares facility and parking charges.

In the following table, we have discussed various noteworthy studies on the Satisfaction level of transit systems and parameters which are dependent on it.

In most of these studies, mentioned in the literature review, opinion surveys were conducted to assess the Satisfaction level and discuss the causes of the inefficiency of PT systems. Cluster technique, correlation matrix and SERVQUAL method have been used for identifying the influencing parameters and also the gap analysis. Limited studies were an attempt to convert survey data into a model for assessing the Satisfaction Level of PT mode. Also, most of these studies have been conducted outside India where they have discussed the issue of the Satisfaction level. Few studies have been conducted for Bhopal but they were limited to identify the causes for the reluctance of using public transport only and did not even discuss the issue of Satisfaction level and any model to assess it.

This research addresses this limitation by conducting a detailed study and developing a model for assessing and estimating the Satisfaction level of BRT mode with the use of correlation and regression processes. Since disaggregated data are required for conducting such kinds of studies, a revealed preference (RP) survey was conducted for this mode and a relationship has been established for the Satisfaction level of BRT modes and their influencing factors (parameters). This study made an attempt to find out individual Satisfaction level, comparison of satisfaction levels for this public transit mode, the reason for low Satisfaction Level and the determination of critical parameters which affect the Satisfaction level of public transport modes.

5 Selection of Parameters

The parameters for satisfaction of any public transport in developing countries may vary from developed countries. The parameters considered for developed countries are different and have varying impacts on travel pattern, travel behavior and travel characteristics as compared to the Indian scenario. Most of the studies are done in developed countries where their priorities may only be travel time and comfort therefore other aspects like travel cost, convenience and accessibility find lesser prominence. Adopting similar criteria as for developed countries may not give a realistic picture of PT systems. Hence, there was a need for an extensive study for selecting parameters for assessing the Satisfaction of Public transit in Indian cities.

There are a number of parameters used in various studies; some of them are referred to in this study as presented in Table 1. A separate opinion survey was conducted to identify the parameters that influence the Satisfaction of the BRT system in Bhopal. Based on the former literature and opinion survey, twenty four parameters were identified that influence the user's satisfaction and transit ridership in the context of Bhopal. A nationwide Delphi survey (with 76 experts in the field of Traffic Planning and Engineering) was conducted by the experts in the said domain to shortlist the relevant parameters. This led to selecting eight impacting parameters out of the twenty-four parameters. Based on the Delphi survey, the shortlisted influencing parameters were accessibility, comfort, reliability, travel time, fare, safety, customer services, and frequency. For the ease of developing the model, we have selected only five critical parameters out of eight influencing parameters, based on the correlation matrix, in modal development for this mode.

6 Defining Parameters

Accessibility attribute represents characteristics of the route of the bus line in terms of path, area coverage, the number of bus stops, the distance between bus stops, and stops location. In other words, accessibility means the distance of the bus stop from the origin and destination of the trip (access and egress distance). Frequency attributes represent the number of buses for a route and time difference between two buses. Amongst service attributes, frequency has the most distinctive aspect in mode choice. Reliability of service is defined as the ability of the transit system to adhere to a schedule or maintain regular headways and consistent travel time. Reliability is mostly related to schedule adherence. Lack of control due to the uncertainty of the vehicle's arrival makes the service unreliable [7]. Comfort means availability of seat, cleanliness, seating arrangement, the spacing between seats and air-conditioning. Safety includes two parameters: one is road accidents and another is security against crime, especially for women. Safety is a psychological parameter and an important factor for the Satisfaction level of any public transit system. Fare is the cost of travel and also an important factor for mode choice in many Indian cities as previous studies show that fare is the deciding factor for mode choice, especially in the Indian context.

Travel time indicates the total journey time from the origin to the destination including access and egress time. Longer travel time in public transport modes can lead to a loss of working trip ridership. The mode choice between public transport alternatives or a car would somewhat depend on the relative time between public transit and the personalized vehicle [4]. Improvements in travel time in the public transport system can attract more passengers. Travel time also depends upon routes and traffic characteristics of that particular area. Customer service attributes include all facilities necessary to make the journey easier, faster and convenient.

7 Methodology and Data Collection

The study is conducted threefold; first to identify the critical parameters that affect the user's satisfaction, second to develop a model to assess and estimate the Satisfaction Level considering these parameters, third to provide suggestive measures for improving the user's satisfaction so that the ridership of existing BRT Systems can be increased in Bhopal. The entire methodology can be divided in six steps and from each step we are achieving a specific objective. The conceptual approach of the proposed methodology is given in Fig. 4.

In the first step of the study, we have identified all influencing parameters (a total of twenty-four) from various literature and also by conducting an opinion survey on the major bus stops. In the second step of the study, we have shortlisted a total of eight influencing parameters by the Delphi method, among the identified parameters from opinion surveys and various kinds of literature, particularly for BRTS Bhopal. In the third stage, for evaluating the Satisfaction Level of existing BRT modes, we required the information of present satisfaction of all eight parameters (on 1–10 scale) as well as the overall Satisfaction of BRT mode on a 1–100 scale from all surveyed users. Questionnaires are the most common tools to investigate a similar aim [7]. To get this information and considering that the user had a discrete choice for selection of public transport mode, an on-board revealed-preference survey on different routes of BRTS was conducted. This revealed preference survey was mainly designed to examine the travel characteristics of the users, satisfaction of respective parameters and the overall Satisfaction Level. These on-board surveys were conducted inside the buses and at the bus stops (as per convenience) along the major routes. These surveys were carried out during morning and evening peak times and on the busiest routes for BRT and non-BRT buses in Bhopal. In this survey, users were asked to provide present satisfaction of each respective parameter on a scale of **1 to 10** and the overall Satisfaction level on a scale of **1–100** for BRT mode. These surveys were conducted mode on major routes and a total of 1230 samples were filled, out of which 1189 were found satisfactory for further analysis. The following steps were followed to conduct the entire study.

In the fourth step, a model was developed for evaluating the Satisfaction level of this mode. To be consistent with the general form of models, as depicted in the various literature, multiple linear regression models were widely used in similar studies. Since the data is distributed in a linear manner, hence to develop a multiple linear regression modal is more suitable in this context. Finally, a multiple linear regression model is developed to evaluate the Satisfaction level for BRT modes by considering only five parameters (having high correlation) amongst the identified eight parameters. To avoid the complexity of modal we have considered only five critical parameters based on the correlation matrix. Statistical Package for Social Science (SPSS) was used for data input, analysis and model development. To summarize and rearrange the data, several interrelated procedures were performed during the data analysis stage. The general form of relationship between Satisfaction Level (Y), as a dependent variable, and satisfaction of each parameter as independent variables

(S) is shown below. Only five independent variables were taken, for ease of model development, which has a maximum correlation with satisfaction level (Y). Here S_1, S_2, ….S_n is the satisfaction of corresponding parameters.

$$Y = C_1 * S_1 + C_2 * S_2 + \ldots + C_5 * S_5 + U \quad (1)$$

Y = Overall Satisfaction (1–100 scale).

C_1 = Coefficient of parameter 1.

S_1 = Satisfaction of parameter 1(1–10 scale, max five parameters).

U = Intercept.

In the fifth step, we validated this model, as we compared the actual and estimated Satisfaction Level of this transit mode from the samples which are not used for the model development process (approximately 5%) and found the average difference in the observed and calculated value of the Satisfaction Level.

In the sixth step, we have given the suggestive measures for improving the Satisfaction Level of Bhopal BRTS.

7.1 Demographic Analysis of Survey

After conducting the opinion survey (for identification of influencing parameters) a revealed-preference survey was conducted at bus stops and inside the buses (On-board) to collect the data from the public transport users. The samples were collected at different times of the day during weekdays and also at all major routes to obtain reliable information. There is no specific framework to compute the exact sample size for any survey. However, the sample size selection will be according to the size of the study population (EMBARQ India). The survey adopted the stratified random sampling survey method. Many studies in India have considered a sample size of less than 1% for the opinion surveys and even up to 0.1%. Due to the limitation of time and resources we have taken only 0.2% sample of total transit demand.

A total of 330 samples were collected and out of these 309 samples were found satisfactory for further analysis. The respondents consisted of 69.5% males and 30.5% females (as per the actual split of PT users). According to the bus operators and other agencies, male users have a high contribution in Public transit ridership as compared to female users in Bhopal. Similarly, the age groups of respondents were divided into three categories. The first category of age group is of respondents less than 24 years (45% samples), the second category of age group is between 25 and 60 years (32%), and the third category of age group is people more than 60 years (23%). Out of these samples, 42% of respondents were students, 17.2% of respondents were working employees in different private and government firms, and

37.14% of respondents were for recreational, shopping and other social activities. Most of the users (71.8%) were captive users as they did not have any alternative mode option.

8 Development of a Model for Satisfaction Level

Revealed preference opinion surveys were conducted to get the present satisfaction of these parameters (1–10 scale) and overall Satisfaction level (1–100 scale) of existing Bus Transit mode. Initially, eight parameters were identified and examined. After the correlation matrix; five critical parameters (having a maximum correlation with the overall Satisfaction Level) were selected based on the set criteria for the mode (Table 2). In other words, five parameters were selected (having maximum influence) from the identified parameters. These critical parameters are used to develop a model to assess the Satisfaction Level of existing BRT mode in Bhopal with the help of SPSS software.

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Model for BRTS (Including Standard Buses)

The following five parameters were selected for Bus transit. These parameters have a high correlation with the Satisfaction level for Standard Buses (Table 2).

1. Comfort (C)
2. Reliability (R)
3. Fare (F)
4. Safety (S)
5. Customer service (CS)

$$Y_{\text{Standard Bus}} = 1.15 * C + 1.54 * R + 2.52 * F + 1.75 * S + 0.90 * CS + 15.8$$

Using the Multi Linear Regression analysis, the following model was obtained for Standards Bus Services (BRTS).

Table 2 Correlation between satisfaction-level and identified parameters (Mode wise)

Mode	Identified Parameters							
	A	*C*	*R*	*F*	*TT*	*S*	*CS*	*FRE*
BRTS	0.576798	**0.745639**	**0.706185**	**0.732988**	0.682409	**0.743763**	**0.74099**	0.494243

Where A is satisfaction of Accessibility, C is satisfaction of Comfort; R is satisfaction of Reliability, F is satisfaction of Fare, TT is satisfaction of Travel Time, S is satisfaction of Safety, CS is satisfaction of Customer Services and FRE is satisfaction of Frequency

Table 3 Summary of observed and calculated Satisfaction level

Public transport mode	Difference % $(Y_{Observed} - Y_{Calculated}) / Y_{Observed}$	Number of samples	% of Data	Average difference
BRTS/standard bus	0–5%	235	58%	**5.30%**
	5–10%	116	28.6%	
	10–15%	40	9.8%	
	15% and above	15	3.5%	

In the above model C, R, F, S, and CS represent the satisfaction of selected parameters for standard buses. The correlation coefficient R^2, for the equation, was 0.86. The t-test and significance level statistics indicated that the variables had good significance.

9 Validation of Model

Based on the above model we can calculate the existing Satisfaction Level of BRTS/ Standard Buses from the collected samples. This Satisfaction Level was compared with the observed (surveyed) Satisfaction Level of the mode during the On-board survey (Table 3).To find the average difference in the observed and calculated value of the Satisfaction Level, the following formula was used.

$$\text{Difference }\% = \frac{Y_{Observed} - Y_{Calculated}}{Y_{Observed}}$$

From Table 3, it is observed that only 3.5% of samples of Standard buses have more than 15%variation from the observed value of Satisfaction Level.

10 Existing Satisfaction Level of Bhopal BRTS

To identify the existing Satisfaction Level for Bhopal BRTS, the average value of satisfaction for all parameters, given by the users, is calculated and considered as an input for the proposed model (Table 4).

From the above values (Table 4), we have calculated the overall Satisfaction Level of BRT mode in Bhopal using the proposed model. The calculated value for the present Satisfaction Level for Standard Bus/BRT is 68%. The commuters have also responded to their willingness in surveys to accept this public transport mode as a primary mode if they have a Satisfaction Level of more than 80%, which means the

Table 4 Average value of satisfaction of each parameter on a scale of 0–10

Parameters	Average satisfaction of parameters for Buses
Accessibility	7.4
Comfort	7.5
Reliability	7
Fare	5.2
Travel time	6.6
Safety	7.5
Customer service	7.2
Frequency	4.9

desired Satisfaction Level should be more than 80% for BRTS in Bhopal. Further, the low value of the Satisfaction level of this mode; clearly indicates that there is a need for immediate improvements in existing BRTS in Bhopal. Existing Satisfaction levels of this PT mode is not acceptable for attracting more transit ridership. The demographic and socio-economic analysis of these surveys shows that 71.28% of users are captive users as they have no alternative choices. The success of any transit system depends on the ability to attract more non-captive users. For attracting these non-captive users, the Satisfaction level of any mode must be increased. Further, it can be understood from Table 4 that BRTS and Standard buses are performing poorly in fare and frequency aspects. This poor satisfaction of selected parameters is one of the causes of the low performance of public transit in Bhopal. Based on this study, planners and service providers can focus on improving these particular services in Bus transit in Bhopal. Therefore, special attention should be given to these parameters, to increase the service quality and overall Satisfaction Level of this public transport mode.

Results of the study show that comfort, reliability, fare, safety, and customer-service are the important parameters for BRTS and Standard bus. Existing BRTS service is plagued by higher cost, low frequency, high waiting time and safety issues. To counter these issues there is a need of lowering the fare, increasing frequency and also providing safety for commuters. The Safety of women is also an important factor and vastly required to improve it in these buses. Finally, if we improve the Satisfaction Level of public transport modes, we can also attract those people, who are using the private mode, in public transport so that congestion can be reduced on urban roads.

11 Conclusion and Recommendations

An increase in the population of the city is generating high travel demands for public transport systems. Bhopal city has lower ridership i.e. only 49% as compared to IRC's recommendation of a minimum 70% ridership for an efficient transport scenario for a city like Bhopal. The basic reason for this poor transit share is low user Satisfaction level. Therefore, there is a need to enhance the satisfaction level for any public transport systems for minimizing the usage of private vehicles. In this study, a model was developed to evaluate the Satisfaction Level of Bus Transit systems in Bhopal and also provide suggestive measures to improve the service quality of this system.

A regression model was developed to assess the Satisfaction Level for this public transit mode in Bhopal. For the preparation of this model, various influencing parameters were identified that influence the user's satisfaction from the literature review and Delphi survey for the public transport modes. Finally, only five critical parameters were selected with the help of the correlation matrix and then linear regression was performed by considering the overall Satisfaction Level as a dependent variable and satisfaction of critical parameters as the independent variable. From this model, we have calculated the Satisfaction Level of BRT Bhopal (68%). These results show the inefficiency of the existing Public Transport System in Bhopal, as the desired Satisfaction Level should be a minimum of 80% (as per opinion survey). To validate our model we have calculated the existing Satisfaction Level of this mode and compared it with the observed (surveyed) Satisfaction Level of the same mode during the On-board survey.

In the context of Satisfaction Level, the BRT system has poor performance in fare and frequency and also, costlier than other PT services. Concerned authorities should investigate and revise the fare policy to attract more transit riders. Demographic data shows that educational and working trips are the maximum in numbers. By providing more facilities we can boost non-captive users to use this public transit mode. BRTS and Standard buses have limited routes and also have poor accessibility from surrounding areas. This study is conducted with limited parameters, data and PT mode. For similar future studies, it is recommended to use other parameters and more detailed information for assessing the Satisfaction Level of any public transit system so that the study can have scope beyond Bhopal. A similar methodology can be adopted to evaluate the Satisfaction Level of public transport systems in other cities.

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Strategies for Improving Travel Time Performance of Multimodal Transport System

P. K. Agarwal, R. Tanwar, and A. Jain

Abstract Multi-Modal Transportation System (MMTS) helps in improving the quality and robustness of roads, transport infrastructure networks, greatly impacts on economic growth, reduce income inequalities and alleviate poverty. The different transportation modes incurred in the strategies include the combination of Walk/Private mode/Public Mode to Bus to Walk/2W/Private mode/Public Mode. This study aims to identify some strategies for improving the travel time performance of multi-modal transportation system. The study is carried out in the Bhopal city, state capital of Madhya Pradesh, India. This study presents a basic methodology framework to identify strategies consisting of three major steps. This includes the identification of key performance indicators, development of the Multimodal Travel Time Index and identification of strategies for improving the travel time performance of the Multi-modal Transportation System. The major strategies identified in this study include reducing the percentage of walk trips to reach the stop, reducing the no. of stops/km length on a route, and increasing the frequency of feeder services to the users. It is expected that the study will help the transit agencies to improve the passenger satisfaction on the level of services by investigating the travel time in different stages for their multi-modal transportation trip. Thus, the strategies identified in this study will also be useful for improving the travel time performance of multimodal transportation systems in a developing country.

Keywords Multi-modal transportation system · Strategies · Travel time

1 Introduction

Multimodal Transportation System (MMTS) has been playing a vital role in shaping countries, influencing the location of social and economic activity, size of cities, style by facilitating trade, permitting access to people and resources, and enabling greater economies of scale worldwide. Multimodal Transportation System may be defined

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as the connectivity of more than one mode of transportation to a line haul, as shown in Fig. 1.

MMTS helps in improving the quality and robustness of roads, transport infrastructure networks, greatly impact economic growth, reduces income inequalities and alleviates poverty. Presently, different public transport systems like Bus Rapid Transit (BRT) System, Light Rail Transit (LRT) System, Mass Rapid Transit (MRT) System and various others like non-motorized transport, many types of intermediate transport systems are being maintained and operated. Despite huge investments in the development of public transport systems, all cities experience the ever-growing problems of congestion, accidents, air and noise pollution and also the economic/financial loss [1]. These days, people in large cities have started using their private vehicles (two-wheelers and cars) and in small cities different forms of intermediate public transport are used. The public transport infrastructure in these cities is not suitable for the present needs of urban transport which is why it experiences many challenges in planning, maintaining and operating services in India. Accordingly, user satisfaction plays a vital role in evaluating the performance of public transport systems based on travel time, travel cost, safety, comfort, reliability, accessibility, etc. [6].

An effective integration of transportation systems is necessary for a city in developing countries. Thus, improving travel time performance made a significant contribution to developing a multimodal transport system. Hence, there is an urgent need to identify strategies for the improvement of the travel time performance of multimodal transportation systems. Hence simple, logical and scientific strategies are required for improving the travel time performance of the multimodal transportation system from a user point of view to select the most appropriate transport system for a given city size. The objectives of the study are identified as follows:

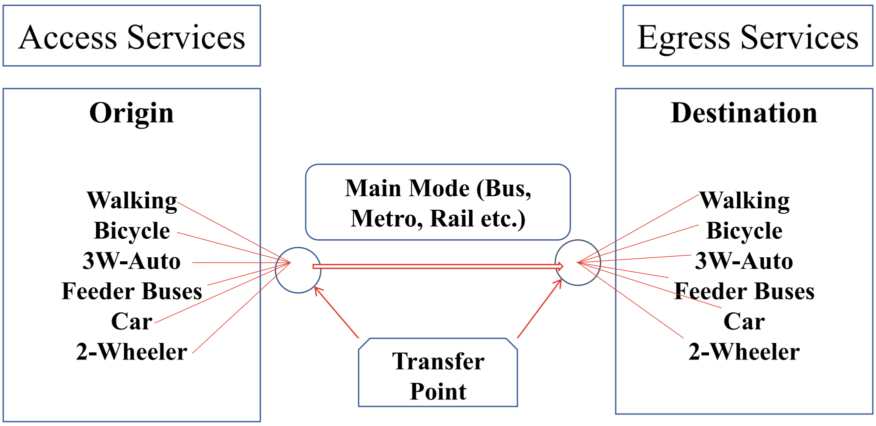

Fig.1 Multi-modal transportation system. *Source* Phani kumar et al.

- To develop the travel time indexes for analyzing the travel time (access travel time index, in-vehicle travel time index, egress travel time index).
- To identify some strategies for improving the travel time performance which includes the strategy for access travel time, in-vehicle travel time and egress travel time.

2 Literature Review

The development of feasible strategies largely depends upon performance indicators. However, many performance indicators are presented in the literature so that identification of appropriate indicators for improving the travel time performance of multimodal transport systems is a challenging task. Further, most of the studies may not be sufficient for improving the travel time performance of multimodal transport systems from a city point of view due to insignificant data. The majority of respondents utilize non-motorized modes of transportation (NMTs), however, the proportions of motorized modes of transportation (MTs) for egress are larger than those for access, including human haulers and buses. According to the findings, rail passengers are more likely than bus passengers to choose MTs for escape [10]. Some of the major findings of the literature review are summarized as follows [1, 2, 6, 9]:

A critical review of the literature [3–9] indicated that limited studies are available on improving the travel time performance improvement of Multimodal transportation system in the Indian context. The performance improvement from a user perspective is an ongoing and complex process.

Further, most of the research studies developed qualitative indices or limited studies are proposed quantitative indices for performance improvement of travel time [2]. However, these indicators are often relatively much more complicated due to the absence of database or the process of data collection is more time-consuming, difficult, and expensive due to availability in abundance.

Furthermore, most of the researchers simply aggregate the performance indicators to estimate the overall performance of multimodal transportation systems from a user perspective. However, significant differences exist between these multiple performance indicators. Hence, there is a need to determine the relative importance (weight) of various indicators and to combine them to determine the weighted overall performance of a multimodal transport system.

Thus, it is necessary to identify strategies based on appropriate performance indicators and provide valuable information based on which important operating decisions can be taken for improving the travel time performance in Indian cities. Hence, there is an urgent need to identify strategies for the improvement of the travel time performance of multimodal transportation systems for developing countries.

3 Strategies for Improvement of Travel Time Performance

The main objective of this study is to identify some strategies for the improvement of the travel time performance of multi-modal transport systems based on travel time parameters. The methodology to identify strategies consists of three major steps. This includes the identification of travel time performance parameters, development of the Multimodal Travel Time Index and identification of strategies for improving the travel time performance of the Multimodal Transportation System.

Multimodal Travel Time is the summation of Access Travel Time, In-Vehicle Travel Time and Egress Travel Time and strategies are identified to reduce the travel time of each. Therefore, there is an urgent need to identify some strategies for improving the travel time performance of multimodal transportation systems. Various travel time parameters are selected in this study for Access Travel Time, Egress Travel Time and In-Vehicle Travel Time. Parameters selected for access travel time are Walking distance from the starting location to the bus stop or public transport stop and waiting time at stop. Parameters selected for egress travel time are the time required to walk from the bus stop or car to the target or destination address. Parameters selected for In-Vehicle travel time are the time required inside the vehicle including transfer time. The different transportation modes incurred in the strategies include the combination of Walk/Private mode/Public Mode to Bus to Walk/2W/Private mode/Public Mode. This study will focus not only on the specific trips but on the trip network of the city.

3.1 Strategies to Improve Access Travel Time Performance

Table 1 represents the details of strategies identified for access travel time performance of multimodal transportation system.

Table1 Details of strategies identified for improving Access travel time

Sr. No	Strategies	Justification
1.	Reduction of walking trips to reach transit stops of BRTS or Metro station	Greater walking trip proportions increase the access travel time which adversely affects the total travel time. As the percentage of walking trips increases, there is an increase in the access time, i.e. Commuters take more time to reach the stop. Hence reduction in the percentage of walking trips to reach the stop may be considered as a strategy to reduce access travel time to improve travel time performance
2.	Quick park and ride facility (having an own vehicle)	Transfer Time plays the most significant role in determining the respondent's preference for improvement of performance of travel time

Table 2 Details of strategies identified for improving In-vehicle travel time

Sr. No	Strategies	Justifications
1.	No. of bus stops/km length on a route	In-Vehicle Travel Time is decreasing with the decrease in the no. of stops/km length of the route or we can say that as the no. of stops/km are decreasing there will be a decrease in the waiting time/stopping time of the vehicle at the stop which in turn reduce the in-vehicle travel time of the journey may be considered as a strategy to reduce in-vehicle travel time to improve travel time performance
2	The number of traffic signals	Minimizing the Number of Traffic Signals will increase the schedule reliability of transit service and escape unnecessary delays
3.	Optimising stoppage time at bus stop	The Total Bus Stop Time (TBST) at stops is the time difference between the time the bus pulls into the bus stop and when it returns to the main traffic stream. By considering variables for a dense urban area like methods of payment, time of the day, crowding level, the time lost serving a stop, and the location of the bus stops, improvements could be made in the optimization of TBST models
4.	Provision of a dedicated corridor	The key to successful BRTS is a dedicated corridor. The dedicated corridor provides optimum speed, and safe and scheduled trips which are essential for performance

Strategies to Improve In-Vehicle Travel Time Performance

Table 2 represents the details of strategies identified for in-vehicle travel time performance of multimodal transportation system.

Strategies to Improve Egress Travel Time Performance

Table 3 represents the details of strategies identified for the egress travel time performance of multimodal transportation systems.

4 Analysis and Results

The strategies for improving travel time performance of multimodal transportation systems in Indian Cities are identified using data of different transport modes in the state capital of Madhya Pradesh, Bhopal through an on-board survey and with the help of Chalo App used for the BCLL (Bhopal City Link Limited) bus service in the city Bhopal. The data have been collected for different parameters which affect the travel time like travel time, access travel time, in-vehicle travel time, proportion of walking trip, egress travel time, average total travel time for a city, no. of bus stops/km length on a route, frequency of feeder services on a route of a city, etc. Various indices are determined and strategies are identified to improve Access Travel Time, to

Table 3 Details of strategies identified for improving egress travel time

Sr. No	Strategies	Justifications
1.	Strategies for minimizing transfer time via inter-connectivity improvement	A well-planned paratransit can prove a means of interconnectivity and enhance and strengthen the PTS network
2.	Availability of Shared Vehicle Trips for Quick Service	A Shared Vehicle based trip has the potential to provide door-to-door transport service with a high level of service and require less than 10% of today's private cars and parking places
3.	Add frequency of feeder services to Bus stops	Increase in the frequency of feeder services reduces Egress Travel Time. Hence, the strategy identified to improve Egress Travel Time is an increase in the frequency of feeder services to the users may be considered as a strategy to reduce egress travel time to improve travel time performance

improve Egress Travel Time and to improve In-Vehicle Travel Time. These Strategies are presented in the following sub-section. The section also presents the analysis for only one strategy. Three multimodal travel time indices namely average access travel time index, average In-Vehicle travel time index and average egress travel time index are developed in this study.

Average Access Travel Time Index (ATIa)

Access to a place in the city generally in the form of walk, public and by any private mode of transport from the origin to the first transit stop/bus stop/station. Access time represents the time taken from the origin to the bus stop or public transport stop/station by a commuter or passenger.

Average Access travel time index can be determined using the Eq. (1).

$$\mathrm{ATIa} = \sum_{\forall V} \mathrm{ATa} \times \mathrm{pa}/\mathrm{TT} \tag{1}$$

where

ATIa = Average access travel time index by the ath mode of transport.

ATa = Average access travel time per km taken by a particular mode, i.e. walking, public transport, private transport (2W).

pa = Proportion of access mode of ath type (pa = walk/2w/Public etc.).

TT= Average total travel time per km from origin to destination.

Average In-Vehicle Travel Time Index (AIVTIv)

In-Vehicle Travel time represents the time taken by the main mode to reach another transit stop (time taken in Bus/Metro or train).

Average In-vehicle travel time index can be determined using Eq. (2).

$$\text{AIVTIv} = \sum_{\forall V} \text{AIVT} \times \text{pv}/\text{TT} \tag{2}$$

where

AIVTIv = Average In-Vehicle travel time index by vth mode in-vehicle transport.

AIVT = Average In-Vehicle travel time per km by a particular mode.

pv = Proportion of In-Vehicle transport mode of vth type (pv = BRTS/Mini Bus, etc.).

TT = Average total travel time per km from origin to destination.

Average Egress Travel Time Index (AETIe)

Egress time represents the time taken to reach the final destination (transit stop to destination) from the bus stop or public transport stop/station by a commuter or passenger.

The average Egress travel time index can be determined using Eq. (3).

$$\text{AETIe} = \sum_{\forall e} \text{AET} \times \text{pe}/\text{TT} \tag{3}$$

where

AETIe = Average egress travel time index by eth mode of transport.

AET = Average egress time per km by a particular mode.

pe = Proportion of egress mode of eth type (pe = walk/Public/Private etc.).

TT = Average total travel time per km from origin to destination.

The identification of the strategies is based on the indices developed above for the Access Travel Time, In-Vehicle Travel Time and Egress Travel Time and are presented in the following section.

4.1 *Analysis to Identify Strategies to improve Access Travel Time*

The average access travel time index is determined for various proportions of walking trips. Table 4 presents the details of the percentage of walk and average access travel time index from origin.

Figure 2 presents the variation of the Proportion of walk (percentage) on the Average Access Travel Time Index.

Table 4 and Fig. 2 indicate that greater walk proportions increase the access travel time which adversely affects the time. As the percentage of walk trips increases, there is an increase in the access time, i.e. Commuters take more time to reach the stop. Figure 2 indicates that a reduction in the percentage of walk trips reduces the access travel time index. Hence, a reduction in the percentage of walk trips to reach a stop may be considered as a strategy to reduce access travel time to improve travel time performance.

Table 4 Details of percentage of walk and average access travel time index

Sr. No	Proportion of walking trip (percentage)	Average access travel time index
1	40	0.23
2	50	0.24
3	60	0.26
4	70	0.28
5	80	0.3
6	100	0.33

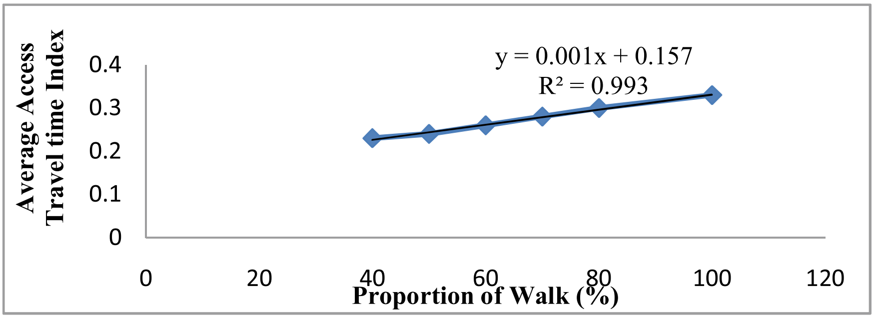

Fig. 2 Variation of proportion of walk (percentage) with the average access travel time index

Table 5 Details of average in-vehicle travel time index and no. of stops/km length on a route

Sr. no	Average in-vehicle travel time index	No. of bus stops/km length on a route
1	0.3	2
2	0.36	3
3	0.38	5
4	0.45	7
5	0.47	8

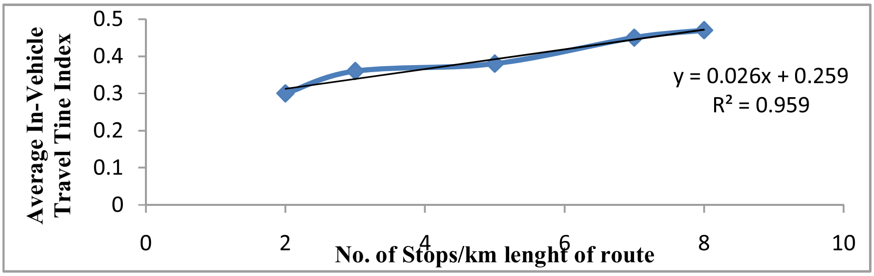

Fig. 3 Effect of no. of stops/km on a route on Average In-Vehicle Travel Time Index

4.1.1 Analysis to Identify strategies for Improving In-Vehicle Travel Time

The average in-vehicle travel time index is determined for various proportions of no. of bus stops/km length on a route. Table 5 presents the details of the average in-vehicle travel time index and no. of stops/km length on a route.

Figure 3 presents the variation of the Average In-Vehicle Travel Time Index is improving with a decrease in the no. of stops on the route.

Table 5 and Fig. 3 indicate that if there are more no. of stops on a route, there will be increases in Average In-Vehicle Travel Time in the city.

In-Vehicle Travel Time decreases with the decrease in the no. of stops/km length of the route or we can say that as the no. of stops/km decreases there will be a decrease in the waiting time/stopping time of the vehicle at the stop which in turn reduce the in-vehicle travel time of the journey may be considered as a strategy to reduce in-vehicle travel time to improve travel time performance.

4.2 Analysis to Identify Strategies for Improving Egress Travel Time

The average egress travel time Index is determined for various proportions of the frequency of feeder services (in min). Table 6 presents details of the frequency of feeder services (in min) and average egress travel time index from transit stop to destination.

Figure 4 presents the variation in the frequency of feeder services and its effect on the Average Egress Travel Time index.

Table 6 and Fig. 4 indicate that if the frequency of the transit services increases then there will be a decrease in the egress travel time index of the city.

Figure 4 indicates that an increase in the frequency of feeder services reduces Egress Travel Time. Hence, the strategy identified to improve Egress Travel Time is an increase in the frequency of feeder services to the users may be considered as a strategy to reduce egress travel time to improve travel time performance.

It is to be noted that in the above analysis, all the parameters for the performance of travel time used are weighed equally but their impact on the overall performance varies. Hence, it is recommended that a weight-based ranking system be developed through an expert opinion survey by applying multi-criteria decision-making techniques such as the Analytic Hierarchy Process. Further, to address and neutralize the intangibility and vagueness in the scaling process of benchmarking the fuzzy logic approach should also be used with a triangular membership function (TMF).

Table 6 Details of frequency of feeder services (In min) and average egress travel time index

Sr. No	Frequency of feeder services (In min)	Average egress travel time index
1	35	0.39
2	30	0.33
3	25	0.25
4	18	0.16
5	10	0.12

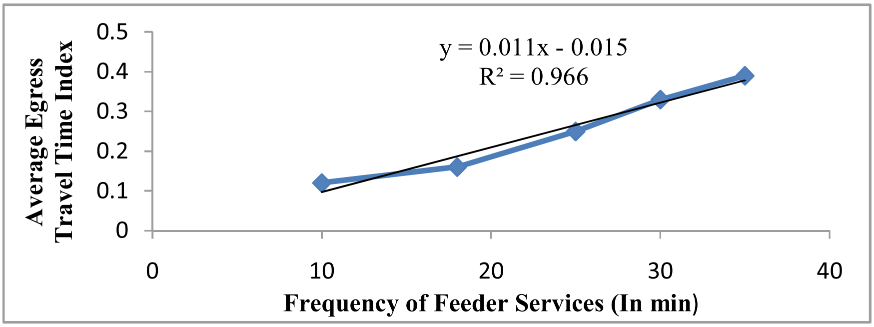

Fig. 4 Effect of frequency of feeder services on average egress travel time index

5 Conclusions

This section presents the conclusions made from this study.

- Travel time is one of the core elements that heavily affect the passenger's opinions and the quality of multimodal transport systems for sustainable mobility in urban areas. Multimodal Travel Time is the summation of Access Travel Time, In-Vehicle Travel Time and Egress Travel Time and strategies are identified to reduce the travel time of each. There is an urgent need to identify some strategies for improving the travel time performance of multimodal transportation systems.
- This study presents a methodology framework to identify strategies. Three travel time indices for analyzing the travel time (i.e. access travel time index, in-vehicle travel time index, egress travel time index) are also developed in this study.

This study has identified some strategies for improving travel time performance which includes the strategy for access travel time, In-Vehicle travel time and Egress travel time. The strategy for access travel time is to reduce the percentage of walk trips to reach the stop. The strategy for in-vehicle travel time is to reduce the no. of stops/km length of the route. The strategy for egress travel time is to increase the frequency of feeder services to the users.

Limitations and Scope

Traffic demand (the number of people entering a network), traffic supply characteristics (the amount of capacity on the infrastructure), and external factors are included by the Indo-HCM travel time reliability indicator (snowfall, tsunami, rainfall, etc.) [11]. This study does not consider for aspect of ridership and travel demand. Therefore it is recommended that the aspect of ridership and travel demand may also be incorporated for future work.

It is expected that the study will help the transit agencies to improve the passenger satisfaction with the level of service by investigating the travel time in different stages in a multimodal transportation trip network for an urban city. These strategies will be useful in enhancing the performance of multimodal transportation systems in terms of travel time performance.

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Traffic Management, Operations, and Safety

A Review on Pedestrian Level of Service for Sidewalks

N. C. Vijay, S. Rokade, and G. R. Bivina

Abstract Walking is a basic and environmentally friendly mode of transportation. For encourage walking, the walking facilities need to be designed and maintained properly. The performance of pedestrian facilities needs to be measured to enhance the popularity of walking as a mode which is termed as Pedestrian Level of Service (PLOS). Several studies have been conducted so far regarding the PLOS for sidewalks. The present study attempts to give a comprehensive overview of the studies conducted on PLOS for sidewalks using Preferred Reporting Items for Systematic Reviews and Metra-Analyses (PRISMA) guidelines. The overview was carried out focusing on the literature published till 2022 all over the world. A total of 25 papers obtained from Google Scholar were reviewed. The main objective of the study was to analyze the available literature on the basis of study locations, methods of data collection, parameters considered and analysis techniques used. Overall findings across the literature reveals that a large number of studies were conducted in India. It was found that the dimensions of the sidewalk were the factor that was most frequently considered in most of the studies. Regression analysis was the most commonly used model to analyze the PLOS. The key insights and limitations of the study are discussed. In the end, conclusions and scope of further research are also presented.

Keywords Walking · Sidewalks · Level of service

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1 Introduction

Walking is considered as the green and environment friendly mode of transportation. It reduces traffic fumes and air pollution and is found to be a zero-emission mode of travel [6]. It promotes equity by presenting a cost-effective mobility option for weaker sections of society [7, 8]. In the domain of health, walking is found to reduce the risk of diseases [5, 10].

Walking can be improved not only by providing quality infrastructure but also by providing a safe and comfortable environment [26]. Safety is a major concern for pedestrians [7–10]. As per the study conducted by World Health Organization [5], vulnerable road users (VRUs) are not considered for the design of walking infrastructure facilities which is the main reason for their deaths on roads while walking. Therefore, the design and assessment of infrastructure facilities is of utmost importance.

Sidewalks are pedestrian facilities that helps to avoid conflict between vehicles and pedestrians. A sidewalk which is safe and pedestrian-friendly promotes walking and also helps in developing a lifestyle that is healthy for pedestrians. It also enables the people to make a transition from private modes of transport to a much more sustainable and eco-friendly mode. It also provides safety to pedestrians by channeling vehicle and pedestrian traffic. But in India, sidewalks are not properly maintained. They are often occupied by vendors, which reduces their effective width. If the sidewalk facilities are not in proper condition, pedestrians tend to use the main carriageway, resulting in conflicts with vehicles. It is reported that out of the total deaths, 37% are pedestrians and out of the total accidents, 35% have happened near sidewalks [5–7]. Therefore, in order to enhance the safety of existing facilities, pedestrian issue needs to be taken into consideration.

Pedestrian Level of Service measures how well the pedestrian network offers good mobility for pedestrians. Pedestrian Level of Service (PLOS) is a standard that often guides the planning and design of pedestrian environments. *Level of Service can be defined* as a quantitative stratification of performance measures that represent quality of service. Pedestrian Level of Service (LOS) has been defined by many studies in different ways. Indo HCM [5] defined PLOS as the measure for assessing the operation condition of the facility in a quantitative manner. Sheikari and Shah [2] defined PLOS as a measure for explaining current facilities, situations, equipment and infrastructure in streets and it also evaluates the quality of service is a measure that is used to determine the quality of service of transportation facilities. PLOS for sidewalks expresses the degree of satisfaction offered by the sidewalk facilities to the pedestrians with respect to safety, comfort and convenience. Most of the PLOS models were developed based on linear speed-density relation [4]. Macroscopic Fundamental Diagram (MFD) was used to measure the level of service on road networks [5]. Fruin [29] introduced the concept of sidewalk evaluation and PLOS based on qualitative methods. The study combined quantitative and qualitative factors such as pedestrian speed-density relationships, personal characteristics of pedestrians, human body dimensions, and behavioral preferences. Later in 1974, Lautso and Murole [30] tried to measure the influence of environmental factors on pedestrian facilities. PLOS concept was then incorporated in the Highway Capacity Manual [5] to quantify sidewalk capacity that recommended including other environmental factors such as comfort, safety, convenience and security. Mori and Tsukaguchi [4] introduced a PLOS measuring pedestrian behavior based on their opinion. Sarkar [32] proposed threshold PLOS levels ranging from A to E, where A denotes the highly pedestrian-friendly streets and E represents incomplete streets that lack basic pedestrian facilities. This study considered factors such as attractiveness, comfort, convenience, safety, security and system coherence. HCM [4] recommends an analysis of sidewalks that focuses on quantitative factors such as the mobility of pedestrians. Safety is adopted as one of the critical factors for the assessment of pedestrian crosswalks. But in the case of pedestrian sidewalks, HCM [10] does not consider any of the safety factors. Pedestrians' perceptions based on the qualitative environmental factors could help in a better understanding of their needs, level of importance, and it will better comprehend the factors that facilitate walking. With this concept, Khisty [33] proposed a methodology considering the same factors adopted by Sarkar [32]. Sisiopiku and Akin [34], Sarkar [35] and Henson [36] also reported pedestrian perceptions and attitudes towards walking facilities.

The present study focuses on presenting an overview of the various literature that are available regarding the PLOS for sidewalks. An overall idea regarding the study locations is presented. The different parameters that are found to have a significant effect on the PLOS of sidewalks are also identified. An extensive overview of the data collection methods and models used in the studies is also given. The conclusions that are identified from the literature are presented. The final section discusses the scope of further research in the area of PLOS for sidewalks.

2 Methodology

A systematic review is a type of review that uses repeatable methods to find, select, and synthesize all available evidence. It answers a clearly formulated research question and explicitly states the methods used to arrive at the answer. It aims to provide a comprehensive, unbiased synthesis of many relevant studies in a single document. While it has many of the characteristics of a literature review, adhering to the general principle of summarizing the knowledge from a body of literature, a systematic review differs in that it attempts to uncover "all" of the evidence relevant to a question and focus on research that reports data rather than concepts or theory. The characteristics of a systematic review are well-defined and internationally accepted. The following are the defining features of a systematic review and its conduct:

- clearly articulated objectives and questions to be addressed
- inclusion and exclusion criteria, stipulated a priori (in the protocol), that determine the eligibility of studies

- a comprehensive search to identify all relevant studies, both published and unpublished
- appraisal of the quality of included studies, assessment of the validity of their results, and reporting of any exclusions based on quality
- analysis of data extracted from the included research
- presentation and synthesis of the findings extracted
- transparent reporting of the methodology and methods used to conduct the review.

Figure 1 shows the flow diagram of the framework of the literature search conducted using PRISMA guidelines. The articles retrieved from the database search were checked for duplication and the duplicate articles were removed. The assessment of the remaining articles was conducted based on their titles and abstracts. The full texts of the remaining articles were analyzed. Finally, the articles were screened for inclusion in the systematic review. Initially100 articles were identified through a search on databases. After the removal of duplicate articles, 55 articles were left. These were then filtered through the title and 36 articles remained. These were then screened based on their abstracts. Finally, 25 research articles were included in the systematic review.

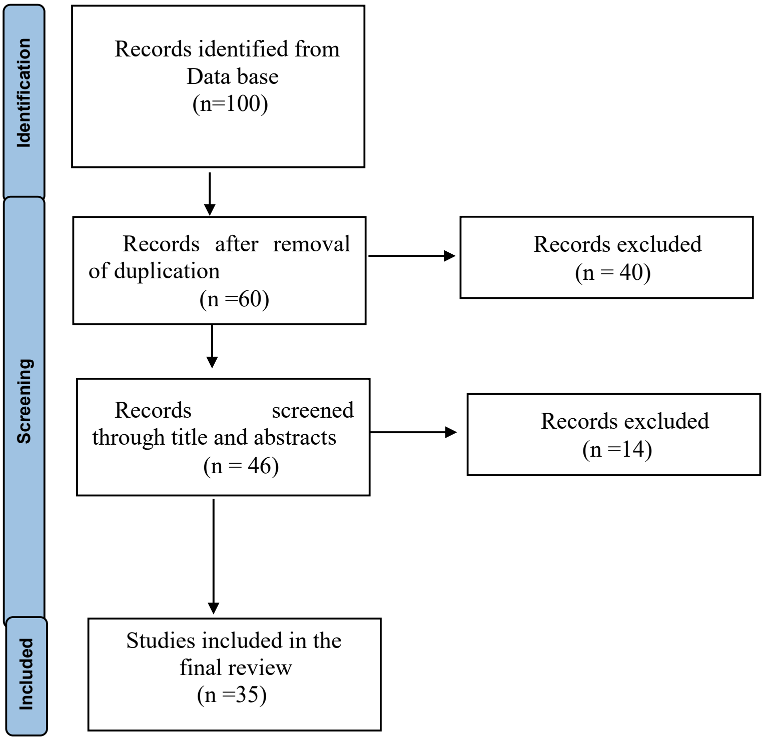

Fig.1 PRISMA flow diagram

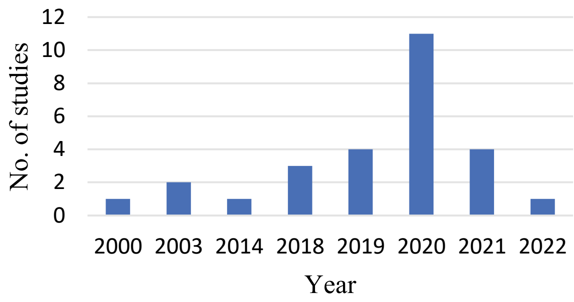

Fig. 2 Number of studies conducted year-wise

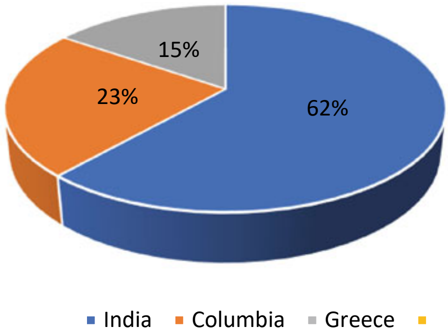

Fig. 3 Country-wise distribution of the selected studies

3 Literature Review

3.1 Based on the Studies Conducted Worldwide

The present review included only the literature published in the English language. Articles from peer-reviewed journals, reports, case studies, general articles, conference papers, book chapters were used for the review and commentaries, opinions, letters, were excluded from the review. Figure 2 represents the number of studies conducted year-wise. It shows that about 50% of the studies were conducted in the year 2020.

The study revealed that many of studies were conducted in India [4, 5, 7, 8, 22], followed by in Columbia [17], followed by in Greece [6, 13]. Figure 3 provides the number of studies conducted in various countries.

3.2 Factors Considered

There were many factors that were considered in analyzing the PLOS for sidewalks in all the studies. These factors can be categorized into quantitative and qualitative.

Among the quantitative parameters, the height and width of the sidewalk was reported as the most significant factor affecting PLOS according to the pedestrians. A study was conducted to analyze the level of service of sidewalk, where width was observed as the most important factor influencing PLOS [8]. The importance of sidewalk height and width in the development of PLOS was confirmed in the studies by Nag and Goswam [7].

Among the qualitative parameters, safety and comfort were found to be the most significant factor in determining PLOS [8], in a study to analyze the service quality of sidewalks found that safety against sexual harassment was the most significant variable affecting the same. The significance of safety in determining PLOS was also evident in the studies conducted by Ujjwal and Bandyopadhyaya [1, 7, 8]. Apart from safety. The presence of street vendors (Ahmed and Islam) and discomfort and externalities [22] were also observed as significant variables in developing PLOS for sidewalks.

3.3 Analysis Techniques Used

The general trend observed was that most of the studies have utilized statistical regression for developing the PLOS [6–10]. Some studies have used advanced soft computing techniques such as cluster analysis [24, 25] and Fuzzy scale [4, 5]. For the classification of LOS into different categories, only very few studies have utilized cluster analysis. Most of the researchers have classified them either manually or based on their experience and judgment which made the results unreliable.

Assessment of the importance of various factors of PLOS of sidewalks as employed Analytic Hierarchical Process (AHP) [6, 7], very few studies have utilized Fuzzy analysis. Table 1 provides a review of the parameters considered and analysis techniques used in the studies.

4 Conclusions

The present study focused on an extensive overview of about 25 papers on the PLOS for sidewalks. The literature were analyzed on the basis of observation sites, factors considered, and analysis techniques used. It was observed that 50% of the studies available were conducted in the year 2020. A significant number of studies were conducted in India. Among the different parameters considered in analyzing the PLOS for sidewalks, the height and width of the sidewalk were considered as most significant one among the quantitative parameters. Among the qualitative parameters, safety and comfort while walking along the sidewalks were considered as most significant. The most common technique used for determining the PLOS for sidewalks in the majority of the studies was found to be regression analysis.

Table 1 Summary of the parameters considered and analysis techniques used in the studies

Sl. No	Author and year	Parameters considered											Analysis techniques
		Height and width of sidewalk	Safety and security	Surface conditions	Presence of street vendors	Obstructions	Cleanliness	Accessibility	Comfort and convenience	Pedestrian flow characteristics	Lighting	Vehicular characteristics	
1	[7]	•											
2	[8]												Step wise regression analysis
3	[5]												Macroscopic flow diagrams
4	[15]												US Code and Proprietary modeling tool
5	[4]												
6	[5]												Fuzzy Likert scale
7	[12]												Regression technique
8	[1]												
9	[22]	•										•	EFA and SEM
10	[8]												Factor analysis and SEM
11	[2]												

(continued)

Table 1 (continued)

Sl. No	Author and year	Parameters considered											Analysis techniques
		Height and width of sidewalk	Safety and security	Surface conditions	Presence of street vendors	Obstructions	Cleanliness	Accessibility	Comfort and convenience	Pedestrian flow characteristics	Lighting	Vehicular characteristics	
12	[17]	•											OLS models
13	[8]	•	•										SEM
14	[7]	•								•			Logit model
15	[9]	•									•	•	AHP
16	[7]	•	•						•				Ordered Probit model
17	[7]		•								•	•	Factor analysis
18	[6]									•			AHP, Ordinal regression model
19	[25]	•							•			•	Clustering
20	[7]												AHP

The present systematic review was found to have certain limitations. The review was confined only to those papers which were published in English language. The review considered only peer-reviewed scientific articles, which would have resulted in publication bias.

5 Scope

The present review sheds light on future research. From the current systematic review, it was observed that most of the studies have utilized either qualitative or quantitative parameters for developing the PLOS. Future studies could focus on using both parameters. There are a limited number of studies available in India. Therefore, more studies can be conducted in the Indian mixed traffic condition. Most of the studies have utilized conventional techniques for analyzing the PLOS for sidewalks. Future research can focus on utilizing soft computing techniques for analyzing PLOS. There is a lack of studies that focus on the factors affecting PLOS for disabled persons, which future research can work on.

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Assessment of Pedestrian Safety at Urban Uncontrolled Intersections Using Surrogate Safety Measures: A Case Study of Bhopal City

Dungar Singh, Pritikana Das, and Vasu Verma

Abstract Pedestrians seem to be the most vulnerable road users, particularly in a heterogeneous traffic environment with inadequate road infrastructure for motor vehicles. However, the rapid growth of traffic would rapidly increase conflicts between vehicles and pedestrians. Due to the uneven pedestrian crossing patterns at uncontrolled intersections, this issue is more prevalent. The conflict-based approach to pedestrian safety analysis is a more reliable proactive method than the previous crash-base method. The aim of the study is to evaluate pedestrian safety by analyzing the conflict of the pedestrian-vehicular interaction at three uncontrolled intersections, using the different temporal proximity measures of Post Encroachment Time (PET), Vehicle Approaching Speed (VAS), and spatial proximity measure Safe Distance (SD). Finally, the K-Means Clustering technique was used to classify the severity level as "safe" or "unsafe" after a study of 1380 separately identified pedestrian-vehicle interactions in two scenarios Vehicle Passing First (VPF) and Pedestrian Passing First (PPF) from three uncontrolled intersections. Four different vehicle categories, namely two-wheeler (TW), auto, car, and heavy vehicle (HV), and the gender of pedestrians (male or female), are considered for analysis. Furthermore, the threshold values for PET, SD, and vehicle approaching speed are defined separately for each category and gender.

Keywords Pedestrian safety · Surrogate safety measures · Severity level · Uncontrolled intersection

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1 Introduction

The safety of pedestrians is a critical component of traffic engineering and a fundamental goal of Transportation engineering. Pedestrians contributed to 17% of all pedestrian fatalities in accidents, while bicycles were responsible for 3% [1]. Pedestrian crossing at uncontrolled intersections in Indian cities is a frequent occurrence. When crossing an urban road with the lack of adequate walking and crossing facilities, pedestrians place themselves in a significant danger of colliding with an approaching vehicle. During pedestrian crossing operations, pedestrians accept gaps and attempt to cross the road based on their skill, experience, and traffic flow characteristics. The intersection is usually specified and planned for automobiles exclusively, but a high prevalence of pedestrian accidents necessitates the implementation of safety measures to safeguard pedestrians from collision. Traditionally road safety analyses were analyzed using crash data, mostly these results are underreported and not produce precise conclusions [2, 3]. However, this is a reactive method that does not represent the causative variables of incidents. Therefore, there is a need to provide alternative safety evaluation approaches. Safety evaluation using surrogate measures is a safer approach for resolving all of the above-described difficulties. The objective of this study is the evaluation of proactive pedestrian safety at unsignalized intersections using SSM parameters such as PET, approaching vehicle speed, and Safe Distance (SD) to evaluate pedestrian-vehicle conflicts. There conflict between pedestrian vehicles were classified based on severity level using K-Mean Clustering Technique.

1.1 Definitions and Terminology

Post-Encroachment Time (PET): PET can be defined as the time interval and times T1 and T2, where T1 is the time at which the road user passing first (vehicle/pedestrian) departs the collision area, that is the rear of the vehicle leaves the conflict area. T2 is the moment when the second road user (vehicle/pedestrian) arrives at the collision area, i.e. when the front end of the vehicle reaches the conflict area in the case of a vehicle. The lower the PET value, the greater the chances of collision will be [4].

Safe Distance (SD): SD is a distance-based proximal safety indicator that can be defined as the distance a vehicle or person travels from the potential conflict location when another conflicting object arrives first. The higher the SD, the safer the pedestrian is, and the lower the SD, the less safe the pedestrian is and the more dangerous or aggressive the motorist or pedestrian is [5].

Vehicle Approaching Speed (VAS): It is the speed of the conflicting vehicle approaching the conflicting grid.

2 Literature Review

Analyzing the safety of pedestrians, several researchers used conflict techniques to evaluate the safety of different traffic facilities. Although the majority of researchers used the temporal proximity SSM indicator, namely PET, the severity level was determined by the speed of the approaching vehicle [1, 6–8]. Although other researchers use the PSM indicator for Mid-Block safety analysis. Another study by Golakiya et al. [5] used a spatial proximity measure of safe distance to evaluate mid-block safety. Furthermore, the aforementioned studies demonstrate that earlier work was done using temporal proximity measures, but Golakiya et al. have suggested taking into account spatial proximity measures [7]. Further, the current study uses temporal and spatial proximity measures to analyze pedestrian safety because temporal proximity measures (PET), speed of approaching vehicles, and Safe Distance are easily observable from video observation. The application of both proximity measures could enhance the reliability of results.

3 Study Methodology

The methodology of a research project outlines the complete work process and plan for achieving the research project's goal. It is a methodical process to achieve the goal of this study. The adopted methodology for the study is shown in Fig. 1. The study aims to classify conflicts depending on their severity level as safe and unsafe by providing threshold PET, safe distance, and vehicle approaching speed values using the k-means clustering technique. For this, three uncontrolled intersections were selected for the study area and videographic data were collected during 4 h of peak and non- peak hours. Image processing tool Kinovea was to extract SSM parameters like PET, SD, and vehicle approaching speed along with various other data like vehicle category and pedestrian demographic composition speed and gender of the pedestrian. The interaction behaviour will be determined by observing which road user is passing the conflict zone first. This incorporates two separate cases that is either the vehicle is passing the conflict zone first (VPF) or the pedestrian is passing the conflict zone first (PPF). Both cases are considered in this study and separate threshold values are concluded for each case. Four vehicle classes were considered, i.e., TW, auto, car, and HV. After that, threshold values of PET, SD, and vehicle approaching speed for severity level classification as safe and unsafe interaction are defined based on k-means clustering techniques in MATLAB.

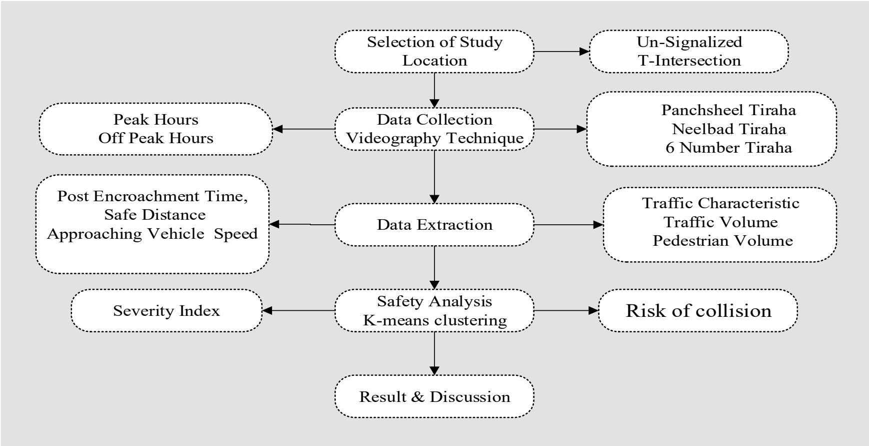

Fig. 1 Methodology flowchart

4 Data Collection and Extraction

Different three-legged urban intersections with uncontrolled intersections were chosen for safety assessments in Bhopal city. All selected intersection geometric detail are demonstrated in Fig. 2 and Table 2. There is no crosswalk marking on the major road, so crossing conflict between pedestrians and vehicles can occur anywhere in the conflict zone. During the crossing movement of the pedestrian at the intersection, pedestrian interacts with different categories of vehicle, in this study pedestrian interaction with two-wheelers (2w), three-wheelers (auto), car, and heavy vehicle (H.V.) are analyzed. Two cases are observed during the crossing movement of pedestrian, Case 1 is the vehicle passes first case in which the vehicle passes the conflict zone while the pedestrian is far away, Case 2 is the pedestrian passes first case in which the pedestrian passes the conflict zone while the vehicle is far away. Both cases are precisely analysed to get the threshold values of SSM parameters for each case respectively. Videographic surveys were done in morning and evening peak hours from the top of a nearby building to capture the conflict area. Data were extracted using image processing based on Kinovea software, which divides a second into 25 frames, the overlayed video was played back on a large-screen monitor. The size of block of the overlaid grid is kept small (0.85 X 0.85) to accurately track pedestrian position. Further, the precise position of a pedestrian and the corresponding location of the vehicle were manually tracked and recorded every 0.04 s over the grid. The four categories of two-wheeler (2W), Auto, Car, and Heavy Vehicles were applied to all the vehicles in the stream. Moreover, collected videos were transferred to a laptop and different parameters such as road geometric information, operational information such as vehicle speed, type of vehicle, gender of pedestrian and SSM indicators PET and safe distance were extracted. Foe interaction between different types of vehicles and pedestrians totally 1380 samples were collected for analysis.

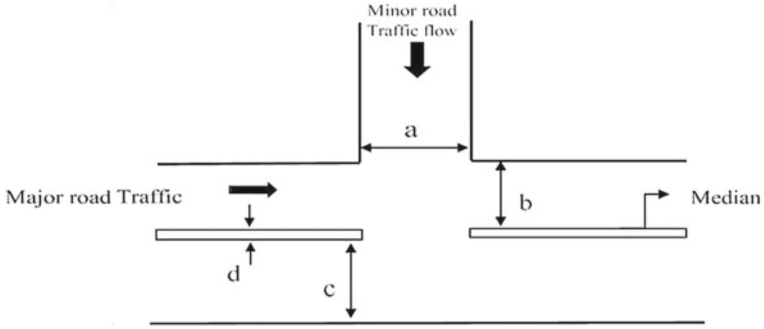

Fig. 2 Representation of intersection geometric details

Table 1 Summary of prior studies on SSM for pedestrian safety assessment

Study	Geometry type	Type of conflict	SSM	Major finding
Kumar et al. [9]	Signalized Intersection	Crossing	PET, TTV, DSTped, TTA, DSTveh	Defined severity level of using K-Means clustering, demographic characteristic has significant influence on pedestrian behaviour
Chaudhari et al. [6]	Midblock, Un-signalized Intersection	Crossing	PET, VCS, YC,	Defined severity level depends on pedestrian, vehicular and traffic characteristic
Kathuria and Vedagiri [7]	Un-signalized Intersection	Crossing	Speed, TTC, PET Gap Time	Defined severity level based on Import Vector Machine (IVM)
Golakiya et al. [5]	Midblock	Crossing	Safe Distance (SD)	Developed safety index threshold based on vehicular speed
Shah and Vedagiri [4]	Un-signalized Intersection	Crossing	PET	Defined severity level using Binary Support Vector Machine (SVM)
Kadali and Vedagiri [10]	Mid-Block	Crossing	PSM	Pedestrian charateristic rolling behaviour, speed change condition significantly influence the probability of conflict
Marisamynathan and Vedagiri [1]	Signalized Intersection	Crossing	PET	Defined severity level of pedestrian safety at crosswalks

Table 2 Geometric details of selected intersections

Geometric Data (m)	Name of Intersection		
	Panchsheel Tiraha	Neelbad Tiraha	6 No. Tiraha
a	15	10.6	24
b	6.9	10	14
c	7	9	12
d	1.2	NA	NA

5 Data Analysis

5.1 *Preliminary Data Analysis*

Using various tools and operations in MS Excel, preliminary data analysis is done to understand the vehicle–pedestrian interaction with four different categories of vehicle (2W, Auto, Car, HV) in two separate cases (VPF, PPF). The following Tables 2 and 3 show the descriptive statistics of the extracted data for safety measure PET, safe distance, and the aggregate Minimum, Maximum and mean of SSM indicators evaluated to assess pedestrian safety. On doing preliminary data analysis for various types of interaction between vehicle and pedestrian, it can be seen that PET and SD values in the case of a female pedestrian are much greater than in the case of a male pedestrian, which reveals that male pedestrians accept a much smaller gap in comparison to female pedestrians while crossing the road. Furthermore, it can be seen that as the size of the vehicle increases, PET and SD values also increase, which reveals that pedestrians accept a much larger gap with large-sized vehicles as compared to small-sized vehicles. PET values in the VPF case depend majorly on pedestrian crossing speed while in the case of PPF, it depends on a vehicle approaching speed..

5.2 *Severity Level Classification*

Clustering based on K-mean is frequently used in data mining, data compression, pattern classification, and pattern recognition because it is directly related to the number of clusters and the location of the problem. In this study, MATLAB is used for k-means clustering. Pedestrian-vehicle interaction with each category of vehicle is analysed separately. Since the dataset is in the high-to-moderate-risk category, two clusters, cluster-1 and cluster-2, were employed. Cluster-1 indicates a high chance of collision and is referred to as an "unsafe condition for pedestrians" whereas Cluster-2 indicates a moderate chance of collision and is referred to as a "safe condition for pedestrians". While the intersection of vehicle and pedestrian highly depends on the category of vehicle, pedestrian interaction as tabulated in Table 4 car has higher PET, SD threshold, than light vehicle Two- Wheeler and Three-Wheeler, because

Table 3 Descriptive statistics of PET

Vehicle Category	Type of Interactio n	Gender	Post Encroachment Time (s)				Mean Vehicle Speed (Kmph)
			Total Sample s	Maximum	Minimum	Mean	
Two-Wheeler	VPF	Male	534	5.72	0.16	1.6	28.39
		Female	223	6.2	0.24	1.99	32.87
	PPF	Male	64	1.4	0.08	0.76	19.09
		Female	17	1.56	0.32	0.81	17.61
Auto	VPF	Male	65	5.72	0.32	1.8	26.75
		Female	50	5.88	0.28	2.38	30.14
	PPF	Male	17	1.28	0.16	0.92	21.29
		Female	13	1.52	0.48	1.06	20.82
Car	VPF	Male	176	7	0.12	1.87	29.01
		Female	63	8.44	0.8	2.45	30.19
	PPF	Male	30	4.2	0.12	1.24	18.57
		Female	15	1.96	0.68	1.26	17.17
Heavy Vehicle	VPF	Male	58	5.76	0.56	2.7	25.44
		Female	36	6.2	0.92	3.54	31.44
	PPF	Male	11	3.04	1.04	1.56	16.27
		Female	8	4.56	0.96	2.52	15.35

increased size of vehicle pedestrians generally take a long time and distance when interacting with the vehicle. Where male pedestrian interaction with Two-wheeler SSMs indicators PET, SD threshold are observed 1.84 s and 1.7 m, respectively, that shows smaller than interaction with car 2.24 s and 2.55 m. However, the pedestrian's gender also plays a significant role. It has been found that male pedestrians have higher PET and SD thresholds than female pedestrians, which clearly indicates that male pedestrians cross the road more aggressively. Similarly suggested by authors [9–11] male pedestrian crosses the road more aggressively,the female cross the road on a comparatively safer side. Pedestrian interaction with Two-wheeler (VPF), observed PET and SD threshold are 1.84 s and 1.7 m, respectively, which is quite greater than that in the case of 2W interaction with a female, which is 2.6 s and 2.55 m, respectively. The interaction of pedestrian vehicle in case of PPF is more dangerous than, VPF, the reason for that in case of PPF, PET mainly depends on vehicle approaching speed, in case of VPF, PET threshold mainly depend on pedestrian crossing speed, it observed that PET threshold for PPF case is 1.2 s, while VPF case 2.68 s. Similarity Safe Distance threshold in the case of PPF is larger than VPF, that shows pedestrians generally avoid passing through the conflict zone first if a vehicle is in a near-miss collision situation. Safe Distance threshold in the PPF case is 9.35 m, and VPF is 2 m. Although, VPF scenario speed of approaching vehicles is higher than PPF case, because the speeds of any category of vehicle, pedestrians give way to vehicles while at lower speeds, pedestrians choose to cross the road. It observed that the vehicle approaching speed threshold value in the VPF case is 28.69 kmph, while it is 22.5 kmph in the PPF case.

Intersection of vehicle and pedestrian highly depend on the category of vehicle, pedestrian interaction with heavy vehicle and car have higher PET, SD threshold, than light vehicle Two-Wheeler and Three Vehicle, because increase size of vehicle pedestrians generally take a long time and distance when interacting with the vehicle. Table 5 demonstrates the high risk of collision in VPF and PPF in below PET threshold 2.28 s and 1.2 s, respectively. The same on another SSM indicator Safe Distance (SD) found a high risk of collision below thresholds 2 m and 9.35 m.

Table 4 Descriptive statistics of safe distance

Vehicle Category	Type of Interaction	Gender	Safe Distance (M)				Mean Vehicle Speed (Kmph)
			Total Samples	Maximum	Minimum	Mean	
Two-Wheeler	VPF	Male	534	5.1	0.85	1.99	28.39
		Female	223	5.1	0.85	2.31	32.87
	PPF	Male	64	12	4	7.44	19.09
		Female	17	14	5	8.99	17.61
Auto	VPF	Male	65	4.52	0.85	2.27	26.75
		Female	50	5.1	1	2.61	30.14
	PPF	Male	17	10.2	6	7.74	21.29
		Female	13	14	5	9.38	20.82
Car	VPF	Male	176	6	0.85	2.49	29.01
		Female	63	8	1	2.7	30.19
	PPF	Male	30	13	5	9.41	18.57
		Female	15	14	6	9.98	17.17
Heavy Vehicle	VPF	Male	58	6	1.7	3.64	25.44
		Female	36	7	1.7	4.33	31.44
	PPF	Male	11	13	7	10.2	16.27
		Female	8	13	9	11.3	15.35

Table 5 Severity level for vehicle–pedestrian with different categories of vehicles

Vehicle Category	Type of Interaction	Gender	Severity Level	Threshold by variables		
				PET	SD	VAS (kmph)
2W	VPF	Male	Safe	> 1.84	> 1.7	< 30
			Unsafe	< 1.84	< 1.7	> 30
		Female	Safe	> 2.6	> 2.55	< 33.75
			Unsafe	< 2.6	< 2.55	> 33.75
	PPF	Male	Safe	> 0.68	> 6	< 19.29
			Unsafe	< 0.68	< 6	> 19.29
		Female	Safe	> 0.84	> 8.5	< 15.88
			Unsafe	< 0.84	< 8.5	> 15.88
Auto	VPF	Male	Safe	> 2	> 2	< 27
			Unsafe	< 2	< 2	> 27
		Female	Safe	> 2.68	> 3	< 32.75
			Unsafe	< 2.68	< 3	> 32.75
	PPF	Male	Safe	> 0.72	> 7	< 16.88
			Unsafe	< 0.72	< 7	> 16.88
		Female	Safe	> 1.04	> 9	< 19.29
			Unsafe	< 1.04	< 9	> 19.29
Car	VPF	Male	Safe	> 2.24	> 2.55	< 30
			Unsafe	< 2.24	< 2.55	> 30
		Female	Safe	> 2.92	> 3.2	< 32.79
			Unsafe	< 2.92	< 3.2	> 32.79
		Male	Safe	> 1.2	> 9.35	< 18
			Unsafe	< 1.2	< 9.35	> 18
		Female	Safe	> 1.48	> 10	< 22.5
			Unsafe	< 1.48	< 10	> 22.5

Table 5 Severity level for interaction of pedestrians with all categories of vehicles

Vehicle Category	Type of Interaction	Severity Level	Threshold by variables		
			PET (sec)	Safe Distance (m)	Vehicle approaching speed (kmph)
All Vehicles	VPF	Safe	> 2.28	> 2	< 28.69
		Unsafe	< 2.28	< 2	> 28.69
	PPF	Safe	> 1.2	> 9.35	< 22.5
		Unsafe	< 1.2	< 9.35	> 22.5

6 Conclusion

This study evaluated the performance of pedestrian safety using the interaction of a pedestrian-vehicle method based on video graphics data. Overall, this research provides different safety surrogate measures to define the severity of the conflict between pedestrians and vehicles in a crosswalk at uncontrolled intersections. Descriptive statistics of estimated conflict indicators provide a general idea of the risk associated with pedestrian crossings. Although cluster analysis was used to categorize these indicators according to severity, the actual risk assessment. The k-means clustering approach is used to define severity levels into four classes. The study identified a high risk of collision in VPF and PPF below PET thresholds 2.28 s and 1.2 s, respectively. The same on another SSM indicator Safe Distance (SD) found a high risk of collision below thresholds 2 m and 9.35 m. Safety of pedestrian during pedestrian vehicle interaction is highly influenced by the speed of approaching vehicles. Collision risk increases while increasing the speed of vehicles. Due to the higher speed of the vehicle, a pedestrian might not have enough time to react and avoid a collision. However, higher thresholds of PET and SD are not considered to be safe interactions. The severity of conflict is jointly attributed to the threshold value of PET & SD and the speed of the approaching vehicle. For instance, if values of PET and SD are higher than the risk of collision threshold, pedestrian-vehicle interaction might not be considered safe if the approaching vehicle speed is higher. The novelty of research is consideration of spatial and temporal SSM indication additionally with a demographic parameter such as gender of pedestrian. The future scope of this study may include analysis of pedestrian age, education affects assessments of pedestrian safety, and extension of this methodology in different traffic facilities such as four-leg intersections, mid-blocks, etc.

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A Review of Safety and Operational Impacts of Various Speed Limits

Abhinav Mishra, A. Mohan Rao, and Darshana Othayoth

Abstract Speed is the uttermost element influencing the incidence and ferocity of road accidents. The mitigation of the frequency of over speeding is seen as an essential goal for mitigating the number and severity of collisions, and the conventional method for doing so is through posted speed zoning or posted speed limit. Various speed control strategies are now being incorporated on roads to curb accidents' frequency and risk. Some of the speed control strategies used worldwide are Uniform Speed Limit (USL), Differential Speed Limit (DSL), Variable Speed Limit (VSL), and Lane-Based Speed Limit (LBSL). In this paper, previous research based on the implementation of DSL on different classes of roads at various road stretches has been summarised, and all the conclusions are considered for an upcoming virgin project titled "Determining safety aspects of differential speed limit on Indian roads".

Keywords Differential speed limit · Uniform speed limit · Microsimulation · Traffic safety

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1 Introduction

Speed limit with a higher compliance rate is the backbone of a safe traffic system. Accidents on the road are unwelcome events that often result in injury or death. According to the Ministry of Road Transport & Highways (MORTH) report, Road Accidents in India [21], the main reason which holds the reason behind most accidents and death on the Indian roads on every class of National Highways is over-speeding. The Nilsson Power Model states that for every 5% rise in average vehicle speed, there are 10% more accidents resulting in injuries and 20% more accidents resulting in fatalities. About 10% fewer accidents result in injuries, and 20% fewer accidents result in deaths for every 5% drop in average vehicle speed [4]. According to Haworth et al. [14], if the speed of the mobile vehicle is above 30 km/h, then the human tolerance for harm from an automobile will be surpassed. According to Mohan et al. [22], road accident is the result of many parameters. Various methodology and techniques are now been implemented to mitigate the frequency of over-speeding. There are lot of studies been carried out to compare various speed control management techniques and their impact on road safety. Those studies include some limitations which can cause irregular relationship between speed and hazard or skewness of any specific result due to inclusion or omission of any specific attribute which or may not affect the road safety explicitly or implicitly. This extensive literature reviewing is carried out to enlist the advantages and disadvantages of various methodologies which enable the future research to improve their methodologies to obtain an non skewed or non-biased conclusions.

2 Speed Limit

Due to the impact of speed on roadway safety, speed limits are one of the traffic concerns in developing countries. The term ‘speed limit’ refers to the maximum speed that can be legally enforced in the interest of safety and mobility on the road, considering environmental, roadway, and roadside factors. Generally, speed limit is of three types i.e., single-speed limit, differential speed limit, and advisory speed limit. The Uniform/Single speed limit (USL) strategy instructs to implementation of an equal maximum speed limit for all classes of vehicles. In contrast, the Differential Speed Limit (DSL) strategy suggests two different maximum speed limits for two different vehicle classes, i.e., passenger class vehicles and goods carrying vehicles. The advisory speed limit is suggested speeds for bends, junctions, or other various areas where operating speed on that stretch is regulated to a lower value than maximum legal speed or implemented speed limit by characteristics of the roadway geometry. The primary parameter in establishing maximum or posted speed limits, which are often applied to all classes of vehicles, are either highway design or geometric constraints.

According to Methods and Practices for Setting Speed limits: An Informational Report, GHSA [12], there are four mainstream techniques for establishing speed limits i.e., engineering approach, expert system approach, optimization, injury minimization, or safe system approach. Factors used in the determination of optimal speed limits are travel time, vehicle operating costs, road collisions, noise pollution due to the movement of traffic, and atmospheric pollution. The maximum speed limit is established in accordance with the collision types that are likely to occur in a safe system approach, the bump or thrust forces that will result from those crashes, and the human body's capacity to withstand those forces. Apart from speed restrictions, there are other facts regarding speed to consider, such as how engineers choose a design speed to guide their design decisions and make plans before creating a new route but when road work is completed, engineers will quantify the prevailing speed by measuring the operating speed.

3 Differential Speed Limit

Controversial findings can be seen in the literature regarding the use of USL or DSL. Regarding the history of implementation of DSL on various kinds of roads, many cities have different speed limits according to the type of vehicle e.g., in Missouri, cars were allowed to travel up to 70 mph, whereas trucks were limited to 60 mph. On rural national routes in Ireland, 15 mph was the differential speed limit posted in 1979 i.e., 55mph for cars and 40 mph for trucks [15].

Many studies and research have been carried out to quantify the effectiveness of DSL in reducing crashes and improving speed variance. Solomon [28] discovered a U-shaped relationship between the amount of speed deviation from the average and the frequency of collisions. Any increment in variance can prompt rise in the number of accidents, particularly accidents including (non-compliant) passenger car and (compliant) heavy vehicles. Increasing reciprocity between cars as a result of increased speed variance might result in an increase in collisions. Duncan et al. [6] discovered that a large speed difference enhances the severity of rear-end collisions between passenger vehicles and heavy vehicles.

4 Previous Studies on Speed Limit and Impact of DSL

Malyshkina and Mannering [20] explored the connection between posted speed limits and the observed severity level of injuries involved in the accident to study the impact of speed restrictions on traffic safety. Their research showed that Interstate speed restrictions had no appreciable effect on the severity of accidents or injuries. Regarding the effect of compliance rate of speed limit on traffic flow and accidents, Bains et al. [3] conducted a thorough analysis of the impact of speed restriction compliance on the capacity of Indian Expressways. According to the findings, roadway capacity increases when the percentage of cars adhering to the legal speed limit rises. In addition, travel durations are reduced somewhat as driver compliance levels rise. Gayah et al. [10] conducted research to observe the impact of posting speed limits below the advisory on mobility and safety. Heavy enforcement inside the lower speed limit zone resulted in a reduction in average speed as well as on 85th percentile speeds approximately by 4 mph and also causes increments in speed limit compliance rate. Speed limits 10 mph lower than engineering guidelines were associated with a statistically significant reduction in the incidence of both total and front-end collisions, but a statistically significant increase in the probability of events with fatal injury. Sugiyanto et al. [29] conducted study to observe the impact of lowering the speed limit on mobility and the environment due to a reduction in the speed limit. Study findings suggested that emissions of a variety of pollutants were found to be reduced at lower speeds and also hydrocarbon emissions were reduced at lower speeds, whilst Carbon Monoxide (CO) and Particulate Matter (PM) had the minimum level of exhalation at medium speeds.

Yao et al. [31] conducted a study to examine the speed limit credibility and compliance on UK roads. The study concluded that the most frequent value of tendered safe speed limit was adopted as the most viable speed limit. Additionally, drivers perceive and want to travel at faster speed higher than the posted speed limit irrespective of whatever the speed limit was established. Kwayu et al. [18] used a multilevel negative binomial regression model to analyse the impact of changing the speed limit on Michigan freeways and found that increasing the speed limit by 15 mph resulted in a 21% increase in overall accidents and 11.9% increment in crashes related to injuries and fatalities which contradict the conclusions from the study carried out by Garber et al. [9] and the Department of Transportation Federal Highway Administration. They carried out a study to examine the safety aspects of the implementation of DSL strategy on rural interstate roads in the US and concluded that irrespective of which speed control strategy was implemented among USL & DSL there will be a growth in crash rate and average speed of the vehicles. Along with the study by Malyshkina and Mannering [20], Ossiander and Cummings [23] also advocated the same findings where they studied impact of freeway speed limit and traffic fatalities in Washington state using crash data and vehicle data on Washington state freeway from 1974 to 1994. They concluded that there was growth in the value of speed variance as well as in the 85th percentile speed on both rural as well as urban roadways from 1980 onwards which remain independent of any changes in speed Huang et al. [16] carried out a study to observe the effect of variation in the speed limit in the maintenance work area of Expressway during morning and night-time. Effects on the capacity of the maintenance work area come out to be different for a different proportion of big vehicles in traffic composition because more oversized vehicles showcase poor performance in terms of acceleration, deceleration, and braking compared to smaller vehicles.

Garber and Gadiraju [8] focused on the impacts of DSL on the traffic speed and rate of accidents. Results concluded that the DSL strategy with not significantly more efficient than the USL strategy in mitigating the risk of exceeding the speed limit. Both strategies showcase the similar results and concluded that traffic phenomenon will fluctuate irrespective of implemented speed control strategy. Korkut et al. [17] conducted a thorough investigation of the impact of lane restrictions for trucks on traffic and crashes, as well as the implementation of the DSL strategy. Crash rates exhibit a strong relationship with variation in mean truck speed, the difference between mean speed of trucks and mean speed of passenger vehicle class, and lane occupancy. Ghods et al. [11] carried out research to observe the influence of DSL on the performance of safety aspects of two-lane highways using microsimulation methodology. The study concluded that in DSL and MSL speed control techniques, the frequency of car-to-car overtakes decreases while the number of car-truck overtakes increases. Habtemichael and Santos [13] carried out a study to provide quantitative evaluations of safety impacts and compliance of VSL on the 12-km study area of Motorway A-5 in Portugal, by simulation methodology. The study concluded that this method provided the safest advantages in heavily dense traffic, then in mildly dense traffic, and at minimum, in an uncrowded traffic state. By analysing vehicle speed characteristics in Indiana, Michigan, and Ohio, Russo et al. [26] conducted a comparative analysis incorporating USL and DSL speed control techniques. It was shown that going from 55 to 60 mph caused the mean speed and 85th percentile speed to increase by 3 to 4 mph. Using the VISSIM traffic simulation software package, Adresi et al. [1] conducted comprehensive research on the safety and traffic performance implications of the LBSL and vehicle-based (containing USL and DSL) strategies for highways. According to the results, the LBSL exhibited a larger speed variance and lane shifting capability than the other strategies. Sadat and Celikoglu [27] undertook research to examine the effects of VSL on the Istanbul motorway, comparing scenarios with the implementation of VSL strategy and in its absence. The study revealed that even at modest compliance levels, the suggested VSL systems contributed in reducing congestion levels. The congestion may be reduced by enforcing the statutory speed restrictions, since higher compliance levels generate better outcomes. Fuel efficiency is improved, and the suggested VSL system shows promise in its potential to be environmentally benign by reducing emission of CO and NO_x as well as the fuel consumptions. Soriguera et al. [7] conducted a study in Barcelona to observe how low-speed limits affect traffic flow on highways. This study concluded that low-speed limits could increase lane-changing behaviour, posing a safety risk on freeways but the limitation of this study might have tilted the results towards this conclusion as all these conclusions are based on high-compliance controlled portions. Lower compliance rates would presumably lower VSL impacts hence this set of conditions needs to be looked at. Liu and Shi [19] conducted a comprehensive study to evaluate the influence of Differentiated per-Lane Speed Limit (DPLSL) on safety aspects of freeway traffic i.e., Frequency of lane changes, variation in traffic speed and hazard rate. They concluded that when traffic congestion is on higher side and space headway is minimal, drivers are more inclined to lane changing phenomenon and Lane changing phenomenon found to be higher in USL setting in comparison to VSL.

Qu et al. [24] carried out a thorough analysis of models of traffic flow and driving behaviour under VSL control, that simulated both individual driving behaviour and associated traffic flow features. Big traffic data was garbled, processed, and investigated to compare traffic data collected before and after the introduction of the VSL technique on driving behaviour analysis. The study concluded that VSL did modify the driver's desired speed. Under VSL management, the proportion of minor headways (less than 1 s) was considerably reduced, implying that some drivers became more cautious showcasing improvement in driving condition and road safety. Costanzo et al. [5] conducted a study to determine the overall portability and conditional advantages of an instinctive VSL strategy for a highway stretch in the city of Naples. The outcomes of the study indicated a small improvement in the average speed of up to + 2.24%, suggesting a reduction in travel time on the considered road network. He also concluded that VSL can provide environmental benefits too as for all 100 carried out simulations, there was decrement of 9.54% reduction in average fuel consumption. In comparison to the non-VSL-based scenario, the VSL-based scenario observed a decrease in both the average delay as well as in the average number of stops, indicating an improvement in the travelling condition on the roadway.

5 Identification of Critical Gaps in the Literature

The general limitations of every research work related to all those studies mentioned in previous sections were the inability to capture real-time data and simulate data in the original condition in which data was collected. Alonso et al. [2] conducted research to observe practices, purpose, and analysis of the efficacy of sanctions for over-speeding. The issue with this study was the inability to capture the real intention behind complying with speeding laws, as the driver was aware that he was breaking the law. Yao et al. [31] carried out a questionnaire technique to conduct a study to examine the acceptability and dependability of speed limit and compliance on UK roads. The response rate in this research was just 10%, and the questionnaire sample size was only 100. The fact that respondents who were not in a hurry took part in this survey might be a drawback. Other limitations were that multiple subcategories of parameters and groups were not considered in the model creation or in the research methodology, and any correlation between multiple variables like age, sex, driving behaviour, and driving experience weren't considered while establishing speed limit.

Also, the metering capabilities of extremely low-speed limit were not investigated in this study. Along with that, analysis was solely focused on static traffic conditions instead of dynamic traffic conditions, but static traffic does not capture full spatial–temporal traffic characteristics. The distribution of headways, particularly the fraction of smaller headways, is essential to the traffic flow's stability. The effect of the VSL on the percentage of cars with a headway of not more than one second was studied by Qu et al. [24]. But focusing solely on the proportion of headways of less than one second will not give a full picture of the safety state of the traffic. Inclusion of other smaller headways scenarios e.g., headways of less than 1.5 s or headways of less than 2 s can be crucial when approaching the flow's maximum capacity. In the study of Korkut et al. [17], DSL had an ambiguous effect on traffic safety since it caused collision rates to rise when trucks went above the truck speed limit but to fall when passenger cars went over the car speed limit. An irregular negative correlation between vehicle speed limit infractions and accident rates was surprising. Since the RTMS devices provided speed for all vehicles combined for each 30 s time interval, the truck and other vehicle's speeds were estimated under diversified-vehicle circumstances.

Extending the limitation of the study, previous research was not able to draw any conclusion on the relationship between the proportion of trucks in total traffic composition and the rate of accident occurrence. In a study by Garber et al. [9] and the Department of Transportation Federal Highway Administration, they were unable to distinguish how the USL/DSL strategies had an impact. The study's analysis of aggregated speed data did not reveal any DSL-related effects. Within the parameters of the investigation, no consistent safety differences between DSL and USL were found. In the finding of Sugiyanto et al. [29], they did not able to find one single speed limit which can minimize CO, NO_x, HC, PM, because for speed limit of 40 kmph the emanation of CO and NO_x were observed to be at its lowest, but for speed limit of 50 kmph particulate matters were observed to be minimal. Russo et al. [26] in addition to looking into the effects of USL versus DSL on trip speed, also looked into the travel speeds of trucks, buses, and passenger cars in jurisdictions with various speed limits. However, according to the research, speed selection could have been influenced by local variables, and those local factors might have been the reason behind the variations in driver speed choices. So, the exact relationship between speed control strategy and crash rate could not be predicted. Liu and Shi [19] conducted research using DPLSL. The results of this study were heavily reliant on simulations, which was one of its limitations. For gathering the speed data and assessing the two hazardous situations, the precise locations of all cars at a given moment were required, and in simulation, traffic movement is predefined and not so erratic in comparison to real traffic conditions. Real traffic flow, on the other hand, is irregular and haphazardous, and hence this was termed as a limitation of this study. Most of previous research on the impact of DSL on safety aspects of traffic and roads used statistical methodologies like the before-and-after approach, which might not reflect the true improvement or might not show the reasoning behind any changes, which can be included as limitations of previous studies.

6 Conclusions

One of the main inspirations for this literature review was to explore the efficacy of DSL on speed and safety aspects of operating conditions on the road. The majority of the previous studies related to this pre-identified strategy were unable to pinpoint which speed control strategy among USL and DSL gives better performance in terms of their impact on road safety. Many drivers will disregard the posted speed limit if it is too low, resulting in common following and acceptance rates. Slower speeds reduce the intensity of collisions; however, higher speed variance inflated the likelihood of collisions. Most of the studies concluded that there will be growth observed in average speed of vehicles and accident rate with time irrespective of whatever speed control strategies have been implemented. A study on the impact of low-speed limits on traffic flows on highways concluded that low-speed limits could increase lane-changing behaviour, posing a safety risk on freeways. Also, other studies on low-speed limits suggested that emissions of various pollutants were found to be reduced at lower speeds and hydrocarbon's emissions were reduced with slower speed, whilst CO and PM have the minimum ejection rate at moderate or at the usual speeds. Stating these findings in more detailed perspective, previous study did not able to find one single speed limit which can mitigate CO, NO_x, HC, PM, as they found out that CO and NO_x were at the lowest emission level. The effects of a 5 kmph reduction in travel speed are greater at lowers speeds, and a reduction of speed limit to 50 kmph brings a substantial decrement in the risk of crashes or collisions and fatalities. Furthermore, few studies concluded that the VSL strategy could cause a reduction in the proportion of smaller headways (less than 1 s), which points towards the positive impact of VSL with respect to safety impact on traffic, and implicitly, it can be said that drivers became more cautious under VSL strategy.

So, summing up the previous study related to DSL and its impact different to of USLs, previous studies didn't able to segregate out any major differences between impact of USL and DSL on safety aspects hence more dwelling in the matter needs to be undertaken. This thorough review of methodologies and pre-established practises involved in managing the speed and enhancement of compliance rate with respect to speed limit will pave a way for understanding the factors and components which implicitly or explicitly does affect the safety of road environment and its users.

Further research needs to be undertaken, incorporating other reliable, realistic factors, and instead of incorporating statistical techniques for formulating a relationship between crash rate and different speed control strategies like USL and DSL One of the most significant flaws in the statistical technique is the constraints imposed on the analysis due to the lack of data hence to counter that a microsimulation study can be proposed which can replicate the real field condition. This extensive literature review will pave the way and can build a solid platform for an upcoming proposed study with a pre-identified methodology (see Fig. 1) and titled as "Determining Safety Aspects of Differential Speed Limit (DSL) on Indian Roads" in CSIR-Central Road Research Institute (CRRI), New Delhi, India.

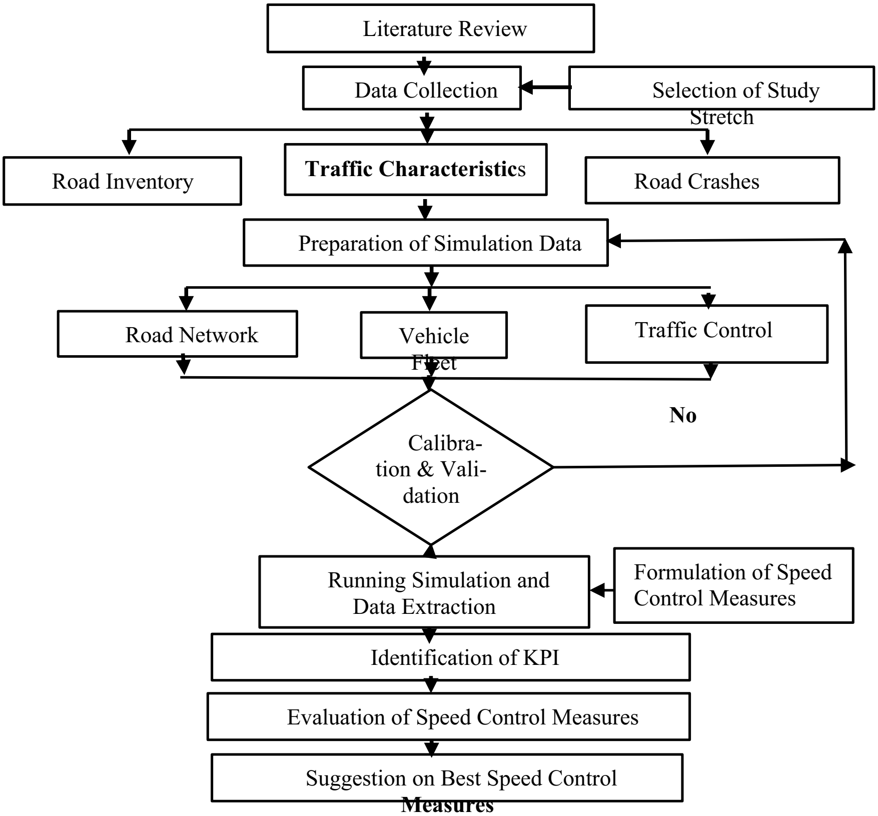

Fig. 1 Methodology adopted for the study titled as "Determining Safety Aspects of DSL on Indian Roads

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Study on Driver Behaviour at Unsignalized Intersection Using Fuzzy Logic

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Abstract In the Indian scenario, mixed traffic condition is seen, all vehicles use one lane, and the walking of pedestrians is a common sight being observed at the intersection. At an unsignalized junction, vehicles usually ignore lane discipline and rules of priority and tend to cross the junction without considering existing traffic. Due to this action, there is a risk of an accident, which affects the vehicle movement and capacity of the intersection. Therefore, it is very important to study driver behaviour which is very complex and it is influenced by traffic and vehicle characteristics. This paper deals with driver behaviour at unsignalized intersections analysed using fuzzy logic. Data from the study area is collected from videos recorded at an unsignalized T intersection for a duration of 1 h during peak hour. Recorded videos are played again for data extraction. Data like vehicle count and type, approach speed, size of gaps, accepted and rejected gaps are known. Models are developed in fuzzy logic using MATLAB with input as accepted gaps, vehicle speed and vehicle type to get the output of drivers' choice as accepting or rejecting the gap. 50% of the data is used for modeling and model evaluation is carried out using 50% of the data. The models developed can be applied to different intersections with similar characters.

Keywords Unsignalized intersection · Critical gap · Fuzzy logic · Gap acceptance

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1 Introduction

An intersection is a place where traffic of different directions meets, converge and move in the direction of their destination. The junction where vehicles and pedestrian movement is controlled by signals or by traffic police is "Signalized Intersection". The intersection where vehicle and pedestrian movement is not controlled neither by signals or by traffic is a police "Unsignalized Intersection". Since there is no objection at unsignalized intersections the drivers or pedestrians tend to cross the junction as soon as possible without considering the existing traffic. Due to improper movements, there is a risk of accidents and injuries, by which unsignalized intersections are proven to be dangerous hotspots. Since driver behaviour is complex and mixed with uncertainties less study is carried out unsignalized intersections. Other than HCM 2010 we don't have an appropriate or standard procedure to evaluate unsignalized intersections in India. So, to solve this problem driver behaviour at unsignalized intersection need to be studied. Fuzzy theory can be used under situations involving uncertainties. nowadays fuzzy theory is used in various applications like traffic signal control, transport planning, braking system, air conditioning, qualitative and quantitative analysis, etc. Driver behaviour is complex and cannot be concluded. Fuzzy theory is a good tool to assess the behaviour of driver at unsignalized intersections. Capacity, delay, level of service or any other characteristics of intersection can be obtained by accepted gaps. therefore, Gap acceptance is used to study the driver behaviour at unsignalized intersections. Gap is the smallest time gap available to cross to the junction safely between two vehicles. If a particular vehicle accepts the gap and crosses the junction, then it's "Gap accepted". If the vehicle rejects the gaps and waits for a sufficient time interval to cross the junction is known as "Gap rejectance". The acceptance of the gap depends on various factors like size of gap, type of vehicle, speed of vehicle, driver age, driver gender geometry of intersection and pedestrian movement. The final decision of accepting gaps depends on these factors which the human brain has to perceive, analyse and respond to which is very complex. Fuzzy logic theory can be used to make a decision of accepting or rejecting the gaps by considering various factors like size of gap, type of vehicle and speed of the vehicle. Models of gap acceptance are developed using Fuzzy logic with the best fit of available data.

2 Literature Review

Gap acceptance is an important objective characteristic of the behavior of drivers and it has an influence on capacities, delays, and road safety at unsignalized intersections. Then adaptive neuro-fuzzy logic was applied with three input variables size of gap, subject vehicle type, and conflict vehicle type and output variable driver's decision indicating acceptance or rejection of the gap is obtained for right turning vehicles in a study by Sangole and Patil (2014). In a study by Manish Dutta and Mokaddes Ali

Ahmed for TW and Four wheelers, statistical analysis and equations were developed. It has given critical gap values of aggressive and non-aggressive driver behaviour. Erratic maneuvers in the intersection area and aggressive driving are two common behaviors of drivers observed at these intersections (2017). The age of a driver plays a very important role in the gap acceptance process as concluded in a study by Harsh and Akhilesh (2017) and Sangolea and Patil (2011). Most of the studies carried out unsignalized intersections concluded that the age of drivers is important factor in determining gap acceptance behaviour. Keeping all the above parameters, we decided to consider the speed of the vehicle and gaps accepted by vehicles to analyse the gap acceptance behaviour at unsignalized uncontrolled junctions.

3 Data Collection

For the collection of data T arm in Rajarajeswari Nagar with heterogeneous traffic is selected in Bangalore, the Capital city of Karnataka, one of the southern states of India for study and video graphic survey is carried out at these locations for a duration of 1 h during peak hour. The intersection is selected on the basis that it lies on plain topography, no bus bay and no parking at the intersection and vehicle movement is continuous. And suitable place is also selected near the junction to place a camera covering all movements clearly to carry out a videographic survey. The selected intersections of major and minor roads are of 4-lane two-way divided carriageway 14 m with a median of 0.5 m. A videographic survey is carried out from 10:00 AM to 11:AM for 60 min on weekdays. Recorded video is played several times to extract data like vehicle movement, vehicle count, intersection crossing time, gap size, gaps accepted and gaps rejected. Two-wheelers, car, auto, LCV, HCV and bus are majorly observed types of vehicles in the intersection. Two-wheeler composition is highly observed and cycles and buses are least observed. 93 Pedestrians are observed crossing the intersection with a speed of 1.06 m/s (Fig. 1).

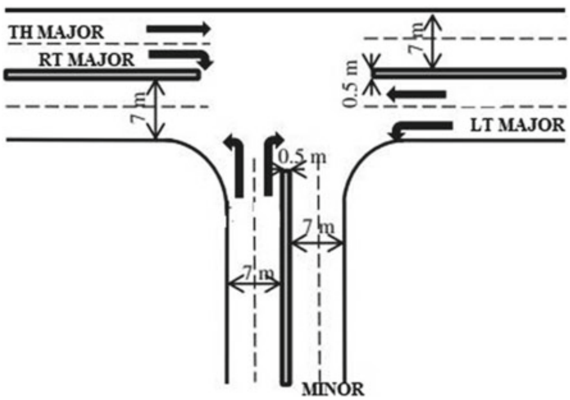

Fig. 1 Geometry of study area

Fig. 2 T intersection in Rajarajeshwarinagar, Bangalore

4 Data Analysis

Data consist of 4241 vehicles per hour. The total composition of traffic is composed of 62% of TW, 26% of cars, 8% of autos, 2 of LCVs, 1% of HCVs, 0.45% of bus and less than 1% of cycle type of vehicles movements observed in the recorded video. The average speed of vehicles is in the range of 13kmph -25kmph. The distance and time of intersection crossing are noted down and the speed of each type of vehicle is calculated. Gaps accepted and gaps rejected by each type of vehicle are also extracted. Critical Gaps are calculated using the Raff method. Among the various methods available for the calculation of the Critical gap, the Raff Method is selected because of its ease of carrying out the study. The sum of the cumulative no. of gaps accepted and gaps rejected are plotted the intersection of these curves is taken as a Critical Gap. From the Raff method, the Critical Gap we got for through moving TW is 1.7 s, car is 1.7 s, Auto is 1.7 s, LCV is 2.3 s, HCV is 2.5 s, and Bus is 2.8 s. And for right turning TW is 1.7 s, car is 1.8 s, Auto is 1.6 s, LCV is 3 s, HCV is 2.5 s, and Bus is 2.8 s (Fig. 2).

5 Descriptive Analyses of Speed and Gap accepted

The descriptive analysis is carried out using EASYFIT Software to describe, interpret and analyse the data. Descriptive analysis helps to know about mean, median, variance, variance, skewness, etc., which is helpful in sensing the data to make conclusions (Tables 1, 2, 3 and 4).

Table 1 Statistical analyses of speed of moving vehicles

Statistical parameters	TW(M)	TW(F)	CAR	AUTO	LCV	HCV	BUS
Range	50	49	44	33	30	8	16
Mean	24.57	24.81	21.65	21.09	21.82	22.57	15.5
Median	24	24	21	20	23	22	15
Variance	69.37	73.22	74.30	60.99	65.49	8.95	36.5
Std. deviation	8.33	8.56	8.62	7.81	8.09	2.99	6.04
Coef. of variance	0.34	0.35	0.40	0.37	0.37	0.13	0.38
Skewness	0.3	0.54	0.31	0.40	0.05	0.10	0.34
Min.%	7	8	3	7	8	19	9
Max.%	57	57	47	40	38	27	25

Table 2 Statistical analyses of gaps accepted by moving vehicles

Statistical parameters	TW(M)	TW(F)	CAR	AUTO	LCV	HCV	BUS
Range	9.16	10.54	11.99	9.28	9.04	2.11	3.02
Mean	3.94	3.79	4.26	3.68	3.95	3.35	3.026
Median	4	3.5	4.15	3.5	4	3.3	3.24
Variance	3.24	3.60	2.77	2.90	2.98	0.15	0.58
Std. deviation	1.80	1.90	1.66	1.70	1.72	0.39	0.76
Coef. of variance	0.45	0.50	0.39	0.50	0.44	0.11	0.25
Skewness	0.80	1.02	0.18	0.89	0.80	0.36	1.15
Min.%	1	1	1.01	1	1.12	2.01	1.22
Max.%	10.16	11.54	13	10.28	10.14	4.12	4.24

Table 3 Statistical analyses of speed by right turning vehicles

Statistical parameters	TW(M)	TW(F)	CAR	AUTO	LCV
Range	38	27	25	25	22
Mean	15.56	17.80	12.53	15.92	13.66
Median	15	17	12	16	13
Variance	47.21	38.47	30.53	35.63	38.17
Std. Deviation	6.87	6.20	5.53	5.97	6.18
Coefficient of Variance	0.44	0.35	0.44	0.37	0.46
Skewness	0.67	0.79	0.65	0.04	0.58
Min. %	4	5	3	5	5
Max. %	42	32	28	30	27

Table 4 Statistical analyses of gaps accepted by right turning vehicles

Statistical parameters	TW(M)	TW(F)	CAR	AUTO	LCV
Range	55.85	10.54	11.99	9.83	9.76
Mean	4.21	3.79	4.33	3.63	3.96
Median	4	3.56	4.18	3.45	3.95
Variance	18.51	3.57	2.87	2.94	3.41
Std. deviation	4.30	1.89	1.69	1.72	1.84
Coefficient of variance	1.02	0.50	0.39	0.47	0.47
Skewness	9.07	1.02	0.11	0.85	0.90
Min.%	0.15	1	1.01	0.45	1.12
Max.%	5.6	11.54	13	10.28	10.88

6 Distribution Fit for a Data

The data were subjected to distribution fittings in the Easy fit software for both inputs (Gap Accepted and Crossing Speed) that are planned to be used as inputs in the MATLAB fuzzy logic. Burr, General Extreme value, Gamma, Pearson, Pareto, Dagum, Cauchy. In K-S distribution analyses for Speed, the General Extreme value showed the best fit most of the time, therefore we have used the General Extreme Value and for Gaps accepted, Dagum value showed the best fit most of the time, therefore we have used Dagum distribution as a membership function in MATLAB. Table below shows the distribution parameters of speed and Gaps Accepted in Easy Fit software (Tables 5 and 6).

Table 5 Distribution data analysis of Speed

Vehicle type	Distribution	K-S		Parameters
		Static	Rank	
TW	Burr	0.0375	1	$K = 6.373, \alpha = 3.536, \beta = 44.77$
CAR	Gen Gamma	0.0430	1	$K = 2.929$ $\alpha = 0.776$ $\beta = 25.61$ $\gamma = 1.518$
AUTO	Pearson6	0.0578	1	$\alpha_1 = 7.57$ $\alpha_2 = 4.369E + 7$ $\beta = 1.22E + 8$
LCV	Gen.Pareto	0.0886	1	$K = -0.8969$ $\sigma = 25.71$ $\mu = 8.129$
HCV	Cauchy	0.1785	1	$\sigma = 2.419$ $\mu = 22.40$
BUS	Johnson SB	0.1506	1	$\gamma = 0.2356$ $\delta = 0.280$ $\lambda = 15.87$ $\xi = 8.891$

Table 6 Distribution data analysis of gap accepted

Vehicle type	Distribution	K-S		Parameters
		Static	Rank	
TW	Normal	0.056	1	$\sigma = 1.74$ $\mu = 3.94$
CAR	Dagum	0.0522	1	$K = 0.255$ $\alpha = 6.96$ $\beta = 4.79$ $\gamma = 0.93$
AUTO	Weibull	0.0560	1	$\alpha = 2.511$ $\beta = 4.107$
LCV	Johnson SB	0.0622	1	$\gamma = -0.201$ $\delta = 0.939$ $\lambda = 6.040$ $\xi = 0.3081$
HCV	Inv. Gaussian	0.074	1	$\lambda = 24.61$ $\mu = 3.348$
BUS	Dagum	0.1041	1	$K = 0.1728$ $\alpha = 3.041$ $\beta = 37{,}735$ $\gamma = 3–37{,}732$

7 Gap Acceptance Fuzzy Models

Gap Acceptance models are created to study the behaviour of drivers. Since human behaviour is very complex and uncertain, fuzzy tools are used to analyse this type of situation. Fuzzy logic in MATLAB is used to analyse the data and generate the models. The models are developed for different types of vehicles using speed and gap accepted by particular types of vehicles. Fuzzy logic toolbox is a six-layer interface system with input, fuzzification layer, rules layer, normalization layer, defuzzification layer and output layer.

In our study Input 1 is taken as gaps accepted and Input 2 is taken as the speed of vehicles to get the output of the drivers choice with NO as rejecting the gaps and YES as accepting the gaps. The distributions fits for gaps accepted and speed values of extracted data are determined using EASY FIT software. In Easy Fit software, the individual values are used and graphs are plotted to check the suitable distribution fit. Among the plotted curves General extreme value distribution shows the best fit for the speed of the vehicle and the Dagum distribution shows the best fit for gaps accepted in the K-S method. These distribution functions are applied to the Membership function in fuzzy logic with Low, Medium, and High-value ranges. Then 9 rules to process the input are applied in the rules layer. Rules are.

If “Gap” is LOW, “Speed” is LOW, then Choice is “NO”.

“Gap” is LOW, “Speed” is MEDIUM, then Choice is “NO…. So, on.

The results were obtained in the form of surfaces and rules for TW and Car are shown below Figs. 3 and 4.

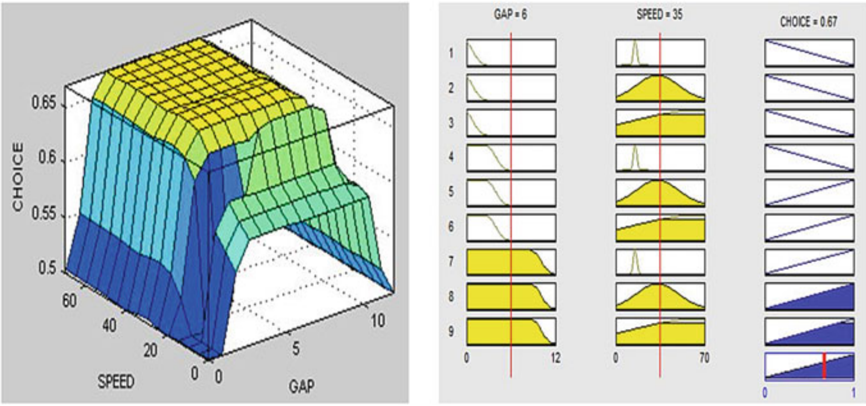

Fig. 3 Gap acceptance model for TW

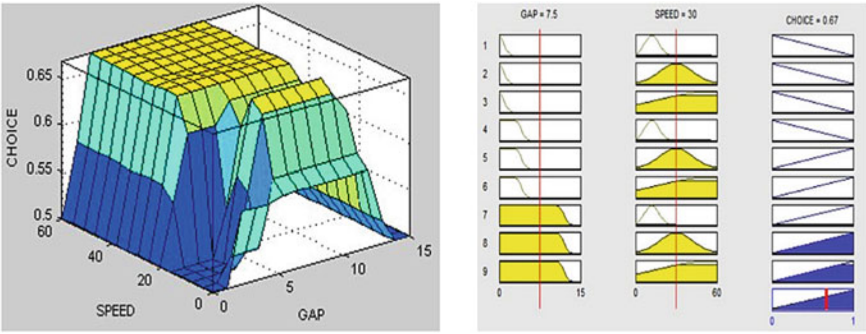

Fig. 4 Gap acceptance model for Car

8 Conclusion

- The average speed of two wheelers is 25 kmph, Car is 22 kmph, Auto is 21 kmph, LCV is 22 kmph, HCV is 23 kmph and Bus is 15 kmph.
- The average gaps accepted of Two-wheelers are 1.7 s, Car is 1.7 s, Auto is 1.7 s, LCV is 2.3 s, HCV is 2.5 s and for the Bus is 2.8 s.
- Distribution Fit for speeds of different vehicles we got using Easy Fit Software are Burr, General Extreme value, Gamma, Pearson, Pareto, Dagum, Cauchy, Chi-Squared, Johnson SB, Pert, Chi-Square. Out of these distributions, In K-S distribution analyses for Speed, General Extreme value showed the best fit.
- Distribution Fit for Gaps accepted for different vehicles we got using Easy Fit Software are General Extreme value, Gamma, Pearson, Pareto, Dagum, Cauchy, Chi-Squared, Johnson SB, Pert, Chi-Square, Gum bell, Nakagami, Weibull, and

log Pearson. Out of these distributions, In K-S distribution analyses for gaps accepted, Dagum value showed the best fit.

- Gap acceptance models are developed for different types of vehicles in MATLAB to study the driver's behaviour

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Capacity Analysis and Safety Assessment of Unsignalized Intersection Using Conflict Technique

P. H. Souparnika, Sheela Alex, and Padmakumar Radhakrishnan

Abstract Intersections pose special safety concerns because of the high probability of critical conflicts resulting from unsafe driver actions and maneuvers. The absence of movement priorities, lack of lane discipline, forced entry by non-priority movements, etc., at unsignalized intersections violate the assumptions involved in capacity calculation. A key aspect of this study is to analyze conflicting flows at unsignalized intersections, which vary depending on site conditions and geometrical features in heterogeneous traffic environments and to estimate movement capacity at unsignalized intersections. At unsignalized intersections, vehicles from different directions cross and turn simultaneously, resulting in severe vehicle-vehicle conflicts. Thus, traffic safety is an important aspect to be evaluated at unsignalized intersections. The study also aims at conducting a safety assessment at unsignalized intersections using the microsimulation model (VISSIM) coupled with the Surrogate Safety Assessment Model (SSAM). The study found that SSAM is adaptable to intersections of varying geometry and serving heterogeneous traffic.

Keywords Unsignalized intersection · Capacity · Vissim · Ssam

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1 Introduction

Unsignalized intersections frequently see serious conflicts between crossing traffic due to erratic traffic maneuverability. Drivers are highly aggressive; therefore, the rules of priority are often ignored at unsignalized intersections and also lane discipline is nearly non-existent. Hence, the capacity estimation procedure followed in developed countries is inapplicable to intersections having mixed traffic conditions. The study intended to estimate movement capacities at unsignalized intersections under heterogeneous traffic. For the capacity analysis at unsignalized intersections, conflicting flow is determined using the volume of traffic and relative levels of hindrance brought by higher priority movements to the subject movement. Indo HCM [1] explains in detail how the conflicting flow computation varies based on intersection geometry and the relative influence of conflicting movements on subject movement. With this motivation, the study examines how the conflicting flow varies based on site geometry and the influence of different conflicting movements on the subject stream at field, to compute movement capacity at unsignalized intersections. The major advantage of this traffic conflict technique (TCT) based analysis is that it examines real-time interactions of traffic prior to the occurrence of a crash.

The conventional approach of crash data analysis includes many drawbacks such as long observation period, lack of crash data, and inaccurate information about the crash patterns. Surrogate safety measures (SSM) were used as a substitute for traditional crash data analysis for a more accurate and rapid evaluation of safety at the site [2]. Therefore, safety assessment is also conducted at selected locations using the surrogate safety measures. The objective of the study is to determine conflicting traffic flow rates based on site geometry and the relative impact of conflicting movements on subject vehicle and to compute site-specific capacity of unsignalized intersections using conflict technique. Also, the vehicle conflicts at selected unsignalized intersections were determined and categorized into various types using the microsimulation model, VISSIM coupled with Surrogate Safety Assessment Model (SSAM).

2 Literature Review

Joewono et al. [3] conducted an analysis of unsignalized intersections with conflict method for capacity estimation under mixed traffic conditions of non-priority intersections. The bending of right-of-way at unsignalized intersections causes the different priority ranking of traffic streams and provides a modification of procedure to determine the volume of the priority stream for capacity calculation [4]. Ghanim and Shaaban [5] investigated the feasibility of using SSAM to identify and classify traffic conflicts between vehicles and pedestrians by analyzing simulated trajectories. Mohan and Chandra [6] focused on developing the entire procedure for estimating the capacities of movements at unsignalized intersections. Using Harder's capacity model as a base, the procedure to estimate the parameters of this model was revised to suit the traffic operations in developing countries. Arkatkar et al. [7] conducted a safety evaluation of unsignalized intersections using PET for through movement as well as turning movements and also analyzed the critical speed of the conflicting vehicles at the intersection for assessing the extremity of conflicts using PET values. Safety assessment at intersections in urban areas is conducted using a micro-simulation model, VISSIM coupled with the Surrogate Safety Assessment Model (SSAM) [8]. The number, type and severity of conflicts were determined in the study and conflicts were categorized into three types according to the conflict's angle, as rear-end, lane change and crossing conflicts. Goyani et al. [9] attempted to investigate crossing conflicts at unsignalized T-intersections under mixed traffic conditions using post-encroachment time (PET) as a surrogate safety measure (SSM). Paul and Ghosh [10] proposed an approach to determine the PET threshold for carrying out an efficient and faster safety evaluation at unsignalized intersections under highly heterogeneous traffic conditions. From the literature reviewed, it is clear that limited studies analyzed the capacity of unsignalized intersections by conflict technique and recent studies explored the viability of using Surrogate Safety Measures (SSMs) to identify and categorize traffic conflicts between vehicles by evaluating simulated trajectories.

3 Methodology

The entire procedure adopted for this study is depicted in the form of a flowchart as shown in Fig. 1.

3.1 Site Selection

The intersection having low to medium flow with sufficiently flat gradients possessing three-legged as well as four-legged configurations were considered for the study. Fig. 2. shows the Vazhuthacaud junction and Sreekaryam junction at Trivandrum city and Chadayamangalam junction situated at Kollam.

3.2 Data Collection

The traffic data were collected from 1.00 pm to 2.00 pm in a day during off-peak hours at each unsignalized intersection using a video-graphic survey. A geometric survey was also conducted to collect road inventory details. Table 1 shows the geometric details of selected intersections.

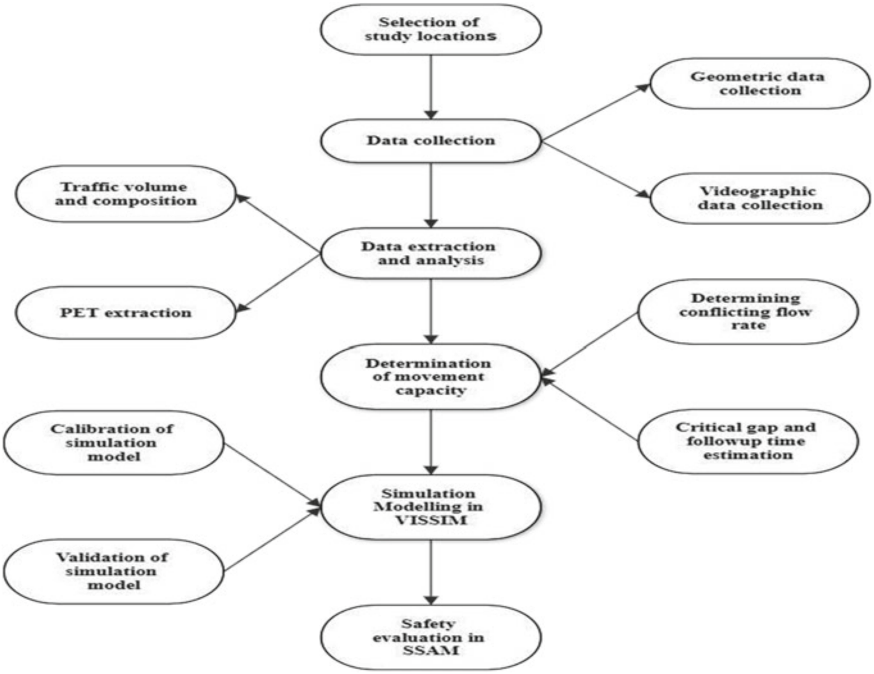
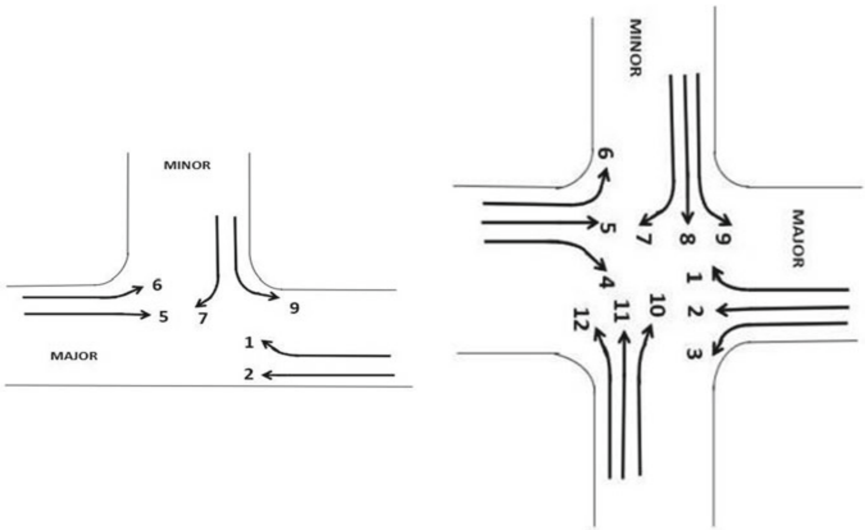


Fig. 1 Flowchart of the methodology

Fig. 2 Study locations: **a** Vazhuthacaud Junction **b** Sreekaryam Junction **c** Chadayamangalam Junction

Table 1 Geometric details of the selected site

Intersection	Intersection type	Major street configuration	Carriageway width (m)
Vazhuthacaud	Three-legged	Four-lane divided	18
Chadayamangalam	Three-legged	Two-lane undivided	7.5
Sreekaryam	Four-legged	Two-lane undivided	7.5



(a) (b)

Fig. 5 Vehicular movements at **a** Three-legged intersection **b** Four-legged intersection

3.3 Data Extraction and Analysis

From the collected video, vehicle composition, classified volume count, PET values, and gap and lag for different vehicle classes for critical gap estimation were extracted.

Vehicle Categories and Composition

Figure 5. shows the vehicular movements at three-legged and four-legged intersections.

Traffic volume and composition were extracted from the video-graphic recording by a manual count at three unsignalized intersections. The volume of traffic executing each turning movement at the intersection is converted into the equivalent number of passenger cars using the PCU values obtained from [1] and presented in Table 2.

Determination of Post-Encroachment Time (PET)

Post-Encroachment Time (PET) is the time difference between when the first vehicle leaves the conflict area, and the time when the second vehicle enters the conflict area. The data extraction was done manually from the video using the grid line technique. Kinovea™ software was used to extract PET values from the field. The PET values are calculated by noting down two time events T1 and T2.

$$\mathrm{PET} = \mathrm{T}_2 - \mathrm{T}_1 \tag{1}$$

Table 2 Traffic converted into PCUs

Vazhuthacaud junction						
Movement	2W	CAR	3W	LCV	HCV	Total (PCU/hr)
1	169	367	126	31	5	698
2	270	426	179	49	7	931
5	240	482	160	71	5	958
6	71	162	36	17	5	291
7	110	209	87	7	5	418
9	66	251	5	14	5	421
Chadayamangalam junction						
Movement	2W	CAR	3W	LCV	HCV	Total (PCU/hr)
1	66	92	86	71	19	334
2	94	328	141	126	124	813
5	109	399	136	136	141	921
6	57	56	86	14	19	232
7	45	80	75	27	10	237
9	46	100	102	41	10	299
Sreekaryam junction						
Movement	2W	CAR	3W	LCV	HCV	Total (PCU/hr)
1	116	241	64	27	12	460
2	239	412	114	82	50	897
3	14	36	31	20		101
4	23	28	20	9		80
5	135	422	106	88	133	884
6	27	60	24	26		137
7	50	80	24	37		191
8	63	65	30	41		199
9	122	221	67	27	29	466
10	22	42	31	21		116
11	61	72	30	24	14	201
12	17	36	11	44		108

where T1 = Time, when the offending vehicle leaves the conflict area; T2 = Time, when the conflicting vehicle enters the conflict area. Fig. 4. shows the screenshot of PET extraction. The extracted PET values show which conflicting movements are critical, i.e. PET< 1.0 s at field [10] and based on that, the impact of conflicting movements on subject movement can be found out and also number of critical conflicts existing at site.

Determination of Conflicting Flow Rates and Capacity Estimation

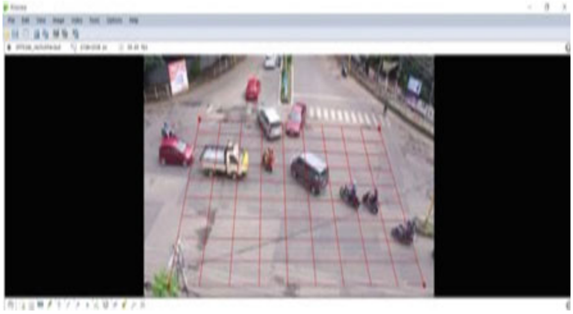


Fig. 4 Screenshot of PET extraction in Kinovea software

Conflict occurs between two movements at an intersection when they compete for a common right of way and conflicting flow refers to the cumulative volume of traffic from different streams that will have an impact on the operation of a non-priority movement at an intersection. Different weights were assigned ranging from 0 to 2 for different conflicting streams to account for the impact of these movements on the subject stream [1]. A weightage of '0' implies having no significant influence, while a weightage of '2' indicates an extensive effect on the driver's gap acceptance behavior. These weights were assigned on the basis of site geometry and the relative impact of conflicting movements on subject movement at the field [1]. Any movement of higher priority with which the subject movement shares the right of way is included as a conflicting flow for that subject movement. In India, priority among movements is rarely followed, thus movements of lower priority might cause hindrance to the movements of higher priority. Hence, lower priority movement is also to be considered while calculating the rate of conflicting flow. Table 3 shows the conflicting flow computation at selected locations.

From the field observation, it is clear that through traffic on the major street was observed to have the highest priority owing to their higher speed and ease of clearing the conflict area. Right turners from major and minor streets experience a high risk of conflict from through movements from the major street. Through and left-turning movements from major and minor streets enjoy the highest priority and are unimpeded by any other movements. In calculating conflicting flow at three-legged intersections having four-lane divided major street configuration, right turning traffic from major, i.e. movement1 is less impeded by left turns from major street. Hence, the weightage for left-turning traffic, in this case will be 0.5. Also at such intersections, due to the absence of lane discipline, right turns from minors are observed to be waiting at the intersections along the center or right side of the approach hindering the subject movement. Hence, the right turners on minor street approach, i.e. movement 7 should also be considered in computing the conflicting flow for right turn from major and is assigned a weightage of 1. The opposing major street through traffic was found to have a greater influence on right turners from major and hence a weightage of 1 is assigned to through movements on major. And for computing conflicting flow for right turn from minor, that is movement 7, the through traffic and right turn from major have greater effect on the subject movement and are assigned a weightage of 1

Table 3 Conflicting flow computation

Vazhuthacaud Junction		
Movement	Conflicting flow	Vcx (PCU/hr)
VC1	V5 + V7 + 0.5V6	1521
VC7	V5 + V1 + 0.5V2	2121
Chadayamangalam Junction		
Movement	Conflicting flow	Vcx (PCU/hr)
VC1	1.5V5 + V6 + V7	1850
VC7	V5 + V1 + V2	2068
Sreekaryam Junction		
Movement	Conflicting flow	Vcx (PCU/hr)
VC1	1.5 V5 + V6 + V7	1654
VC4	1.5V2 + V3 + V10	1563
VC7	V4 + V5 + V1 + V2	2321
VC10	V1 + V2 + V4 + V5	2321
VC8	V4 + V5 + V1 + V2 + V3 + V10	2538

and also merging through movement found a lesser effect on the subject movement, is assigned a weightage of 0.5. At the same time, when the major street is of two-lane undivided configuration and if left turning movement is not channelized, in such case, left turners from the major street will conflict with the subject movement as well, so its effect is considered. Moreover, when there is no divider on the major street, the impact of the major street through movement on the subject movement is enhanced, thus giving a weightage of 1.5.

Capacity for any movement at an unsignalized intersection is computed using Equation 2 given in [1]. Table 4 shows the movement capacity at selected locations. Site-specific critical gaps for different movements were obtained using the base critical gap values. Raff's method is used for determining base critical gap values. The critical gap for any movement is obtained using Equation 3 given in [1]. The follow-up time is obtained by taking 60% of the critical gap value [1]. Table 4 shows the movement capacity at selected locations.

$$C_x = \frac{a \times V_{cx} \times e^{\frac{-V_{cx}(t_{cx}-b)}{3600}}}{1 - e^{\frac{V_{cx} t_{fx}}{3600}}} \quad (2)$$

Cx = Capacity of movement 'x' (PCU/h)

Vc, x = Conflicting flow rate corresponding to movement x (PCU/h) tc,x = Critical gap of standard passenger cars for movement 'x' (s) tf,x = Follow-up time for movement 'x' (s)

'a' and 'b' = Adjustment factors based on intersection geometry

Table 4 Movement capacity at study location

Vazhuthacaud Junction								
Movement	tc,base	fHV	PHV	tcx	tfx	a	b	Cx (PCU/hr)
1	2.7	0.46	6.6	3.57	2.14	0.8	1.3	784
7	3.8	0.88	4.3	5.08	3.05	1.0	2.16	456
Chadayamangalam junction								
Movement	tc,base	fHV	PHV	tcx	tfx	a	b	Cx (PCU/hr)
1	3.5	0.78	4.2	4.62	2.77	0.7	-0.11	385
7	3.8	0.01	3.68	3.81	2.29	0.8	0.72	383
Sreekaryam junction								
Movement	tc,base	fHV	PHV	tcx	tfx	a	b	Cx (PCU/hr)
1	3.2	0.78	4.18	4.3	2.58	0.7	-0.11	468
4	3.2	0.78	5.7	4.5	2.7	0.7	-0.11	214
7	3.6	0.01	5.78	3.62	2.2	0.8	0.72	377
10	3.6	0.01	6.17	3.62	2.29	0.8	0.72	377
8	4.2	0.07	5.1	4.31	2.58	1.1	0.72	265
12	4.2	0.07	5.7	4.32	2.53	1.1	0.72	242

$$t_{cx} = t_{c,base} + f_{HV} \times \ln(P_{HV}) \quad (3)$$

tc, base = Base critical gap value (s)
fHV = Adjustment factor for large vehicles
PHV = proportion of large vehicles in the conflicting traffic stream.

Determination of LOS for Movements

LOS is defined as the level of perception of the service offered by a facility by the user. In the traffic engineering context, it is arrived at using measures of effectiveness such as volume/capacity ratio (v/c), delay and density Indo HCM [1] classified LOS for different v/c ratios. LOS derived based on the observed volume-capacity ratio at the intersections is shown in Table 5.

LOS E for a movement implies that the movement is overloaded and operating at near capacity. It requires geometric improvements at the intersection for its stable flow.

Simulation Modelling Using VISSIM

VISSIM is a microscopic time step and behaviour-based traffic simulation model that efficiently replicates heterogeneous traffic flow and it is widely used today to simulate heterogeneous traffic flow. The selected intersections were modelled in VISSIM by assigning geometric features, traffic composition, vehicle volume, vehicle routes and speed, so that it represents the actual field conditions. Fig. 6 shows the modelling done in VISSIM. The developed VISSIM model is calibrated by comparing travel time values obtained from the field and VISSIM and Mean Absolute Percentage

Table 5. LOS calculation for movements

Vazhuthacaud junction			
Movement	V/C Ratio	LOS	Condition
1	0.89	E	Unstable flow
7	0.92	E	Unstable flow
Chadayamangalam junction			
1	0.86	E	Unstable flow
7	0.62	D	High-density flow
Sreekaryam junction			
1	0.98	E	Unstable flow
4	0.37	C	Stable flow
7	0.5	C	Stable flow
10	0.3	B	Stable flow
8	0.72	E	Unstable flow
11	0.83	E	Unstable flow

Error (MAPE) values were found to be within allowable limit, i.e. less than 15%. The validation of the model was done by evaluating a calibrated simulation model with a new set of field data. The MAPE values were found to be within the allowable limit, i.e. less than 15%.

Safety Assessment Using SSAM

After developing the simulation model in VISSIM, the vehicle trajectory files were exported to SSAM software. Several indicators for traffic conflicts were computed by SSAM, involving post encroachment time (PET), time to collision (TTC), max speeding (Max S), the rate of deceleration (DR) and max deceleration (Max.D). The number and type of conflicts existing at the site were also evaluated by SSAM. The conflicts were categorized into three types according to the conflict's angle rear-end, lane change and crossing conflicts. The conflicts are classified according to the absolute value of the conflict angle. The conflict type is classified as a rear-end conflict if $\|$the conflict angle $\| < 30°$, a crossing conflict if $\|$the conflict angle $\| > 85°$ and a lane change conflict if $30° \leq \|$conflict angle$\| \geq 85°$. The SSAM computes

Fig. 6 Simulation modelling done in VISSIM

several surrogate safety measures, among these the most important indicators, post-encroachment time (PET) and time to collision (TTC) were used for analysis. Fig. 7a, 7b and 7c shows the TTC and PET variations for different conflict types at selected intersections.

Table 6 shows the SSAM output results of selected intersections and Table 7 shows the comparison between crossing conflicts obtained at field and SSAM. The crossing conflicts obtained from the field and SSAM shows close approximation, thus SSAM is the best tool to perform safety analysis at the field.

4 Conclusions

Conflicting flow for any movement should include all those movements of higher priority with which the subject movement competes for the right of way. However, due to the absence of movement priorities at uncontrolled intersections in India, the conflicting movements and their contributions towards conflicting flow rates will be different. Based on the site geometry and relative impact of conflicting movements on subject movement, the computation of conflicting flow rates was modified. Using these conflicting flows, the movement capacity was determined. Thus, the proposed methodology can be used for finding the capacity of movements at uncontrolled intersections functioning under heterogeneous traffic conditions. Also, the determination of the V/C ratio helps to find out the LOS for each movement and to know the prevailing conditions of movements at the site. The study helps to suggest geometric improvements required at intersections to make flow stable for different movements. The safety assessment done at selected intersections using VISSIM and SSAM computes various surrogate safety measures, classifies conflicts according to conflict angles and also determines the total number of conflicts at intersections. Lower traffic and congestion at the Chadayamangalam intersection result in a higher value of mean TTC and PET value which lies above 2.5 s. Higher traffic flow at the Sreekaryam intersection having a four-legged configuration showed a lower value of mean PET value of about 2.26 s compared to other intersections. Heavy through movements and more number of turning traffic at the Sreekaryam intersection results in greater number of conflicts compared to other intersections. The queue formation at the site by the drivers to find a suitable gap for their desired maneuvers generates a higher percentage of rear-end conflicts at all intersections. Compared to other conventional safety analysis, the use of simulation software VISSIM and SSAM consumes less time for safety evaluation. By extracting path files constructed using VISSIM, SSAM classified conflicts according to conflict angle and also determined the number and severity of conflicts at intersections.

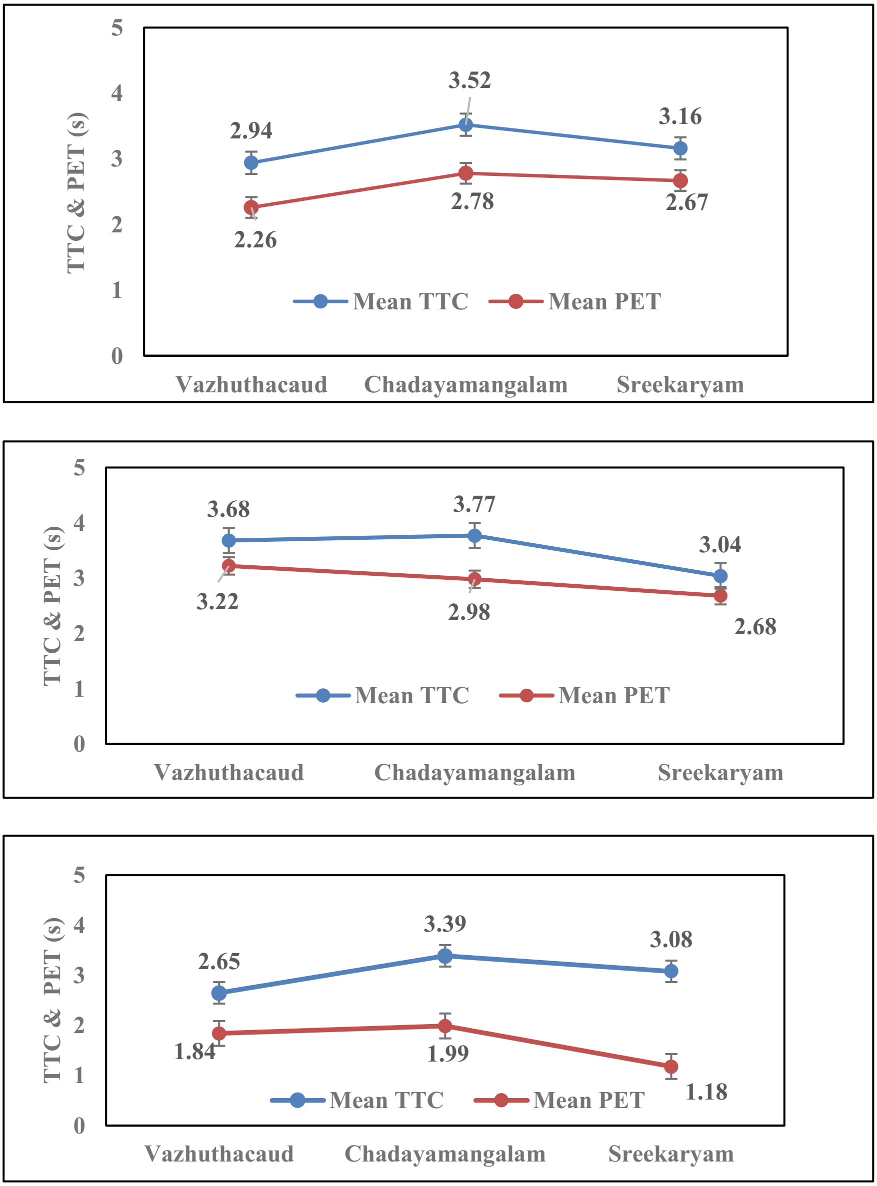


Fig. 7. **a** TTC and PET variations for crossing conflicts at selected intersections. **b** TTC and PET variations for lane change conflicts at selected intersections. **c** TTC and PET variations for rear end conflicts at selected intersections

Table 6 SSAM output results of selected intersections

	Vazhuthacaud	Chadayamangalam	Sreekaryam
Mean TTC (s)	2.48	2.95	2.59
Mean PET (s)	2.37	2.63	2.26
Crossing conflicts	278	225	395
Lane change conflicts	133	171	235
Rear end conflicts	426	527	697
Total conflicts	837	923	1327

Table 7 Comparison of crossing conflicts obtained from field and SSAM

Intersections	No. of crossing conflicts from field	No. of crossing conflicts from SSAM	Percentage difference
Vazhuthacaud	249	278	11.64
Chadayamangalam	198	225	13.64
Sreekaryam	354	395	11.58

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A Statistical Approach to Estimate Gap Acceptance Parameter at Three-Legged Uncontrolled Intersection

Khushbu Bhatt and Jiten Shah

Abstract The estimation of the gap acceptance parameter for right-turning movements at uncontrolled T-intersections is the main focus of this paper. It takes a lot of effort to investigate an intersection that is unsignallised or is uncontrolled. In general, a driver's perception of the priority at an intersection is based on the volume of traffic, the design of the intersection, and the speed of the vehicle on main and minor approaches. Three uncontrolled T-intersections in the field were recorded as video in order to acquire the data. Accepted gaps, rejected gaps, vehicle types, and traffic composition are the data that were extracted from the video. The accepted and rejected gaps are fitted with various statistical distributions. The best-fitted model is recommended for the accepted and rejected gap based on goodness of fit tools, and the critical gap is estimated. Additionally, the critical gap value determined by best-fitted distributions (log-normal distribution) is compared to the values determined using Indo-HCM (Indo-HCM: Indian Highway Capacity Manual (Indo-HCM). CSIR-Central Road Res. Institute, New Delhi. (2017)). The value of the gap obtained using detailed traffic parameters can be further used for risk prediction to reduce the severity of crashes at uncontrolled intersections under mixed traffic conditions.

Keywords Gap acceptance · Maximum likelihood estimate · Critical gap

1 Introduction and Background

In developing countries, like India, the unsignallised intersection does not follow priority rules for right-turning movement. The drivers themselves decide the acceptance or rejection of the gap which depends on the geometry, speed of vehicles, type

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of conflicting vehicles, etc. Hence, the gap accepted or rejected at intersections plays a very vital role in right-turning movement at an uncontrolled intersection. The gap is the minimum space or time interval within which the vehicle will safely make a maneuver from a major street to a minor street or vice-versa. The tendency of drivers that intellects the opportunity to move further to cross the intersection safely is known as gap acceptance behavior. This concept is useful for switching the traffic stream from major to minor or even in case of lane changing. The gap acceptance parameter generally depends on the gap and lag of the vehicle in the field. However, to evaluate the behavior of the driver for acceptance or rejection of the gap further depends on many other parameters like occupancy time, conflicting vehicle type and speed, etc.

The gap acceptance parameter is the micro-level traffic characteristics that explain the interaction of traffic when the minor street vehicle enters the major street. The critical gap and follow-up time are the two important parameters of gap acceptance. As per the Indian Highway Capacity Manual [1] the critical gap is defined as the minimum time between successive major-stream vehicles in which minor stream vehicles make a manoeuver. The value of the critical gap lies between the accepted and the largest rejected gap. Critical Gap is the smallest gap that a driver is willing to accept to merge with the conflicting traffic and mainly determines the gap acceptance behavior of the driver. A vehicle making the turn to either direction increases the risk of road accidents if the accepted gap is less than that of the critical gap. Moreover, the value of the critical gap is used to calculate the capacity of the intersection, delay, and level of service [2, 3]. The follow-up time also plays a crucial role to ensure the driver's safety. The follow-up time is defined as the time between two successive vehicles from the minor street entering the conflict area using the same gap. The follow-up time is generally considered to be 0.6 times the critical gap [1]. The estimation of the critical gap is a challenging task because it cannot be directly deduced from the data obtained from field observations, it needs to be estimated based on the accepted or/and rejected gap.

A minor street driver at an intersection needs to judge the gap and whether it is suitable to cross the traffic stream safely. In general, the driver accepts all gaps larger than the critical gap and rejects the other gap [4]. Hence, the critical gap is the least value of the gap that is acceptable to a driver. In the Highway Capacity Manual [1], the terminology of the critical gap is replaced with the critical headway, however, the universal applicability of the term has not been justified. The critical gap is not a constant value but varies on individual perceptions of drivers. The various parameter that affects the value is a vehicle type, the geometry of intersection, delay, gradient, etc. [5–8] but the value is independent of conflicting traffic volume [9, 10]. As critical gaps cannot be measured in the field, continuous efforts are made by researchers to develop new techniques to estimate the mean critical gap. Therefore, different methods are recognized for estimating the critical gap which broadly classifies as a deterministic, probabilistic, stochastic, and fuzzy model. A deterministic method is a conventional approach that provides a single average value whereas the probabilistic method solves the inconsistency elements in gap acceptance behavior by using a statistical approach [11]. From the past literature, few studies have been claimed that the maximum likelihood method estimates the precise results as compared to other methods like the Raff's method underestimates the value whereas the Ashworth method overestimates the value of the critical gap. Hence in this paper, MLE is focused on considering different distributions of the accepted and rejected gap. The use of the different distribution is to understand the realistic behavior of the driver on the section. The present paper aims to estimate the critical gap by applying the maximum likelihood estimation (MLE) process by ten different statistical distributions. Further, to check the suitability of the distribution, the gap acceptance data are analyzed by the goodness of fit. The best-fitted model for the critical gap is chosen based on the Akaike Information Criterion (AIC).

Several models and distributions have been developed by various researchers, based on the traffic situation, capacity, field conditions, vehicle specifications, human behaviour, and other elements for developed and developing countries. The comparative analysis of the various models for estimating the critical gap by data required, methodology, and its limitations [12].

Based on the assumptions and limitations of the different methods for estimation of the critical gap, a simple technique of MLE is proposed in this paper by understanding the realistic behavior of drivers at the intersection.

2 Approach for Estimating Driving Gap Acceptance Behavior

As different researchers have suggested, the maximum likelihood method is the best when compared with the other methods. But, it has the assumption of the driver to be constant and homogeneous which is practically impossible, especially, in the case of the heterogeneous traffic condition [13]. Hence, the approach of maximum likelihood is used in this paper with consideration of the individual driving behavior. The gap acceptance behavior of the driver is being observed using the different distributions and the best-fit model is used for the estimate of the value of the critical gap.

2.1 Maximum Likelihood Estimation

In this section, the methodology of the Maximum Likelihood Estimation (MLE) [14] procedures for fitting accepted and rejected gaps and critical gaps is discussed in detail. The MLE is one of the popular and widely used methods of estimation and it holds prominent properties such as consistency, asymptotic normal, and invariance. Suppose X and Y denote the accepted and rejected gap time, respectively, and follow the same distribution function with probability distribution function (pdf), $f_X(x, \theta)(or f_Y(y, \theta))$ with unknown parameter θ. The likelihood function for the accepted gap time X is defined by.

$$\updownarrow\left(\theta|\underline{x}\right) = \prod_{i=1}^{n} f_X(x_i, \theta) \tag{1}$$

where $\underline{x} = \{x_1, x_2, \ldots, x_n\}$ the observed sample is on accepted gap time X and n denotes the number of drivers. Similarly, the likelihood function Y can be defined by

$$\updownarrow\left(\theta|\underline{y}\right) = \prod_{i=1}^{n} f_Y(y_i, \theta), \tag{2}$$

where $y = \{y_1, y_2, \ldots, y_n\}$ is the observed sample on rejected gap time Y. For the given data, the MLE estimate of the unknown parameter θ is the value that maximizes the likelihood or log-likelihood function.

It illustrates the procedure for Weibull distribution and other distributions that can be implemented similarly. The pdf of the Weibull distribution is given by

$$f_W(x) = \alpha\theta x^{\alpha-1} e^{-\theta x^{\alpha}}, \theta > 0, \alpha > 0, x > 0. \tag{3}$$

The log-likelihood function for the Weibull distribution is given by

$$\mathrm{Log}\updownarrow\left(\theta, \alpha|\underline{x}\right) = n\log(\theta) + n\log(\alpha) + (\alpha - 1)\sum_{i=1}^{n} \log(x_i) - \theta \sum_{i=1}^{n} x_i^{\alpha}. \tag{4}$$

The MLEs of α and θ are the numerical values, at which, the $\mathrm{Log}\updownarrow(.)$ function achieves its maximum. The usual optimization procedure gives

$$\widehat{\theta}_{mle} = \frac{n}{\sum_{i=1}^{n} x_i^{\widehat{\alpha}_{mle}}} \tag{5}$$

where $\widehat{\alpha}_{mle}$ is the solution of the non-linear equation

$$\frac{n}{\alpha} + \sum_{i=1}^{n} \log(x_i) - \frac{n}{\sum_{i=1}^{n} x_i^{\alpha}} \sum_{i=1}^{n} x_i^{\alpha} \log(x_i) = 0. \tag{6}$$

The equation above can be numerically solved using Newton's method. Further, the MLE procedure for estimating the critical gap parameter using the accepted and rejected gap times given for each driver. The procedure is based on the fact that critical gap time cannot be directly observed on the field but it is known that its value (say Z) lies in between accepted and rejected gap times with probability $P[Y < Z < X] = F(X) - F(Y)$, where $F(.)$ denotes the distribution function assumed for the critical gap variable. The assumption has been made for the Z function, as it follows the Weibull distribution. The likelihood function is given by

$$\updownarrow\left(\theta, \alpha|\underline{x}, \underline{y}\right) = \prod_{i=1}^{n}[F(x_i) - F(y_i)] = \prod_{i=1}^{n}\left[e^{-\theta {y_i}^{\alpha}} - e^{-\theta {x_i}^{\alpha}}\right], \tag{7}$$

where $x_i\,(y_i)$ denotes rejected and accepted gap time for i th driver. The MLEs of α and θ can be obtained by solving the following log-likelihood equations

$$\frac{\partial \mathrm{Log}\ell\left(\theta,\ \alpha|\underline{x}, \underline{y}\right)}{\partial\theta} = \sum_{i=1}^{n}\frac{\left[x_i^{\alpha}e^{-\theta x_i^{\alpha}} - y_i^{\alpha}e^{-\theta y_i^{\alpha}}\right]}{\left[e^{-\theta y_i^{\alpha}} - e^{-\theta x_i^{\alpha}}\right]} = 0,\ \frac{\partial \mathrm{Log}\ell\left(\theta,\ \alpha|\underline{x}, \underline{y}\right)}{\partial\alpha}$$
$$= \sum_{i=1}^{n}\frac{\left[x_i^{\alpha}\log(x_i)e^{-\theta x_i^{\alpha}} - y_i^{\alpha}\log(y_i)e^{-\theta y_i^{\alpha}}\right]}{\left[e^{-\theta y_i^{\alpha}} - e^{-\theta x_i^{\alpha}}\right]} = 0 \tag{8}$$

The equations given above need to be solved numerically using Newton's method. Once the MLEs $\widehat{\theta}_{mle}$ and $\widehat{\alpha}_{mle}$ are obtained, the MLE of the critical gap (mean), variance, and median are given by

$$\widehat{CR}_{mle} = \frac{\Gamma\left(1 + 1/\widehat{\alpha}\right)}{\widehat{\theta}^{1/\widehat{\alpha}}}, \tag{9}$$

$$\widehat{StD}_{mle} = \hat{\theta}^{-2/\hat{\alpha}}\left\{\Gamma\left(1 + {}^{2}/_{\hat{\alpha}}\right) - \Gamma^{2}\left(1 + {}^{1}/_{\hat{\alpha}}\right)\right\}, \tag{10}$$

$$\widehat{Md}_{mle} = \widehat{\theta}^{-1/\widehat{\alpha}}\left[\log 2\right]^{1/\widehat{\alpha}}. \tag{11}$$

A similar procedure can be applied to perform estimation for other probability distributions. After the MLE, the values of AIC are used to measure the goodness-of-fit for the accepted gap, rejected gap, and critical gap.

The Akaike Information Criterion (AIC) is a statistical measure used for the selection of the best possible model for the given data set. The AIC has two components; one is based on likelihood and the other includes the number of parameters to be estimated through the data. The later part penalizes the model for having a higher number of parameters that increase the computational costs. Hence, AIC is applied in this study to investigate the risk of underfitting and overfitting of the data. A smaller AIC value corresponds to a better-fitting model. It is mathematically represented as.

$$\mathrm{AIC} = 2\mathrm{k} - 2\ \ln(), \tag{12}$$

where k is the number of estimated parameters in the model; Ĺ is the maximum of the likelihood function for the model.

3 Data Collection

The survey was conducted for three locations to analyse the gap acceptance behaviour of Indian drivers on similar geometric sections. The data collection had been done for different vehicle types at uncontrolled T- Intersection of state highways. The videography for classified traffic volume count was conducted for 12 h from 9:00 AM to 9:00 PM on working days in fair weather conditions. The accepted and the rejected gaps were calculated from the video for one peak hour in the morning and evening. Presently, three rural intersections of Gujarat are considered Nadiad (SH 60)–L1, Anand (SH 83)–L-2, and Halol (SH 5)-L3. All three selected intersections are four-lane divided–major roads and two-lane undivided- minor roads. The locations are shown in Fig. 1.

The videography was done by placing a camera at a vantage point to obtain precise data as per the standard guidelines. The geometric and traffic details of all the sections are represented in Table 1.

Fig. 1 Study location as T- intersection **a** Nadiad (L1) **b** Anand (L2) **c** Halol (L3)

Table 1 Details of the study locations

Location	Nadiad (SH 60)-L1	Anand (SH 83)-L2	Kalol (SH 193)-L3
Geometry	Three-legged Intersection	Three-legged Intersection	Three-legged Intersection
Section–Major Road Minor Road	Four-lane divided Two-lane undivided	Four-lane divided Two-lane undivided	Four-lane divided Two-lane undivided
Height of Camera	15 m	13 m	12 m
Width of Major road	15 m	15 m	17.5 m
Width of Minor road	10.5 m	10.5 m	7.5 m
Conflict area	16 m*20 m	16.6*14 m	18*20 m

4 Data Extraction

The traffic volume is extracted manually from the recorded video for classified vehicles. Subsequently, the traffic counts are converted into equivalent passenger cars; using PCU values provided by Indo-HCM 2017.

The data extracted denotes the major proportion of two-wheeler traffic at all intersections and due to the smaller area of vehicles, the motorized vehicles have more aggressive behavior at the intersection during maneuver [15]. Therefore, the critical gap for two-wheelers is analyzed for all three intersections in this paper. The values are estimated using various statistical models using the maximum likelihood approach and then are compared with the other methods.

4.1 Accepted and Rejected Gap

Data for the measurement of accepted gaps is extracted for each possible movement. For understanding, the movement from the major street to the right turn is presented in Fig. 2. When a vehicle has free movement from a major to a minor-street or vice-versa, it is considered as an accepted gap but when the vehicle applies brake in the conflict area for a particular time during the movement is considered as a rejected gap. Figure 2 illustrates that when a vehicle is moving from point A to B, without applying any brake in between is termed as the accepted gap. It simply means that the driver has accepted the gap for crossing the manoeuvre. A vehicle with random movement is shown in the figure with a blue line moving from a major street (1) to a minor street (3) with a right turning, the time required to travel from A to B is the accepted gap, and time required to travel from A to C is occupancy time. Whereas, the green shaded area is the conflict area of the intersection.

In the present study, to understand the effect of gap acceptance behaviour of the driver at an uncontrolled intersection for right turning movement, the data size of 60, 100, and 86 observations for two-wheelers are considered for L1, L2, and L3,

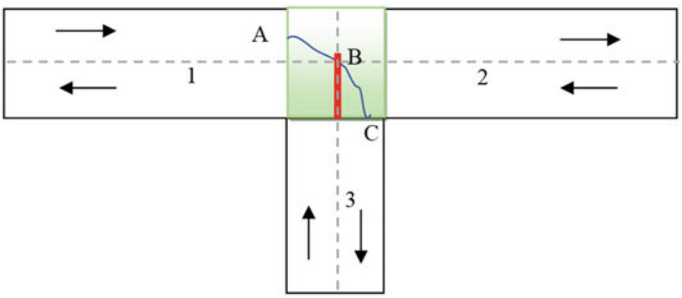

Fig. 2 Measurement of an accepted and rejected gap

respectively. It depends on the traffic flow for one hour, as in the case of L2 and the proportion of vehicles was more as compared to location 1. The descriptive statistic for the values of the accepted gap and the rejected gap is shown in Table 2.

The minimum value of the accepted gap is 1.09 s and the maximum value is noted to be 8.06 s, whereas the minimum rejected gap is 1.85 s and the maximum is 9.54 s. The obtained value of Pearson's coefficient of skewness (CoS) [16] for the data on the accepted and rejected gap for all three locations is designated. For the accepted gap, the CoS for L1, L2, and L3 are 0.724, 0.185, and 0.277, respectively. For the rejected gap, the CoS for L1, L2, and L3 is 0.349, 1.392, and 0.485, respectively. These CoS indicate that the empirical distribution is positively skewed having tailed at right in all six cases. It indicates that the value of the accepted gap is more towards the positive side which results in enhancing the prediction accuracy of the critical gap using distributions. In this section, simple statistical distributions are considered to be competing with each other and used for fitting time-to-event data. They are called the Weibull, Log-normal, Chen, Generalized Exponential, Generalized Lindley, Gamma, Burr, Gompertz, Power Lindley, and Lomax distributions. Each distribution is indexed by two parameters and is pertinent to use in practice. There is a lack of research in the existing literature related to the best-suited statistical distribution for the data sets using the goodness-of-fit criterion. First, fit accepted and rejected gap times individually by all ten distributions. Later, the estimation of the critical gap value using accepted and rejected gap values was combined in the MLE procedure.

Table 3 shows AIC values to understand how the model fits the data sets without over or underfitting it. The AIC value models that achieve high goodness of fit and deal severely with them become complex. AIC score is of much use when compared the score with the competing model. The lower the AIC score the better the model is suitable and this shows the balance between its ability to fit the data. Therefore, as per the least value (deviation) of AIC the accepted and rejected gap defines the driver's realistic behaviour on the field. For the accepted gap, the lognormal and generalized exponential is the best-fitted distribution for all three locations. Whereas, rejected gap follows the gamma and burr for location 1; Weibull and Burr for location 2 and location 3, respectively. The distribution is substantiated by determining the

Table 2 Statistics for accepted and rejected gap (sec) for two-wheelers

Parameters	Accepted gap			Rejected gap		
	L1	L2	L3	L1	L2	L3
Minimum	1.12	1.09	1.46	1.85	1.85	2.23
Maximum	5.90	8.06	7.72	8.56	9.16	9.54
Range	4.78	6.97	6.26	6.71	7.31	7.31
Mean	2.55	2.94	3.89	4.68	5.99	5.77
Median	2.27	2.85	3.71	4.46	5.15	5.45
Std. Dev	1.16	1.46	1.95	1.89	1.81	1.98

goodness of fit. The least AIC value is considered for the distribution fit as very little variation in the value of log-normal and G-exponential for an accepted gap for all three locations. The Kolmogorov–Smirnov (K-S) test and Anderson–Darling (A-D) test values are particularized to validate the best fit for the gap acceptance parameter in Table 4.

In Table 4, validation is done for the four distributions having minor variations in the AIC values for the accepted gap. Using easy-fit software, distribution fitting is executed which as a result obtains the p-value of the two tests named the K-S test and the Anderson–Darling test. This p-value assists in identifying the appropriate distribution whichever has a lesser p-value. As per the K-S test and Anderson–Darling test, the least p-value is obtained for the log-normal distribution as shown in Table 6. This distribution shows the realistic gap acceptance behavior of driver on the field which enable to attainment of the specific value of the critical gap. The goodness-of-fit result verified that the log-normal is best suitable for the accepted gap which will be further used to estimate the value of the critical gap using MLE.

4.2 Different Statistical Distribution for Estimation of Critical Gap

By different distribution, this paper tends to understand the realistic behaviour of drivers which is inconsistent depending on several parameters like vehicle type, traffic and geometrical characteristics, driver behaviour, speed, etc. The MLE is used as a statistical method of estimating the parameters of probability distribution by maximizing the likelihood function so that in the statistical model the observed data depicts the risk-compelling behavior of the driver on the field which enables to predict the proximate value of the critical gap.

A driver's behaviour in a particular location is different and the gap value also varies based on the geometrical features of the intersection. To observe the difference, the accepted and rejected gap of the two-wheeler is considered in this paper for three intersections having similar road features (uncontrolled T-intersection). The different statistical distribution functions are applied using R-software by the MLE method. All ten models' cumulative distribution functions are plotted for three locations to select the best fit out of all. The models are selected based on their suitability for two-parameter estimation. The input parameters are accepted and rejected gaps for two-wheelers. The cumulative distribution function of the parameters is represented in Fig. 3 for L1, L2, and L3, respectively. It can be seen from the graph that the value of the critical gap (shown in blue colour) lies between the accepted and rejected gap.

In Fig. 3a, for location 1, the MLE method is applied for ten likely distributions to get the best fit and obtain the precise value of the critical gap. Weibull distribution, the deviation for the accepted gap is between 2 and 4 s whereas for the rejected gap deviation is observed in the range of 3−8 s. In a similar way when all the curves are observed with the actual values and deviation from the distribution curve the

Table 3 AIC for statistical distributions for accepted and rejected gap

	Weibull	Gamma	Log-Normal	G-Exponential	Burr	Chen	Gompertz	P-Lindley	G-Lindley	Lomax
Location 1										
Accepted Gap	85.259	79.650	77.038	77.155	82.549	95.379	95.279	84.751	77.700	112.244
Rejected Gap	116.840	116.403	117.121	117.000	116.491	120.448	120.690	116.897	116.859	148.611
Location 2										
Accepted Gap	158.692	148.919	143.982	144.670	155.374	177.341	176.478	157.513	145.740	197.226
Rejected Gap	194.680	196.081	198.729	198.534	195.250	199.226	199.840	195.359	197.814	256.888
Location 3										
Accepted Gap	160.779	156.802	154.603	155.446	159.695	170.103	169.700	160.633	156.245	193.017
Rejected Gap	174.718	176.146	178.482	178.573	175.429	177.982	178.522	175.780	177.874	232.771

Table 4 Goodness-of-fit for statistical distributions of accepted gap

		Log-Normal	Gamma	Weibull	G-Exponential
L-1	K-S Test	0.0960	0.1173	0.1207	0.1608
	A-D Test	0.2718	0.5640	1.1236	1.7916
L-2	K-S Test	0.1014	0.1168	0.1424	0.1699
	A-D Test	0.6343	0.8012	1.5484	2.2479
L-3	K-S Test	0.1215	0.1216	0.1346	0.1943
	A-D Test	0.6981	0.7111	1.1867	1.1425

minimum deviation is observed for the lognormal distribution for the accepted gap. However, the maximum deviation from the observed curve from the actual value for the accepted and rejected gap is perceived in the Lomax distribution. The deviation in the values in the graph is statistically represented using the AIC value in Table 5 to get the best fit for the value of the critical gap.

In Fig. 3b, for location 2, the deviation of the curve for Chen, Gompertz, Lomax and p-Lindley distribution is higher as compared to the other distribution. In the Weibull distribution, the deviation is observed for the accepted gap in the range of 3–4 s whereas no observable deviation in the case of a rejected gap. Similarly, for G-exponential and lognormal distribution, the difference in the observed value and curve varies slightly which is estimated using the AIC value of the critical gap depicted in Table 5. The value of AIC is 81.844 and 81.988 for G- exponential and lognormal distribution. The exponential distribution is based on the calculation of the product of reliability and cannot be used to assume future probabilities from the records. Therefore, the lognormal distribution will be considered for hazard function analysis in further research.

In Fig. 3c, for location 3, the deviation of the curve for accepted and rejected gap from the observed values is depicted for all the ten distributions in which Weibull distribution shows deviation ranging between 2 and 4 s in the accepted gap whereas for rejected gap less variation is observed in the range of 4−6 s. In the case of gamma, lognormal, and burr distribution slight deviation for the accepted and the rejected gap is observed whose statistical measure is depicted using AIC value for the critical gap. The AIC value for lognormal and gamma distribution is 125.440 and 125.879, respectively. Although gamma and lognormal have similar curves when its log value is considered the gamma has a heavier tail on the negative side and is negatively skewed which results in less value on the left side of the curve. But in the field, values of the accepted gap range maximum proportion in 1.09−5.9 s (Table 2), and if the curve is negatively skewed the value will be more on the right side, i.e. on the higher range which will vary the result. Therefore, log normal will be considered to get the precise value of the critical gap. For the Lomax distribution, the deviation is slightly higher for both accepted and rejected gaps, hence this distribution is neglected and cannot be considered for further research or analysis.

As per the least AIC values of the critical gap, the best-fitted model is identified for three locations. In general, the best fit is either given by Generalized Exponential

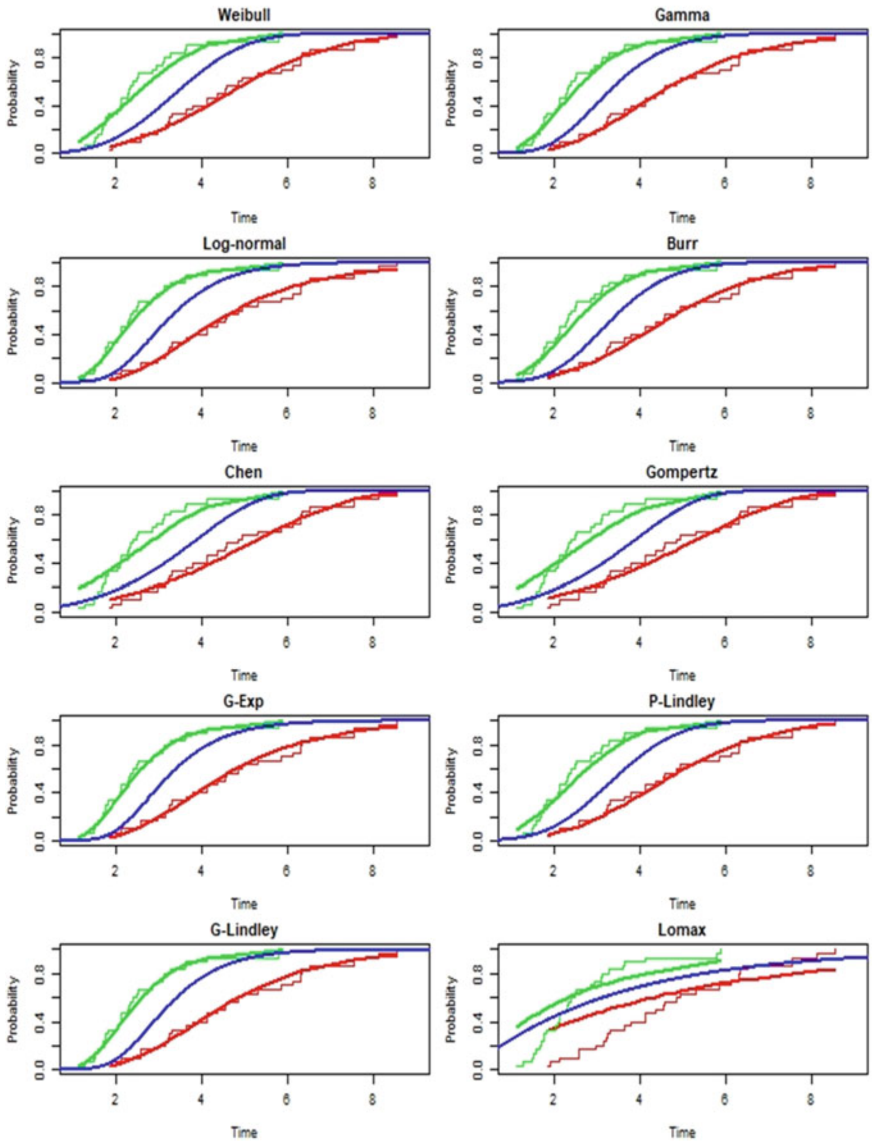

Fig. 3 Maximum likelihood estimates cumulative distribution function different locations **a** L–1 **b** L–2 **c** L–3

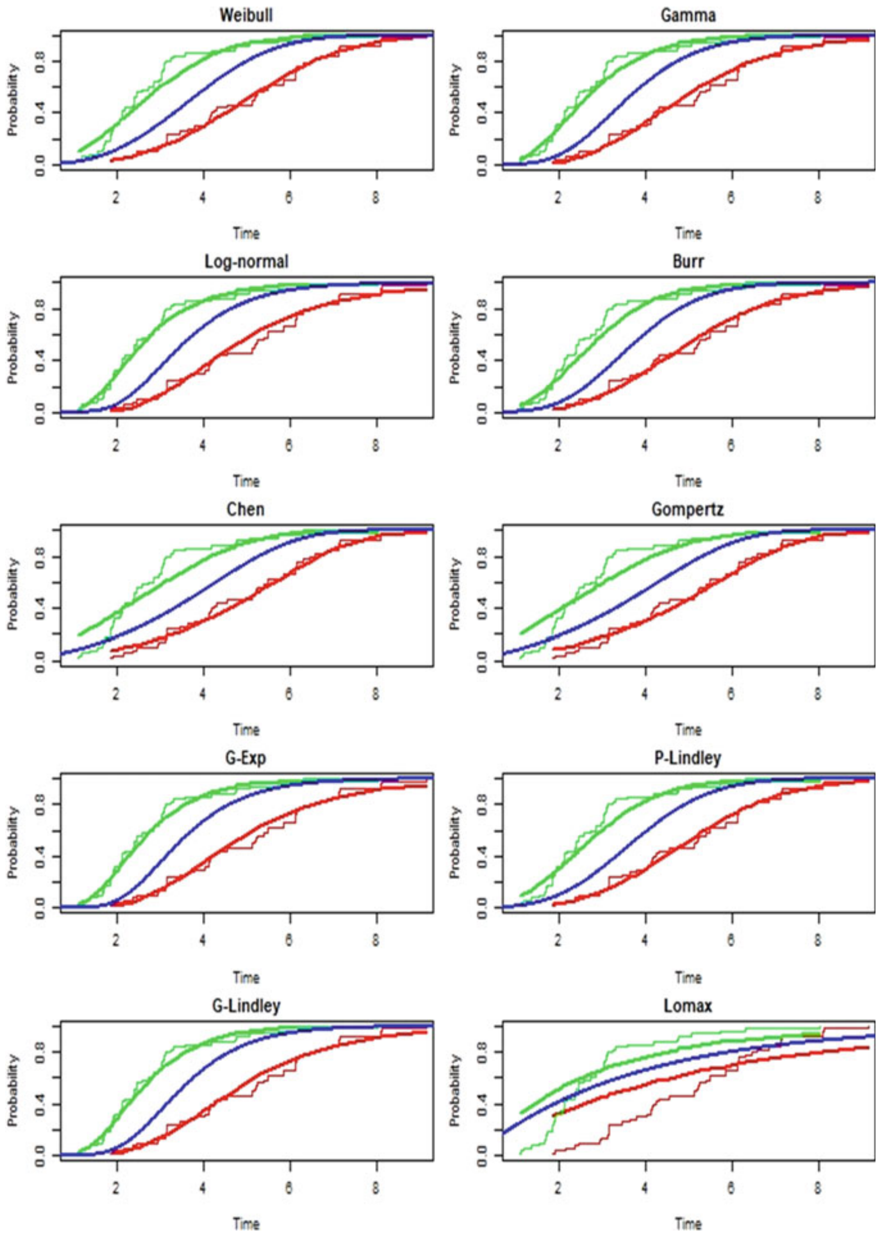

Fig. 3 (continued)

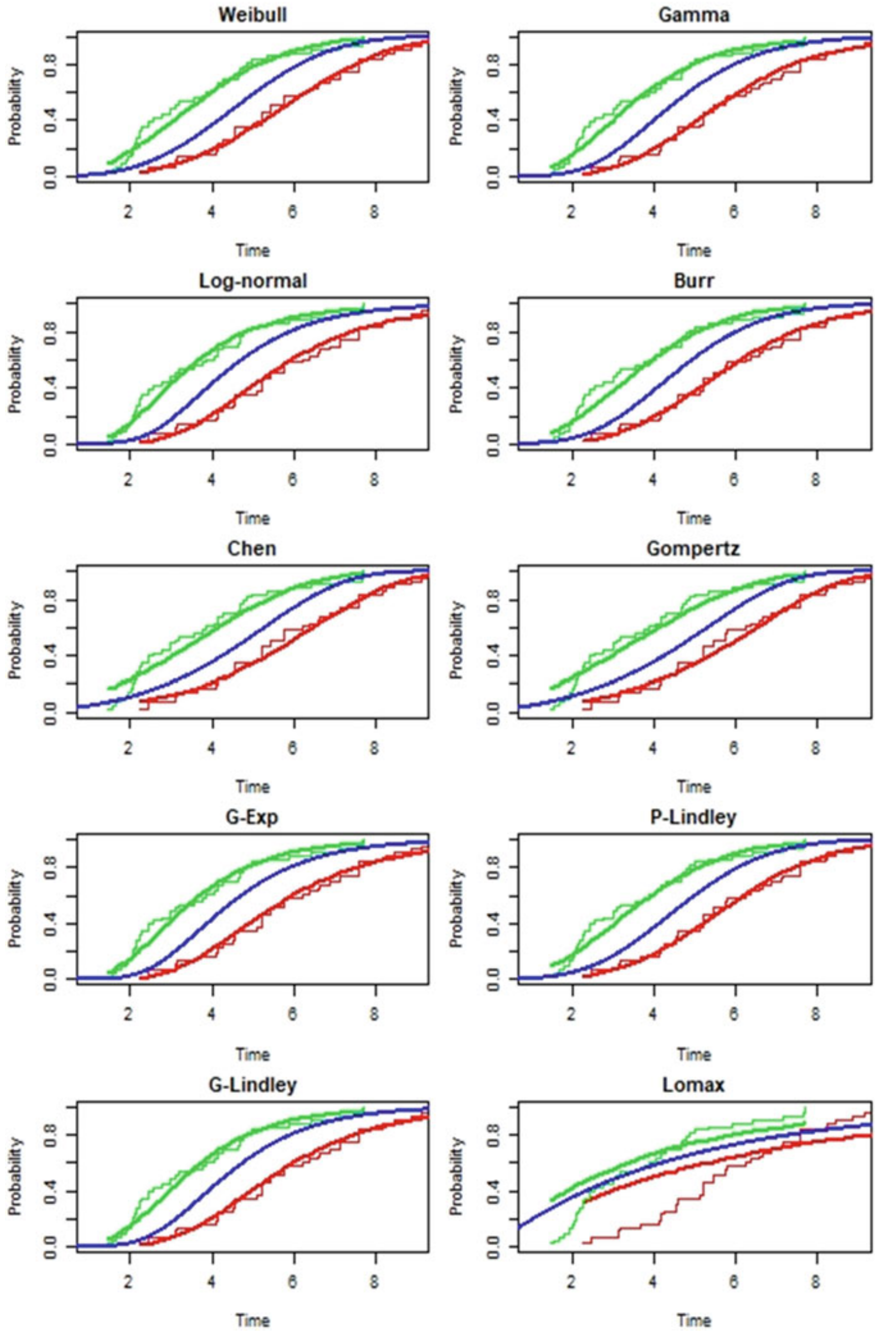

Fig. 3 (continued)

Table 5 Estimated AIC, mean, median, and standard deviation for the distribution for all three locations

	Model	Weibull	Gamma	Log-Normal	G-Exponential	Burr	Chen	Gompertz	P-Lindley	G-Lindley	Lomax
L1	AIC	85.854	82.755	81.948	81.844	83.468	91.826	92.149	85.148	81.984	120.962
	Mean	3.409	3.356	3.333	3.320	3.372	3.297	3.406	3.394	3.324	3.433
	Median	3.382	3.230	3.147	3.120	3.279	3.522	3.532	3.348	3.139	2.380
	Std Dev	1.227	1.138	1.166	1.163	1.113	1.228	1.431	1.189	1.145	3.436
L2	AIC	135.981	129.304	127.301	127.107	131.253	148.615	149.713	134.090	127.461	189.412
	Mean	3.756	3.705	3.665	3.651	3.731	3.501	3.730	3.741	3.654	3.710
	Median	3.697	3.544	3.439	3.418	3.623	3.819	3.809	3.670	3.443	2.573
	Std Dev	1.464	1.326	1.353	1.333	1.298	1.341	1.754	1.405	1.311	3.719
L3	AIC	127.057	125.440	125.879	125.936	125.988	133.053	133.666	126.567	126.790	175.293
	Mean	4.654	4.611	4.601	4.603	4.631	4.160	4.629	4.642	4.607	4.577
	Median	4.607	4.403	4.279	4.279	4.488	4.787	4.798	4.550	4.314	3.174
	Std Dev	1.724	1.714	1.825	1.839	1.677	1.424	1.938	1.700	1.801	4.585

or Lognormal for locations 1 and 2. The mean value for the critical gap for the best-fit model is 3.32, and 3.33 s for L-1. And the mean value for the critical gap estimated is 3.65 and 3.66 s for L-3. For location 3, the best-fit models are Gamma and lognormal distributions and the value of the critical gap is 4.60 and 4.61 s, respectively.

The value of the critical gap varies marginally but for the safety analysis, a few seconds are also precise to reduce the severity of accidents. A minor error in the estimated value may lead to a severe crash. Hence, the best distribution is required for evaluating the gap acceptance parameter for each location. The log-normal distribution is best suited for all three locations and will be considered for further analysis of risk and severity. It is well-defined that the value of the critical gap is independent of conflicting traffic volume [9, 10] nevertheless this proved incorrect; traffic volume and speed indirectly affect the value of the critical gap.

5 Results and Discussion

The critical gap has been estimated using MLE with lognormal distribution and selected on the basis of AIC value as it is one of the suitable statistical analyses in order to choose the best-fit model. Hence the least AIC value is considered to be the quality model as it loses fewer input data which minimizes the error. As the estimated value of the critical gap is based on the input parameter of the accepted and rejected gap, therefore for further check, the goodness-of-fit for the gap acceptance parameter is applied for validation. Five out of ten distribution functions are found to be the most suitable model. To be more precise, the different models are used to select the best-fit model out of five by comparative analysis and therefore, two statistical tests were performed, i.e. K-S test and Anderson–Darling test for further research. Out of which log-normal shows the best suit for each location for heterogeneous traffic conditions of rural highways.

The critical gap estimation using Indo-HCM cannot be used for the further analysis of the safety and severity prediction as it considers parameters of the traffic and geometric features. Whereas for the implication to safety, the other parameters like vehicle type, conflicting speed, gap acceptance behavior of driver are required for accurate prediction of safety at the intersection. Even various statistical techniques are used for evaluating the gap acceptance parameter value which depends only on the driver behavior rather than the other parameters like conflicting vehicle, speed, gap acceptance behavior of driver with vehicle type.

The distribution enables us to know the realistic behavior of the driver at the intersection. But it needs to be specific when it is been analyzed for risk and severity prediction, the precise value of critical gap in such kind of analyses add a great value for safety at intersection depending on different geometric and traffic characteristics. A suitable distribution for the gap acceptance behavior enables to obtain the precise model to reduce the severity of accidents at an uncontrolled intersection. Hence, the MLE technique is found to be more suitable as discussed in the previous sections and it can be used to get the critical gap and use it for risk and severity analysis for uncontrolled intersection.

6 Conclusion

Based on the past literature, different statistical techniques are developed for the estimation of critical gaps to estimate the capacity and LOS of the intersection. However the researchers have lacked their emphasis on the implication of critical gap to safety which is a point of major concern at unsignallized intersections. Hence estimating the precise value of the critical gap is important Hence, the maximum likelihood estimate with the lognormal distribution provides the best estimates of the critical gap parameters. The use of appropriate distribution will enable to prediction of the suitable value and can be used for further analysis of safety prediction. This paper reveals that taking different distributions for estimating the value of critical gaps is feasible depending upon the distribution fit for gap acceptance and rejection. The value of the critical gap may vary with smaller variation and it is important to identify the best distribution among all. Hence, AIC technique was selected to identify the best fitting model by analyzing the gap acceptance parameter i.e. accepted and rejected gap which is difficult to analyze by any other statistical analysis. Therefore, it is recommended to consider various probability models and choose the best-fitting model using the AIC. Hence, it can be enlightened from the results obtained that the driver behavior is adequate to estimate the value of the critical gap. The impact of the proportion of heavy vehicles and traffic volume indirectly impact the value obtained from the statistical technique of MLE. Some of the conclusions are drawn based on the analysis:

- The accepted gap rejected gap, and critical gap follows different distributions based on the geometric and traffic characteristics of the section. But lognormal is the best-suited distribution for three-legged intersections under heterogeneous traffic conditions for rural highways.
- The critical gap obtained by the maximum likelihood estimate using different distributions is similar to the values attained from the Indo-HCM 2017. It demonstrates that the critical gap value estimated using the driver's gap acceptance behavior is similar to the value calculated using Indo-HCM. Hence, the gap acceptance behavior is influenced by the traffic volume and proportion of heavy vehicles which results in the estimating the similar value when different parameters and technique is used.
- As the critical gap at the uncontrolled intersection indicates the gap acceptance behavior of a driver, the precise value needs to be obtained using traffic, geometric, and gap acceptance parameters which can be used for further analysis of risk and severity of the driver at uncontrolled intersection.

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Identification of Infrastructural Causative Factors for Road Accidents on Urban Arterial Roads: A Case Study of Ahmedabad

Poojan Pasawala and Bhavin Shah

Abstract Every year approximately a 1.35million people lose their lives in a road accident, which is the world's eighth leading cause of death. There are 1.51 lakh fatalities per year in India because of road accidents, making India first across the world in terms of deaths. Accidents are caused by three factors: humans, vehicles, and infrastructure. Because humans are involved in driving, human error is generally considered to blame. However, most of the time, human error and a combination of other factors are to blame, so it is not only the driver who must accept responsibility for the accident. This research aims to identify the infrastructural element of the urban arterial road of Ahmedabad that can lead to the accident by conducting a Road Safety Audit and establishing its correlation with the past three years' accident data. The research is validated with a closed-ended questionnaire survey. From the results, the stretches are prioritized for corrective measures. The result shows that the factors related to visibility and intersection; pedestrian and cyclist; speed and road cross-section and alignment are correlated with the accident.

Keywords Road safety · Road safety audit · Transportation

1 Introduction

1.1 Background

Transportation is the backbone of the country's development; it provides access and mobility to the public. In the past decade, with the increase in the population and urbanization, combined with the increase in income, no. of vehicles on the road has

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skyrocketed. With the increase in vehicles on the road, road users suffer accidents, pollution, and congestion; among all the three consequences, accident affects road users most negatively.

Road accident is an increasing cause of death worldwide, and it has emerged to be the primary concern in the past decade. Despite all the progress in road safety, the death because of road accidents in the world is around 1.35 million, which is approximately 3700 deaths/day, which is unacceptably high [1]. Also, it is one of the leading causes of disability; approximately 20–50 million people suffer from road accidents [2]. Road accidents have increased from the 16th to the 8th cause of mortality globally between 1990 and 2016, and 54% of those fatalities are either pedestrians, cyclists, or motorcyclists. [1]. According to WHO, it is the leading cause of death for people in the 5–29 age group [1]. According to the estimates, road accidents cost the country approximately 3% of its GDP, affecting the low- and middle-income group countries to a large extent as they contribute to 93% of road accidents, with only 60% of the road vehicles in the world [3]. With roughly 17.5% of the global population, India ranks third globally for the most accidents per year. As not all incidents are reported to the police, it is believed that the number is understated by a factor of 5 to 20 times. [4]. From 1970 to 2019, the number of fatalities due to road accidents increased 12 times in India. Due to road accidents in 2019, there were 1.51 lakh fatalities per year in India, which makes it first across the world in terms of death due to road accidents, constituting 11% of the deaths worldwide due to road accidents. Because of road accidents, India is losing its people and negatively affecting the economy; the accidents account for 0.77% of the nation's GDP. [5].

1.2 Need for Study

Three factors contribute to road accidents: human error, vehicle error, and road infrastructure error. Even though accidents can be a combination of more than one error, the causation is generally pointed to only one factor–human error. This leads to an overrepresentation of accidents because of human error. There is repeated wisdom, especially for Ahmedabad city, that "Driver causes 90% of the accidents" [6], even though infrastructure-related and mechanical problems are inseparable parts of the accidents. Also, the Penal section under which police register a complaint is related to road users; hence, they will only see the driver's error in most cases. Hence it becomes necessary to identify the elements that can probably cause the accidents and to prioritize the stretches of road for corrective measures (Fig. 1).

1.3 Scope of Study

The study will be limited to the Ahmedabad area and to the Arterial roads of Ahmedabad.

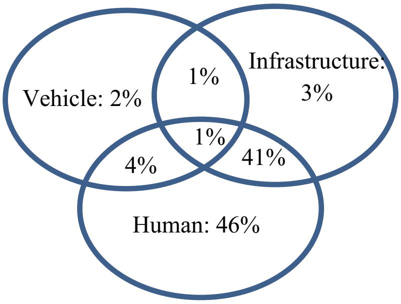

Fig. 1 Responsible factor for road accidents [6]

The scope will be limited to identification, correlation and to prioritize the road section and will not include the remedial measure for the identified stretch.

2 Literature Review

According to a review of other research on the effect of geometric design on road safety, shorter or no sight distance and small curve radius significantly impact road safety and can significantly increase the accident rate. The reviewer also pointed to the fact that accident is a factor for many reasons, and in the geometric design also there is a significant effect of one element on the other element [7].

A study conducted in China with the help of the Bayesian network studied the combination of the factors that can affect road accidents. The results show that bad weather (which causes slippery roads) and speeding can be critical factors while single-vehicle traffic is considered. When the multi vehicular traffic is considered, then the night-time with the road not physically separated, poor braking, and the use of the wrong lane can be critical [8].

A road safety audit (RSA) was conducted on NH-58 for a 65 km stretch; with the help of the multiple-regression model, the author tried to establish the safety influencing parameter, which, if corrected, can decrease the likelihood of the accident rate. They found that the traffic volume, road marking, condition of the shoulder, cross drainage, and other warning signs significantly affect road safety [9]. Also, a similar study was conducted on the NH-4 highway with the help of a multiple linear regression model; they found that the horizontal curvature and the number of the junction on the road are related to the accident on the highway, whereas the sight distance and vertical curvature are not or rarely related to the accident. Additionally, there is a negative correlation between the number of accidents and road width and roughness. [10]. An analysis of a 360 km highway in Iran using the analytical hierarchy process concluded that the main factors affecting road safety on the selected stretch are high speeds and inappropriate or non-standard horizontal curves [11].

Ahmedabad's urban arterial roads are the subject of the study. Various research articles were reviewed to determine the road to be used and the appropriate time to conduct the road safety audit.

In Ahmedabad, accidents happen highest in peak hours, from 9 to 11 am and from 6 to 8 pm [12, 13]. In Ahmedabad, the number of accident cases has increased on urban highways like the S.G. highway, 132ft ring road, and the Narol-Naroda Highway, finding of the study states that almost 85% of a fatal accidents are found along that stretch. In Ahmedabad, the eastern side of the city has more share of the accident than other parts of the city [13]. Many low-income and middle-income households are located in this area whose primary mode of transport is either 2-wheeler or walking, and research shows that the 2-wheeler and pedestrian account for 74% of the total fatalities in Ahmedabad. [13].

3 Methodology

Firstly, the need for the study was identified, and the existing literature was reviewed to understand the different methods available for a road safety audit, road classification and their characteristics, and other factors related to the accident which were specific to Ahmedabad city, were also studied.

In the next step, primary data was collected by conducting the road safety audit, questionnaire survey, and secondary data–the past three years' accidents recorded by the police, was collected from Ahmedabad police.

The next step is data analysis, in which Pearson's correlation was established, and the result was validated with the expert's opinion collected from the questionnaire survey. The road section was prioritized from the response to the questionnaire survey with the help of the Relative Importance Index.

4 Data Collection and Data Analysis

4.1 Selection of the Road

From the Indian Road Congress IRC:86, 2018, and MoHUA guidelines, the criteria for the selection of the road were formed. The road section selected for the study is Chandranagar BRTS–Kankaria–Jashodanagar Cross road–Naroda Patiya; the entire road section is 17 km long (Table 1 and Fig. 2)

Table 1 Characteristics of the selected road

Head	Parameter	Characteristics of selected stretch
Classification of the road	Arterial road	The maximum stretch is Arterial Road
Dominant user	2-wheeler, cyclist and pedestrian, and car	41% cycle, 35% 2-wheeler, 22% other, and 2% car
Existing public transport	AMTS, BRTS, preferably railway station	All 3 modes are there in the selected stretch
Land use in proximity	Mixed–use–Industrial, commercial, and residential	Industrial and residential dominant, commercial to a certain extent
Dominant income group	EWS, LIG, MIG	42% LIG, 27%EWS, 22% MIG, 9% HIG
Width of the road	The majority of the road shall be between 45 and 60 m but can be relaxed till 36-60 m	60mtr from Jashodanagar crossroad to Naroda Patiya 36 m for the majority of the section from Chandranagar to Jashodanagar crossroad
Lanes	Shall be 4 or 6 lanes (Total of both carriageways)	6 on the Narol-Naroda highway & 4 on the remaining stretch
Construction	Shall be free from ongoing major construction	
Length of the road	> 12 km	Total: 17.0 km
Actual to design PCU ratio	> 0.9	Vasna bridge = 5585/6000 = 0.93 Krishnanagar (Naroda) = 5531/6000 = 0.92

4.2 Checklist for the Road Safety Audit

The road safety audit checklist is prepared with reference from Indian standards IRC: SP-88: 2010, guidelines of MoHUA [14], and international standard Austroad: AGRS06-09. The road safety checklist was further divided into the following heads, and the observations were recorded in the following heads:

(1) Visibility and Intersection
(2) Pedestrian and Cyclist
(3) Speed limit
(4) Sign, pavement, and delineation
(5) Cross-section and alignment
(6) Road-side hazard
(7) Light and Night-time issues
(8) Miscellaneous.

Fig. 2 Selected road section

4.3 Observation from Road Safety Audit

For the road safety audit, the road was divided into equal sections of 500 m, except Sect. 1 of 1 km, as there is a bridge in Sect. 1, so there were 33 sections on the selected road. For RSA, the road section was divided into 500 m stretches considering the spacing of major intersections on the selected stretch and BRTS stands, as the pedestrian will be crossing the street more frequently near the BRTS stand. If the road sections are divided into shorter segments, some stretched will have only mid-blocks, and if divided into longer segments, more than two major intersections are coming, leading to less accurate RSA data. The RSA were conducted two times a day for a particular stretch; during morning peak hours and evening peak hours, which are from 9 to 11 am and from 6 to 8 pm, as identified in the literature review. Also, the audits were conducted on random weekdays and once on Sundays. In order to evaluate road user safety without bias based on the type of vehicle, audits were done on a two-wheeler without gear and on a bicycle. The checklist was used to record the observations, and issues were labeled as Yes, No, or NA. A score was assigned to checklist observation to convert qualitative data to quantitative data [15]; a score of "1" was assigned for "Yes," and a score of "0" for "No." If the answer to a direct question, such as "Is the pedestrian facility available?" is yes, the score is "1," and if the answer to an indirect question, such as "Is road having dangerous pot-holes?" is yes, the score is "0," as it can cause an accident, so a "0" score is given for the element that can cause an accident. If a question does not apply to a section, it is marked as "NA," and no score is assigned to it; for example, if a flyover is not present, it is marked as "NA". If there are 5 questions in the head "Visibility and Intersection", and 2 of them have scored "1", other 2 have scored "0," and 1 is "NA", then to convert to the score of "10":

$$= \frac{\text{Sum of all the element with score 1}}{\text{Total no.of element} - \text{No. of an element with"NA"}}\text{x}10$$

Hence, the head “Visibility and intersection” score will be 5 out of 10. The higher the score, the better will be the road section in terms of safety and the lesser probability of accidents happening on that road.

4.4 *Questionnaire*

The questionnaire survey was conducted to validate the result, which will be obtained from the road safety audit. The questionnaire is totally based on the road safety audit checklist questions. In the questionnaire, the Likert scale with five responses was used to quantitatively get the likelihood of the accident different elements of the road can cause. 0 being accident less likely and 5 being accident more likely.

The sampling method that is used is the stratified sampling method. The probable respondent should have worked in areas of transport engineering and preferably worked in the area of road safety or academicians who teach transportation engineering, or the traffic police, who are involved in accident investigations, but to eliminate the biases, the police are eliminated as we have seen in the literature review that they point towards the road user as the primary cause of the accident.

So, for the questionnaire, the respondents selected are academicians or transport engineers working in the industry (professionals). The number of respondents was chosen according to the Table 2.

Set 1 is selected for the questionnaire survey; 20 responses will be required. After the determination of the sample size of each stratum, further survey will be done

Table 2 Determination of sample size

Head	Formula	Symbol	Variable	Set-1
Determination of sample size ‘n’	$n = p(1\text{-}p)(z/e)^2$ [16]	z	Confidence level (Value obtained from Z-value table)	1.96
		e	Margin of error	5%
		p	Unknown population proportion (Taken so that the value of P(1-P) becomes maximum)	0.5
		n	Desired sample size	384.1
Stratified sampling	5% of n	N		~ 20
	Academician (50%)	S1		10
	Professional (50%)	S2		10

with the help of judgmental sampling. The questionnaire was floated to the academicians with master/Ph.D. in transportation engineering and the professionals with any experience in the industry having master/Ph.D. in transportation engineering. The name and the institute where the respondent works are kept confidential to maintain the anonymity of the respondents.

Based on the profile of the respondents, 40% of the professionals were of 0–5 years of experience, which primarily includes working professionals, 20% of the respondents are of 5–10 years of experience, 25% of the respondents are of 10–15 years of experience and remaining 15% of the respondents are of 20–25 years of the experience.

The questionnaire responses were recorded on a scale of 5; for analysis, the score of 5 was converted to a score of 10 using a weighted average.

$$\text{Formula : W} = \frac{\sum_{i=1}^{5} W_i x N_i}{\sum N};$$

wnere,

N_i = number of responses for W_i; N = Total responses; = W_i Assigned score. (From 1 to 5); W = Weighted average.

4.5 Crash Data

After the road safety audit for the selected stretch was completed, secondary data on crashes in the previous three years (2018–2020) was collected from the police station, and the accident spots were marked on the map using the longitude and latitude registered in the police data. Then the number of accidents was counted for the 500 m road section for which the road safety audit was conducted. The accident on the selected road stretch for the past 3 years for the different sections of 500 m is (Table 3).

4.6 Data Analysis

For the results, the value of Pearson's correlation coefficient R can be between –1 and 1, in which + 1 signifies the positive correlation and –1 signifies the negative correlation. For the analysis of the collected data and to establish the correlation, the results were obtained from a scale of 10, in which the score of 0 means the section of the selected road is more prone to accidents and 10 means the road is less prone of the accident, for the analysis purpose the scale is reversed, which means 0 means the section will be less prone to the accident and the 10 will be more prone to accident. The following step is done to avoid confusion as when we do correlation on the same

Table 3 No. of accident on the selected road

Section	No. of Accident		Section	No. of Accident
Section 1	2		Section 18	7
Section 2	3		Section 19	6
Section 3	2		Section 20	7
Section 4	3		Section 21	5
Section 5	3		Section 22	3
Section 6	1		Section 23	1
Section 7	2		Section 24	1
Section 8	1		Section 25	0
Section 9	4		Section 26	6
Section 10	3		Section 27	1
Section 11	2		Section 28	1
Section 12	3		Section 29	2
Section 13	1		Section 30	0
Section 14	3		Section 31	0
Section 15	3		Section 32	0
Section 16	4		Section 33	0
Section 17	6			

observation without inverting the scale, and if we get -1 as the correlation coefficient will mean that with the decrease in the section's score, there will be an increase in the accidents, and to avoid such confusion the scale was reversed.

The weights were assigned to every observation of the road sections:

$$\text{Weight for the safety audits}: \frac{1}{No.of\ heads} = \frac{1}{8} = 0.125$$

$$\text{Weight for the expert opinion}: \frac{Score\ of\ the\ 1\ head}{Sum\ of\ score\ of\ all\ head}$$

Here, Head = group of similar elements of the road, for example, visibility and intersection, speed limit, and others, as mentioned in Sect. 4.2 and (Table 4).

Hence, the final score of all the heads for all the sections = score of the head x Weight of the head.

$$\text{The formula for correlation}: \mathrm{r} = \frac{\sum(x_i - \overline{x})(y_i - \overline{y})}{\sqrt{\sum(x_i - \overline{x})^2(y_i - \overline{y})^2}}$$

where,

r = correlation coefficient.

Table 4 Weight for calculating the weighted score

Issue	Weight from Audit	Weight from a Questionnaire survey
Visibility and Intersection	0.12	0.13
Pedestrian and cyclist	0.12	0.12
Speed limit	0.12	0.12
Sign, pavement marking, and delineation	0.12	0.11
Cross-section and alignment	0.12	0.11
Road-side hazard	0.12	0.14
Light and Night-time issue	0.12	0.13
Miscellaneous	0.12	0.13

Table 5 Correlation matrix–Road safety audit & accidents

Heads (Refer 4.2)	1	2	3	4	5	6	7	8	Acci-dent
1	1.0								
2	0.5	1.0							
3	0.2	0.3	1.0						
4	0.2	0.5	0.2	1.0					
5	0.1	-0.2	0.1	-0.3	1.0				
6	0.0	0.3	0.2	0.2	-0.3	1.0			
7	-0.1	0.1	0.1	0.2	-0.2	0.4	1.0		
8	0.0	0.2	0.1	0.4	-0.2	0.5	0.3	1.0	
Accident	**0.5**	**0.4**	**0.3**	0.0	**0.3**	0.00	0.0	0.0	1.0

x_i & $y_i =$ Value of variable X & Y; for example, if correlation must be found between Accident and Visibility and Intersection head, then the X & Y will be the weighted score of the respective variable; $\overline{x}$, $\overline{y}$ = Mean of the value of X & Y variable (Tables 5 and 6).

Now, with the help of the strongly and moderately correlated factors, prioritization of the stretches was done with the help of the Relative Importance Index:

Formula: $\sum$ (score of the head questionnaire x Weight of the head) for the strongly or moderately related element of the road (Table 7).

5 Results and Discussions

In the data analysis section, the data collected from the road safety audit which is conducted by the researcher and the data from the questionnaire survey, in which the respondent was experts in the field of transport engineering was done (Refer Tables 6

Table 6 Correlation matrix–A questionnaire survey & accidents

Heads (Refer 4.2)	1	2	3	4	5	6	7	8	Accident
1	1.0								
2	0.5	1.0							
3	0.2	0.3	1.0						
4	0.2	0.5	0.2	1.0					
5	0.1	–0.2	0.1	–0.3	1.0				
6	0.0	0.3	0.2	0.2	–0.3	1.0			
7	–0.1	0.1	0.1	0.2	–0.2	0.4	1.0		
8	0.0	0.2	0.1	0.4	–0.2	0.5	0.3	1.0	
Accident	0.5	0.4	0.3	0.0	0.3	0.00	0.0	0.0	1.0

Table 7 Prioritization of the road section from the co-related element of the road

Rank	Section	Priority to be given	Rank	Section	Priority to be given
1	**Section-15**	3.71	**18**	**Section-24**	2.09
2	**Section-18**	3.64	**19**	**Section-29**	2.08
3	**Section-9**	3.39	**20**	**Section-26**	1.89
4	**Section-19**	2.88	**21**	**Section-22**	1.83
5	**Section-17**	2.86	**22**	**Section-23**	1.83
6	**Section-14**	2.81	**23**	**Section-5**	1.82
7	**Section-7**	2.72	**24**	**Section-11**	1.82
8	**Section-2**	2.70	**25**	**Section-10**	1.67
9	**Section-3**	2.50	**26**	**Section-28**	1.53
10	**Section-16**	2.42	**27**	**Section-31**	1.38
11	**Section-20**	2.42	**28**	**Section-21**	1.33
12	**Section-1**	2.42	**29**	**Section-25**	1.21
13	**Section-13**	2.34	**30**	**Section-8**	0.93
14	**Section-6**	2.29	**31**	**Section-30**	0.74
15	**Section-12**	2.24	**32**	**Section-27**	0.55
16	**Section-32**	2.15	**33**	**Section-33**	0.14
17	**Section-4**	2.12			

and 5); the results show that both the results are highly correlated and hence we can say the research has high validity.

From the data analysis Table 5, it can be clearly seen that there is a positive correlation between the different elements of the road, which are related to visibility and intersection; Pedestrian and cyclist; Speed limit, sign pavement marking, and delineation; cross-section, and alignment; Miscellaneous, and Light and night-time hazards are positively correlated with the accidents, whereas road-side hazard shows no relation with the accidents.

There is a positive correlation between the various types of road elements and accidents, but only two are strongly correlated: pedestrian and cyclist and sight distance and intersection. Elements related to speed limit and cross-section and alignment are moderately related; otherwise, no strong relationships can be seen in the other elements, which means that if we focus on the elements related to pedestrians and cyclists; sight distance and intersection; cross-section and alignment; speed limit, there will be a drastic decrease in road accidents caused by road elements, increasing the safety of road users.

From RSA, we can observe that the main factors in the Visibility and Intersection are Inadequate sight distance, U-turn provision on an unsignalized intersection, and unsignalized intersection. The main factor in the pedestrian and cyclist head are not providing facilities like footpaths and cycling tracks and not providing enough crossing points in the midblock. The main factor in speed limit is not providing speed calming measures, and in cross-section and alignment, the main factor is not having a sign before enough distance of flyover so that road users can decide on which road to go.

From Table 7, we can see that Section 15, Section 18, and Section 9 are the first 3 roads to be prioritized, so if we apply the corrective measures on the prioritized road section, then there is the possibility that the accident on that section will be decreased. Hence, there will be an overall decrease in road accidents as these are the sections where most of the accidents occurred in the past. It can also aid the relevant urban authority (here in this case–Municipal Corporation) in prioritizing the road sections for implementation of road safety engineering interventions.

A sign showing a flyover ahead shall be provided before an adequate distance from the flyover so that the driver can make an informed decision beforehand. Speed-calming measure like speed humps has to be provided, along with the proper delimitation and adequate crossing in mid-blocks and near BRTS stands. All the sight distance blocking elements like hoarding, illegal structures shall be removed from the road. Special cycling lanes and pedestrian lanes shall be provided alongside the road. The intersection with heavy traffic flow shall be signalised.

These are the measures to make the roads forgiving and reduce the accidents, but there should also be strict enforcement of the laws on human factors such as overspeeding, using the wrong lane, driving vehicles without proper safety equipment like helmets and seat belts, etc.

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Modelling Longitudinal and Lateral Vehicle Movement Behavior Under Multiple Influencing Vehicles

Dhiraj Kinkar, Madhu Errampalli, Mukti Advani, and Saraswathi Sethi

Abstract In a microscopic traffic analysis, realistic modelling of vehicular movements in mixed traffic conditions is most important to arrive at outputs with adequate accuracy. However, the majority of the studies considered single leader vehicles as influencing vehicles to estimate the following vehicle movement which may not be a realistic behaviour under heterogeneous and non-lane discipline traffic circumstances. Moreover, both longitudinal and lateral movements take place simultaneously in such non-lane discipline situations. Considering these, the present study emphasizes the significance of the influence of other surrounding vehicles on the subject vehicle while modelling its movement and developing longitudinal (car-following) and lateral (lane change) movement models of a motorized vehicle with heterogeneous traffic conditions. For this, vehicular trajectory data was extracted from videography data from nine mid-block road sections. Accordingly, relevant independent variables of surrounding vehicles along with subject vehicle characteristics and road geometry were considered to develop car-following and lane change models for estimation of longitudinal and lateral movements respectively. The developed models have been validated using the observed data which can be utilized for estimating longitudinal and lateral vehicular movements under prevailing mixed traffic conditions with reasonable accuracy.

Keywords Car following · Lane changing · Heterogeneous traffic · Multiple influencing vehicle

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1 Introduction

1.1 General

In India, due to mixed traffic conditions and poor traffic discipline, traffic management becomes a big challenge for the authorities and planners in providing efficient vehicular traffic operations. The presence of prevailing mixed patterns of traffic, lane indiscipline and also high traffic volume on roads needs a good consideration of data and its interpretation along with appropriate traffic flow analysis. Since vehicles of various types are allowed to mix and share the same road space along the length of a carriageway, traffic analysis becomes more complex compared to homogeneous traffic conditions. Even traditional field data in such situations are generally not suitable for studying vehicle interactions with variations of traffic volume and vehicle composition on stream speed and capacity. Researchers have studied alternate methods and techniques for measuring transportation network performance considering various parameters including travel speed, delay, level of service, connectivity, safety, link performances, volume-capacity ratio, etc. Since the past few decades, traffic simulation technique has become popular among traffic and transportation professionals, a large number of microscopic traffic simulation models have been developed worldwide, but very few focus on typical mixed traffic comprising the space sharing based traffic behavior.

In view of this, there is a high need for an appropriate microscopic traffic simulation model that can estimate driver behavior precisely and realistically focusing on mixed traffic conditions. While developing a microscopic traffic simulation model, core inbuilt traffic models like the car-following model and lane-changing model for the urban road network need to consider these mixed traffic conditions and also able to evaluate a wide range of transport policies. However, the majority of the studies considered single leader vehicles as influencing vehicles to estimate the following vehicle movement which may not be able to estimate realistic behaviour under heterogeneous and non-lane discipline traffic circumstances. Moreover, both longitudinal and lateral movements take place simultaneously in such non-lane discipline situations. Considering these, the present study proposes to develop longitudinal (car-following) and lateral (lane change) movement models of motorized vehicles with heterogeneous traffic conditions under the influence of other surrounding vehicles on the subject vehicle. Accordingly, relevant independent variables of surrounding vehicles along with subject vehicle characteristics and road geometry have been considered to develop car-following and lane change models for estimation of longitudinal and lateral movements respectively. The developed models were further validated using the observed data in order to utilize for estimating longitudinal and lateral vehicular movements under prevailing mixed traffic conditions with better accuracy.

1.2 Longitudinal and Lateral Movements

Gipps [1] developed a framework to model lane-change decisions in urban driving conditions, however, presupposes that a lane change manoeuver will only happen when it is safe, that is, when there is sufficient space in the target lane from the leader vehicle. Under heterogeneous and non-lane discipline traffic circumstances, estimation of vehicular movements depends not on a single leader vehicle, but surrounding vehicles that are travelling close to it. So, not only the longitudinal gap, but the vehicle tries to adjust lateral gaps with these surrounding vehicles in order to maintain safety while moving on roads. This would be different for motorized two-wheelers (MTW) as the width of the vehicle is small and tries to squeeze in between adjacent and leading vehicles. Mallikarjuna et al. [3] carried out an analysis of the lateral dispersion of vehicles on 10-m-wide highways and studied the effect of the lateral positions of the leading and following vehicles on the longitudinal space. Relationships between the lateral space and area occupancy are displayed for various vehicle combinations. In addition to vehicle type and speed, it has been discovered that staggering affects the longitudinal spacing that cars maintain. Malikarjuna et al. [2] examined lateral gap-keeping behavior in mixed traffic conditions and found that the lateral distance between the subject vehicle and the adjacent vehicle increases as the speed of the adjacent vehicle increases. Kanagaraj et al. [4] studied the characteristics and factors relating to the longitudinal and lateral movement of the vehicles and demonstrated that MTW moves also in the lateral direction.

Asaithambi and Basheer [5] studied car-following behavior in mixed traffic situations under different vehicle classifications and different follower-leader pair types to investigate the vehicle-following behavior. The analysis demonstrates that the size of the leader has a greater influence on the follower's decision to stagger; when the leader's speed is higher, the following vehicle has more freedom to choose a different manoeuver; heavy vehicles as the pursuers are unwilling to change the manoeuver in the shorter time interval. A multinomial linear regressive model was created to account for vehicle following behavior. The macroscopic and microscopic features of mixed traffic on MTW and cars were studied by Wong et al. [7] to determine all obtrusive vehicle movements and derive specific features for various vehicle modes, lanes, and density levels such as lateral positions, lateral gaping, and longitudinal gaping. Raju et al. [6] studied driver behavior in mixed traffic on multi-lane expressways to conclude that smaller vehicles switch leader vehicles more frequently to avoid delays, which leads to shorter following and perception times and more aggressive gap acceptance.

From there, it has been observed that the datasets of most studies are limited to one or two locations and models were developed for heterogeneous traffic based on single leader vehicles; however, the effect of other surrounding vehicles which is also influential in heterogeneous traffic with no-lane discipline behavior (i.e., space sharing based behavior) is not considered. This highlights the need to consider the different vehicles and their types in the surroundings of the subject vehicle.

1.3 Objectives, Scope and Methodology

The main objectives of the present study are:

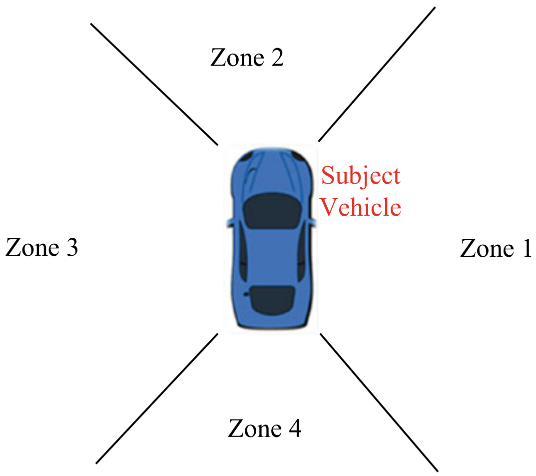
- Analyzing the longitudinal and lateral behavior of drivers considering surrounding vehicles on the mid-block sections in mixed traffic conditions.
- Development of a car following model for mixed traffic conditions under the conditions of having multiple influencing vehicles and no influencing vehicle (i.e., free flow) of motorized vehicles at the mid-block section.
- Development of lane changing model for mixed traffic conditions of motorized vehicles at mid-block section.

The scope of this study covers mid-block road sections. For this, detailed data collection, extraction, and analysis have been carried out to estimate vehicular movement in response to surrounding influencing vehicles. Further, the scope of this study is limited to motorized vehicles only.

2 Methodology and Data

The present study intends to develop various models for vehicular movement in urban mixed traffic conditions. Therefore, a detailed analysis of vehicle trajectory data of traffic flow has been carried out. The data is mainly extracted for the subject vehicle and influencing vehicles. These influencing vehicle(s) can be on any side of the subject vehicle and the proximity area around the subject vehicle is divided into four zones as presented in Fig. 1.

Fig. 1 Depiction of the 4 zones adopted for analysis around the subject vehicle



Keeping the objectives in focus, detailed data collection and extraction methodology have been proposed. As mentioned in the scope of this study, only mid-block road sections have been considered and a total of nine mid-block locations in four different cities of Ahmedabad, Meerut, Delhi and Hyderabad have been considered with varying road widths as given in Table 1. To obtain the data, a video camera was mounted at a vantage point to collect the vehicular movement behaviour. After the collection of traffic data, extraction of the collected data using the traffic data extractor (TDE) which is a semi-manual software package developed by IIT Bombay (TDE User Manual). The recorded videos have been imported into the TDE tool and calibrated using the marked rectangle. The road stretch is described by specifying the entry line and the exit line. The length and width of the stretch are entered as sides of a rectangle. Adopting this methodology, the trajectory data have been extracted using the TDE software tool. The sample size for different vehicle types is shown in Table 2. Since the modelling of vehicular movements is the intent of the study, the vehicle position in each frame is considered as one sample. A total of 49,251 frame counts have been extracted for different vehicle types which have been further analysed to understand vehicle behaviour and modelling vehicular movements. From these data, the significance of zones and the different influencing variables of each of these zones surrounding the subject vehicle have been analyzed.

Table 1 List of mid-block locations considered in the study

S. no	Location name	Road width
1	Harkesh Nagar, Mathura Road, New Delhi	11.2 m
2	Rajaram Kohli Marg, New Delhi	7.5 m
3	Balaji Temple Bypass Road, Hyderabad	9.0 m
4	Vijay Char Rasta, Ahmedabad	10.5 m
5	Paldi Road, Ahmedabad	8.0 m
6	Narol Sarkej Road, Ahmedabad	17.0 m
7	Garh Road, Meerut	8.0 m
8	Swastik Road, Ahmedabad	7.0 m
9	Maharani Bagh Road, New Delhi	10.0 m

Table 2 Extracted sample size of motorized vehicles on mid-block locations

Vehicle type	Sample size (Frame count)
Auto Rickshaw	5803
Bus	728
Car	16,402
Battery Cycle Rickshaw (CRB)	876
Heavy Commercial Vehicle (HCV)	799
Light Commercial Vehicle (LCV)	2275
MTW	21,921
Multi-Axle Vehicle (MAV)	447
Total	49,251

3 Data Analysis

Analysis has been carried out to find out speed variation with respect to the lateral displacement of the vehicle. Since there is no specific lane discipline, the road width has been laterally divided into three equal areas (left, centre and right) in order to understand vehicular behaviour with respect to placement on the road. Left area is near to shoulder/footpath and right area is near the median side. Figure 2 presents the average speed characteristics of Car, MTW and Auto moving in these left, central and right areas on the road. As can be seen in Fig. 2, the speed of Car, MTW and Auto moving in the right area, i.e. near the median side is higher compared to the other two areas, i.e. Left and Centre. Further analysis has been done on lateral placement distribution as shown in Fig. 2. It can be seen from Fig. 2 that Cars have a high tendency to travel on right rightmost side of the road whereas MTW and Auto have a high tendency to travel on the central part of the road.

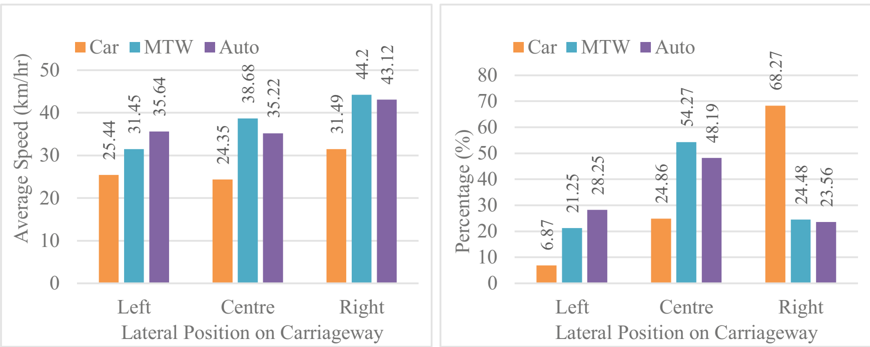


Fig. 2 Average speed and percentage distribution with respect to lateral placement on the carriageway

4 Car-Following Model

4.1 Subject Vehicle with Surrounding Vehicles

The present study intends to develop car following models to estimate vehicular longitudinal movement for urban traffic conditions within mixed traffic. Therefore, a detailed analysis of vehicle trajectory data of traffic has been carried out to prepare the data sets. The data was mainly extracted for two types of vehicles, viz. subject vehicles and influencing vehicles. Influencing vehicle(s) can be in any of the four zones surrounding the subject vehicle as shown in Fig. 1. The car-following model for motorized vehicles as subject vehicles with all surrounding influence vehicles has been developed considering various independent variables utilising multiple linear regression modelling. The independent variables considered are: Speed of the subject vehicle in the previous time interval, speed of the influence vehicle in that zone in the previous time interval, Lateral distance from the median for the subject vehicle in the previous time interval, Passenger car units of subject and influence vehicle to consider mixed traffic in terms of vehicle type, Road width and gap between subject and influence vehicle. The format of the proposed car-following model is given below.

$$\begin{aligned} \mathrm{V_S^i(t)} = {} & \mathrm{a_1 * V_S^i(t-1) + a_2 * DM_S^i(t-1) + a_3 * PCU_S + a_4 * RW} \\ & \mathrm{+ a_5 * PCU_{IV}^i + a_6 * V_{IV}^i(t-1) + a_7 * RD^i} \end{aligned} \tag{1}$$

where V is speed in m/s

Subscripts 'S' and 'IV' represent the subject vehicle and the influencing vehicle

Superscript 'i' represents zone (1-left; 2-front; 3-right; 4-back)

't' and 't–1' are current and previous time intervals, respectively

DM is the lateral distance from the median for the subject vehicle

RW is Road width in m

PCU is Passenger car unit

RD is the relative distance (gap) between the subject and the influencing vehicle in m

a_1, a_2, a_3, a_4, a_5, a_6, and a_7 are regression coefficients to be estimated.

The car-following equations for each zone are developed separately as given below:

$$\begin{aligned} V_S^1(t) = {} & 0.872 * V_S^1(t-1) + 0.018 * DM_S^1(t-1) - 0.099 * PCU_S \\ & + 0.020 * RW + 0.047 * PCU_{iv}^1 + 0.080 * V_{iv}^1(t-1) \\ & + 0.005 * RD^1 \cdots (R^2 = 0.97) \end{aligned} \tag{2}$$

$$
\begin{aligned}
V_S^2(t) = {} & 0.882 * V_S^2(t-1) + 0.029 * DM_S^2(t-1) - 0.099 * PCU_S \\
& + 0.053 * RW + 0.032 * PCU_{iv}^2 + 0.070 * V_{iv}^2(t-1) \\
& + 0.013 * RD^2 \cdots (R^2 = 0.98)
\end{aligned} \tag{3}
$$

$$
\begin{aligned}
V_S^3(t) = {} & 0.873 * V_S^3(t-1) - 0.003 * DM_S^3(t-1) - 0.116 * PCU_S \\
& + 0.033 * RW + 0.007 * PCU_{iv}^3 + 0.093 * V_{iv}^3(t-1) \\
& + 0.086 * RD^3 \cdots (R^2 = 0.97)
\end{aligned} \tag{4}
$$

$$
\begin{aligned}
V_S^4(t) = {} & 0.891 * V_S^4(t-1) - 0.012 * DM_S^4(t-1) - 0.062 * PCU_S \\
& + 0.040 * RW + 0.013 * PCU_{iv}^4 + 0.062 * V_{iv}^4(t-1) \\
& + 0.031 * RD^4 \cdots (R^2 = 0.98)
\end{aligned} \tag{5}
$$

The final resultant speed for the subject vehicle is considered as the average of speeds estimated in each of the zones as given below:

$$
\mathrm{V_S(t)} = \frac{1}{\mathrm{n}} * \sum_{\mathrm{i}=0}^{\mathrm{n}} \mathrm{V_S^i(t)} \tag{6}
$$

where n–Number of zones (left, front, right and back)

The R^2 values of Eqs. (2)–(5) show good statistical validity, hence it is considered that the developed car-following equations would be able to explain the vehicular movements with reasonable accuracy.

4.2 Subject Vehicle with No Surrounding Vehicles

Many times, the subject vehicle will be travelling on a link without having any influence from other vehicles because there will not be any vehicle present in the surrounding influence area. The car-following model for motorized vehicles as the subject vehicle with no influence vehicles in the influence area has been developed considering various independent variables utilising multiple linear regression modelling. The independent variables considered are: Speed of subject vehicle in the previous time interval, Maximum acceleration of subject vehicle, Lateral distance from median for subject vehicle in the previous time interval, Passenger car unit of subject vehicle, Road width and Free speed of subject vehicle. The developed car-following equations are given below:

$$
\mathrm{V_S(t)} = \mathrm{V_S(t-1)} + \begin{pmatrix} 0.048 * \mathrm{a_S} + 0.019 * \mathrm{DM_S(t-1)} \\ + 0.001 * \mathrm{PCUs} + 0.048 * \mathrm{RW} \end{pmatrix}
$$

$$* \left(1 - \frac{V_S(t-1)}{v_S^f}\right) * \left(1.15 + \frac{V_S(t-1)}{v_S^f}\right)^{2.975} \left(R^2 = 0.97\right) \tag{7}$$

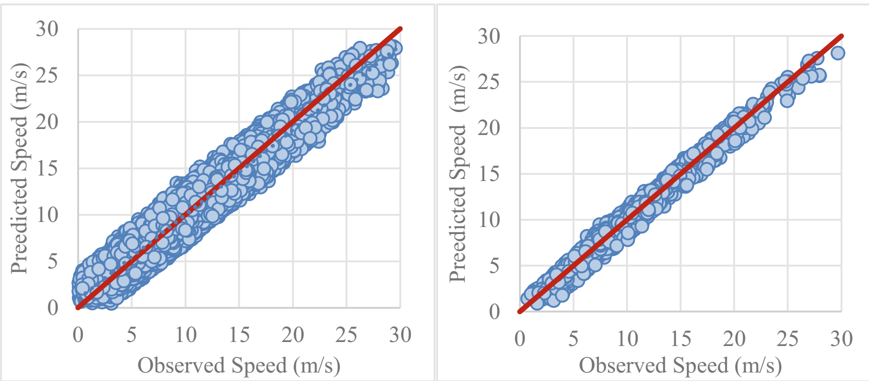
where v^f is free speed in m/s

a is maximum acceleration in m/s^2

4.3 Validation

In order to validate the above-developed equations, the predicted speed from Eq. (6) is compared with the observed speed in the field. A comparison graph has been plotted between the observed speed and predicted speed for the validation of car-following model with an influence vehicle as shown in Fig. 3a. Root Mean Square Error (RMSE) value for this model is found to be 1.45 m/s. From this, it can be concluded that the developed car-following model with surrounding influencing vehicles is able to predict the speed of the subject vehicle realistically with adequate accuracy.

Similarly, in order to validate the developed car-following model with no influence vehicle, the predicted speed from Eq. (7) is compared with the observed speed in the field. A comparison graph has been plotted between the observed speed and predicted speed for the validation of the car following the model with no influence vehicle as shown in Fig. 3b. The RMSE value for this model is found to be 0.83 m/s. From this, it can be concluded that the developed car-following model with no influencing vehicles (free flow) is able to predict the speed of the subject vehicle realistically with adequate accuracy.



(a) With surrounding vehicles (b) With no surrounding vehicle

Fig. 3 Observed and predicted speeds of subject vehicle in different conditions

5 Lane Change Model

From the data, the vehicle lateral movement behaviour in terms of lateral shift (difference in lateral position in two successive time intervals is measured in m) has been analysed with respect to the speed of the subject vehicle. The variation of lateral shift for different speeds of Car is shown in Fig. 4. From Fig. 4, it can be observed that the lateral shift of cars is more at slower speeds (<10 km/hr) and also for higher speeds (>70 km/hr). As the speed of the car increases, lateral shift decreases up to 0.15 m at 25 km/hr and increases up to 0.4 m at 95 km/hr. It can also be seen from Fig. 4, as the speed of MTW increases, lateral shift increases up to 0.4 m at 95 km/hr. As the speed of the auto increases, lateral shift increases up to about 0.18 m at 40 km/hr and reduces subsequently as shown in Fig. 4.

The developed lane change equations to estimate lateral shift for different vehicles are given below:

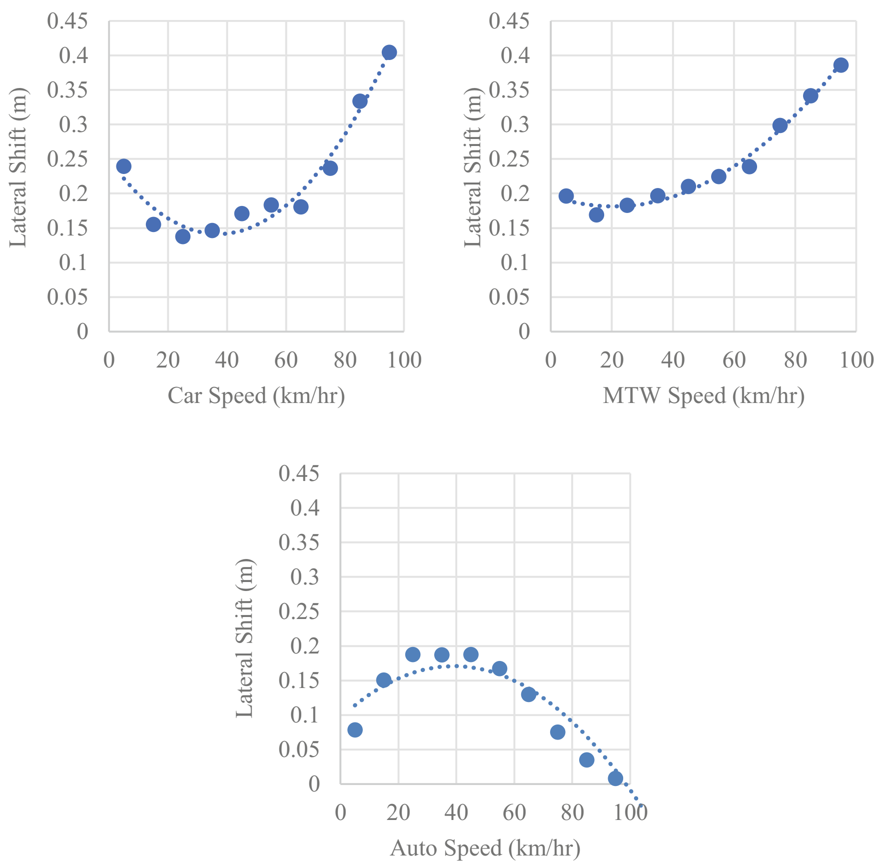


Fig. 4 Relationship between speed and lateral position of different vehicles

$$\mathrm{X}_{\mathrm{Lat}}^{\mathrm{Car}}(\mathrm{t}) = 0.2489 + 0.00008 * \mathrm{V}^{\mathrm{Car}}(\mathrm{t}-1)^2 - 0.0058 * \mathrm{V}(\mathrm{t}-1)\left(\mathrm{R}^2 = 0.96\right) \quad (8)$$

$$\Delta\mathrm{X}_{\mathrm{Lat}}^{\mathrm{MTW}}(\mathrm{t}) = 0.197 + 0.00004 * \mathrm{V}^{\mathrm{MTW}}(\mathrm{t}-1)^2 - 0.0015 * \mathrm{V}^{\mathrm{MTW}}(\mathrm{t}-1)\left(\mathrm{R}^2 = 0.99\right) \quad (9)$$

$$\Delta\mathrm{X}_{\mathrm{Lat}}^{\mathrm{Auto}}(\mathrm{t}) = 96.384 - 0.0485 * \mathrm{V}^{\mathrm{Auto}}(\mathrm{t}-1)^2 + 3.7976 * \mathrm{V}^{\mathrm{Auto}}(\mathrm{t}-1)\left(\mathrm{R}^2 = 0.87\right) \quad (10)$$

where ΔX_{Lat} is the lateral shift in m.

6 Conclusions

In this study, the longitudinal and lateral behaviour of vehicles under multiple influencing vehicles have been analysed and car-following and lane change models have been developed. The summary of findings from this study has been given below:

- Lateral placement distribution analysis found that Cars have a high tendency to travel on the right side of the road whereas MTW and Auto have a high tendency to travel on the central area of the road.
- Speed analysis found that the speed of Car, MTW and Auto moving on the right side of the road is higher compared to the other two areas, i.e. Left and Centre.
- Car following model has been developed under mixed traffic conditions considering the surrounding vehicles and no surrounding vehicle case for mid-block location.
- Vehicle behaviour in terms of lateral shift has been analysed with respect to speed and found that lateral shift in case of car is more in slower speeds (<10 km/hr) and higher speeds (>70 km/hr). As the speed increases, lateral shift decreases up to 0.15 m at 25 km/hr and increases up to 0.4 m at 95 km/hr. the speed of MTW increases, lateral shift increases up to 0.4 m at 95 km/hr. As the speed of the Auto increases, lateral shift increases up to about 0.18 m at 40 km/hr and reduces subsequently.
- Lane change model has been developed to estimate lateral shift under mixed traffic conditions for mid-block locations.

The developed car-following and lane change models in this study would eventually improve the predictions thus realistic evaluation of transport policies through microscopic simulation. In the future, it is also proposed to consider other vehicle types in modelling longitudinal and lateral behaviour so that predictions can become more realistic.

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Comprehensive Analysis of Road Accidents and Surrogate Measures to Enhance Road Safety

B. S. Jisha and M. Satyakumar

Abstract Worldwide, more than 50 million casualties occur in road crashes each year, in which 80% of road crashes are in developing countries. The country ranks one in the number of road accident deaths and an increase of about 47% in road crashes is expected in the next 20 years. Kerala is one of the top five accident-prone states in the country at the present time. Road safety becomes more and more important every year as the annual growth rate of traffic is more than 10% in Kerala. Road crashes tend to result in personal injury, loss of life, or damage to property. The challenging factors in the traffic conditions existing in our road networks are the mixed traffic conditions and vulnerable road users. The state needs a comprehensive road safety plan, which in turn requires enormous data with respect to the accident and its severity over a period of time in order to reduce the accident scenario. Contributing factors of road accidents need to be documented in order to decide the most appropriate solutions. The objective of this study is to analyze the accident data over a period of time, identify the causative factors, and suggest appropriate solutions. A 60 km stretch of National Highway NH 66 passing through the Kollam district of the state was selected for this purpose. Data from the State Crime Records Bureau (SCRB), over a period of three years from 2017 to 2019 was collected and analyzed. Year-wise accident statistics, trends and causative factors were identified. Certain locations were identified to be blackspots as per MoRTH standards were later taken up for detailed study and appropriate solutions were suggested.

Keywords Road safety · Road crashes · Crash severity

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1 Introduction

Road safety refers to the methods and measures used to prevent road users from being killed or injured in road accidents or road crashes. Traffic safety is a major concern in developing countries like India. The most common measures to define road safety are the number of road crashes, number of road casualties and other negative consequences of road crashes. India ranks 1 in the number of road accident deaths across 199 countries reported in the World Road Statistics, 2018. As per the World Health Organization, accident-related deaths are known to be the eighth leading cause of death and the first largest cause of death among children aged 5–14 and adults aged 15–29. Road accidents are the leading cause of injuries, death and disabilities. Hence, a comprehensive road safety plan is required in India to reduce the number of accidents and severity of accidents.

Road accident tends to result in personal injury, loss of life or damage to property. Accident is an unpredictable event caused by one or a combination of multiple factors. These factors can be grouped into three main categories, namely infrastructure, road user behavior, environmental. While road accidents are unpredictable events, the intensity can be reduced to a certain extent by employing suitable road safety measures. Therefore, a systematic analysis of road accidents is essential. A systematic approach is required to identify the contributing factors so that the most appropriate treatments can be selected and implemented. It is necessary to come out with a more comprehensive and systematic road safety plan to effectively reduce the crash frequency and to ensure safer traffic on our road networks.

1.1 Literature Review

Rolison et al. [8] The main causes of road accidents are identified by multiple sources of reviews: expert views of police officers, lay views of the driving public, and official road accident records. The results reveal potential underreporting of factors in existing accident records, identifying possible inadequacies in law enforcement practices for investigating driver distraction, drug and alcohol impairment, and uncorrected or defective eyesight. The need for the accident report forms to be continuously reviewed and updated to ensure that contributing factor lists reflect the full range of factors that contribute to road accidents is highlighted. The delay in completing accident report forms should be minimized, possibly by the use of mobile reporting devices at the accident scene.

Hu and Xiang [7] from their study, it was concluded that the geometric characteristics of the study corridor were observed to be varied, which affects the operating speed thereby affecting the level of safety. The operating speed of the highway was found to be the most influential factor in all models developed. The relationship between contiguous elements was established by developing operating speed models for the curve and tangent section and was observed that the operating speed of one element is associated with the other element. The developed crash prediction models gave insight that not only the segment under study have an influence on the crash but also succeeding and preceding sections have a significant role in the crash occurrence.

Baktha [3], mobile apps have become an integral part of our daily lives due to the various functionalities that they offer. The developer should consider the challenges faced and try to overcome them by following the proper steps. Also, it is imperative for the developer to have an open mind and should be well apprised about the current technologies, requirements and events in the mobile application field. To build a successful, all the guidelines should be properly considered and followed appropriately to avoid the risk of losing users due to lamentable/falling apps which leads to a scope for future areas of research.

1.2 Objectives

The objectives of the study are

- To identify the hotspots from the accident records
- To find out the causative factors by detailed study
- To suggest surrogate measures to improve the safety in the selected stretch.
- To develop a mobile application to alert the drivers.

2 Methodology

The methodology of the study is shown in Fig. 1.

3 Accident Scenario

Total number of crashes occurred, total injuries and no. of deaths due to road crashes from the year 2009 to 2019 were collected from the State Crime Records Bureau. Figure 2 shows the plot of the total number of crashes, total injuries and the death rate from 2009 to 2019. The number of reported accident cases is increasing year by year in general. Even if the total number of accident cases shows variation, the death rate as well as total injuries due to accidents continuously increasing. It shows that the effectiveness of the safety measures adopted so far is questionable. Implementation of more effective safety measures is essential in the present scenario.

Analysis shows that in most of the district's number of crashes increases year by year. Our road network includes different types, geometrical and traffic characteristics of each type are different. In Kerala, the road network includes the National Highway, State Highway, Major District Roads, Other District Roads, Village Roads, and PMGSY Roads. Figure 2 shows the road network has only 1% of the road network

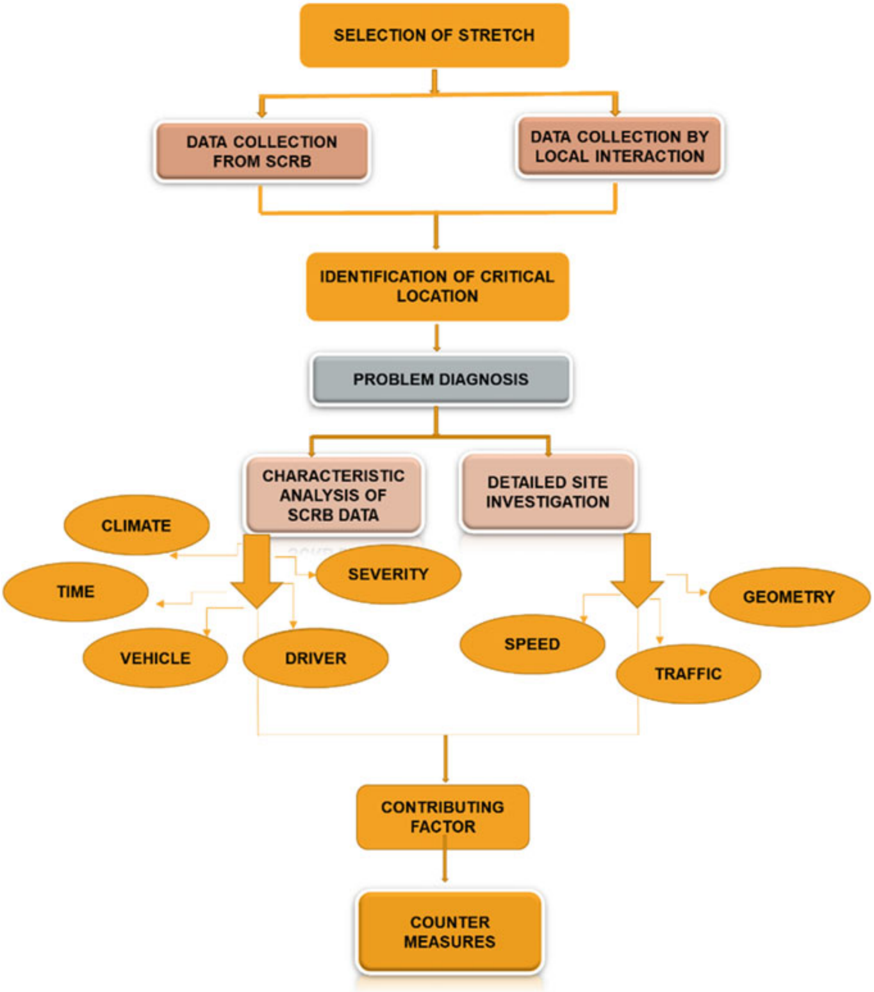


Fig. 1 Methodology

in Kerala is National Highways and 24% of the total road crashes occur in this category of road. In State Highways also, the crash frequency is more. National Highways and State Highways are the higher road categories, a greater number of road users and thereby a greater number of road accidents. The design speed and design specifications of both the categories are same. Figure 3 shows the road network and percentage distribution of crashes on different categories of roads.

Traffic conditions existing in the road networks of Kerala are mixed in nature. The type of vehicle involved in the majority of accidents is two-wheelers. Two-wheeler users are the most vulnerable. 40% of the accidents occurred in the state are due to two-wheelers.

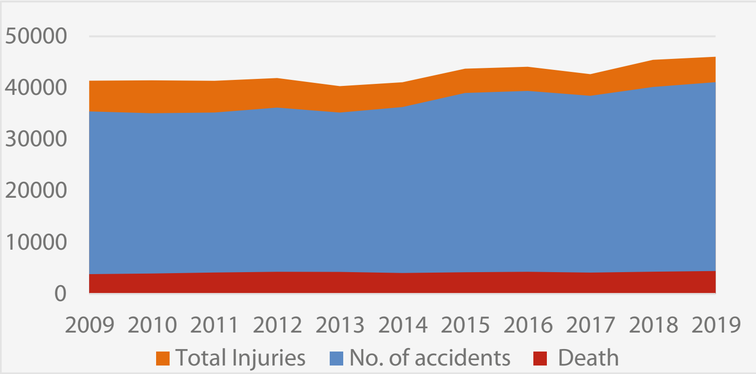


Fig. 2 Road crash scenario in Kerala from 2009 to 2019

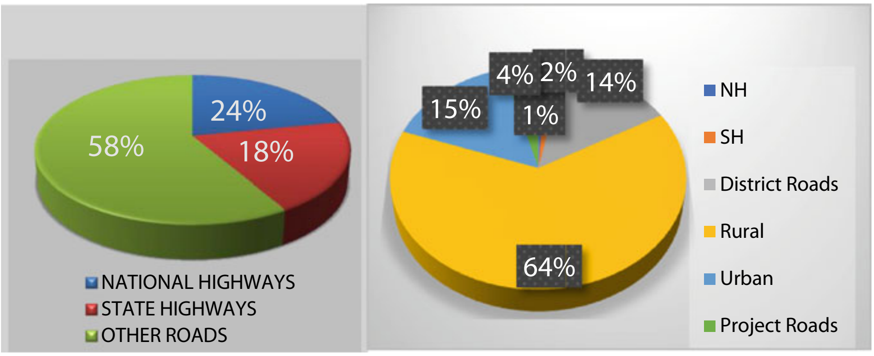


Fig. 3 Category-wise road network and crash distribution

4 Study Stretch

For detailed analysis, 60 km road was selected as the study stretch. The selected stretch is a portion of NH 66 passing through Kollam district. The traffic condition on the road is mixed in nature and it is a two-way two-lane undivided highway. The stretch includes straight portions, curves, bridges, intersections, urban roads, etc. The accident records of this stretch were collected from the State Crime Records Bureau. Three years' accidents data was collected (Fig. 4).

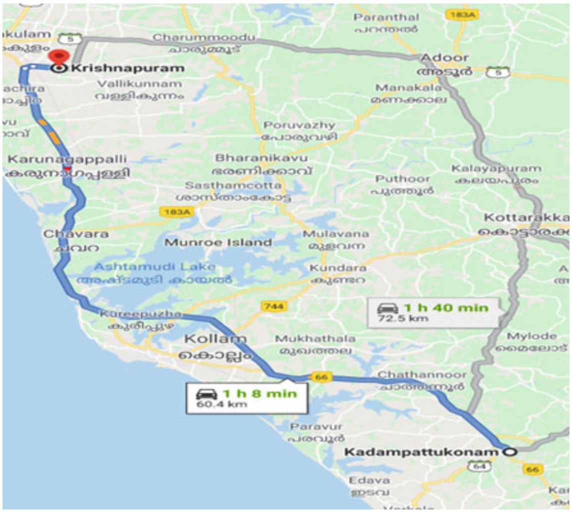


Fig. 4 Study stretch

5 Characteristic Analysis

Characteristic analysis of the accidents occurred was performed based on time of occurrence, category of vehicles, driver characteristics, climatic conditions. The severity of the accidents was also analyzed in the selected stretch.

5.1 Time of Occurrence

Majority of accidents occurred between 3 and 9 pm. About 40% of crashes occurred during this time. Daytime 9 pm to 12 pm is also critical, about 20%. Traffic volume is higher during these hours and evening 6–9 pm the visibility of the drivers is restricted. However, 65% of total crashes occurred in the daytime, from 6 am to 6 pm, and 35% were during night, 6 pm–6 am. Even if the traffic volume is lesser after 9 pm 15% of accidents occurred from 9 pm to 6 am. If special attention should be given to making the drivers more vigilant during night driving this 15% can be reduced as the traffic volume is very less. Most of the accidents occur during the night because of fatigue or drowsiness (Fig. 5).

5.2 Category of Vehicles

The traffic consists of different vehicle compositions. From the analysis, it is clear that the most vulnerable road users are two-wheeler drivers about 55% of vehicles

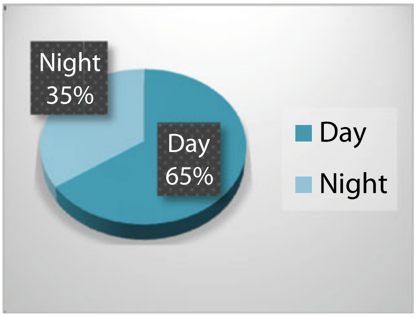


Fig. 5 Distribution of crashes during day and night

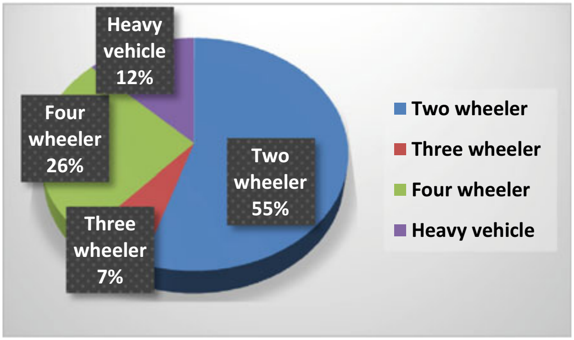


Fig. 6 Percentage wise distribution based on category of vehicles

involved in accidents are two-wheelers and 26% are four-wheelers which includes cars, jeep and taxis (Fig. 6).

5.3 *Driver Characteristics*

Driver characteristics, age of the driver and sex of the drivers of the vehicles are studied. Young drivers, age group of 18–29, make more accidents. This may be due to a lack of experience and immature behavior to traffic. Out of the drivers, 60% of drivers are male drivers on the selected stretch. Male drivers make more accidents compared to female drivers (Figs. 7 and 8).

5.4 *Severity of Accidents*

The severity of accidents that occurred on the selected stretch is also studied. 74% of the accidents are grievous in nature (Fig. 9).

Fig. 7 Percentage of accidents based on age of the driver

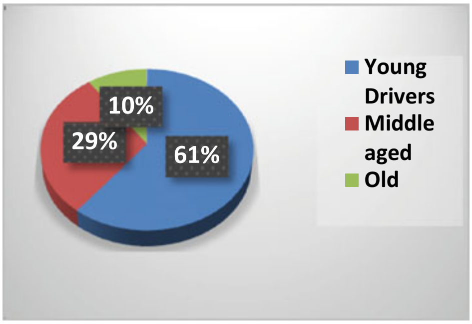


Fig. 8 Percentage distribution based on gender

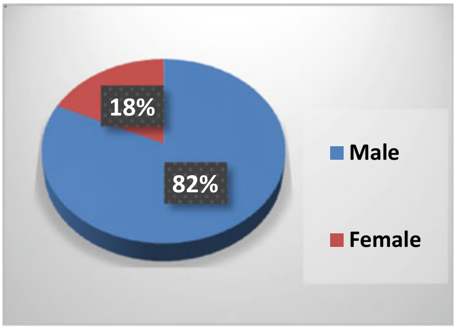
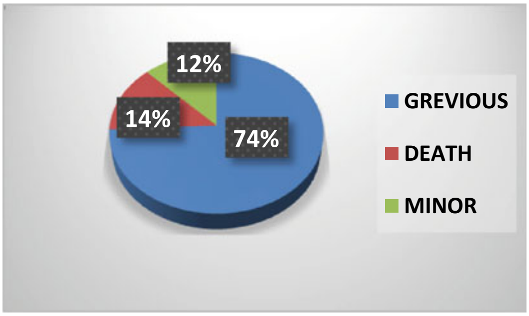


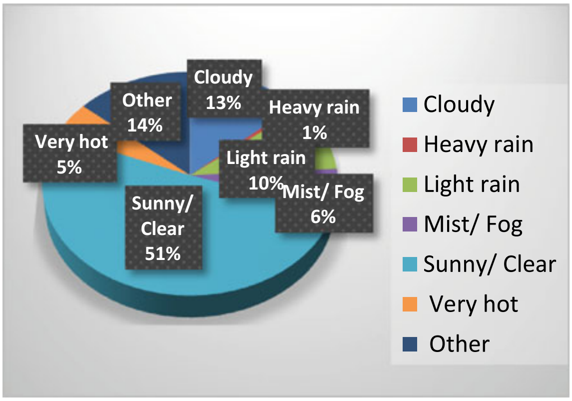
Fig. 9 Severity of accidents



5.5 Climate

The climatic conditions at the time of occurrence of the accidents were also studied. About 51% of the accidents occurred in clear sunny climates. From this, it is inferred that the worst climatic condition is not the main factor that leads to accidents. Drivers are more careful while driving the vehicles in worst climatic conditions. And also, the absence of two-wheelers during rainy times may also a reason behind this (Fig. 10).

Fig. 10 Percentage wise distribution based on climatic conditions



6 Detailed Investigation

The critical locations (hotspots) were identified on these stretches in terms of crash frequency. A stretch of about 500 m in length in which either 5 road accidents (in all 3 years put together involving fatalities/grievous injuries) took place during the last three calendar years or 10 fatalities (in all 3 years put together) took place during the last 3 calendar years is noted as hotspot (MoRTH). 27 hotspots were identified in the selected stretch and listed in Table 1.

These 27 hotspots comprise straight stretches, curved portions and intersections as listed above. Detailed investigation includes measuring the carriageway width, shoulder width, presence of road markings, sign-boards, signals, etc.

6.1 Straight Stretches

26 straight stretches and 14 intersections were analysed. Out of which, most of the stretch's carriageway width is just sufficient but almost no shoulders or insufficient shoulder width. Out of the total locations investigated, about 70% of the test sections have no shoulders. Only paved or dense-graded shoulders were considered as shoulders. Grass and soil are not suitable driving surfaces and therefore normally do not function as shoulders. Traffic signage helps drivers remain aware of what's coming up and what's important on the road ahead. It's one of the oldest ways of enforcing safety for both drivers and pedestrians on the road. In some portions absence of signboards, markings, speed breakers and other safety measures were missing.

Table 1 Identified hotspots in the selected stretch and its geometry

Sl:no	Place	Bridge	Straight	Curve	Intersection
1	CHANGANKULANGARA		✔		✔
2	CHATHANNOOR		✔		✔
3	CHAVARA		✔	✔	
4	CHINNAKKADA		✔		✔
5	EDAPALLYKOTTA			✔	✔
6	ITHIKARA	✔	✔		
7	KANNETTY	✔	✔	✔	
8	KARUNAGAPALLY		✔		✔
9	KMML JUNCTION		✔		
10	KOTTANKULANGARA		✔		
11	KOTTIYAM		✔		✔
12	KSRTC JUNCTION (KNPLY)		✔		
13	KUTTIVATTOM		✔	✔	
14	MEVARAM		✔	✔	✔
15	MYLAKKAD		✔		
16	NEENDAKARA	✔	✔	✔	
17	OCHIRA		✔		
18	PALLIMUKKU		✔		✔
19	PARAKKULAM		✔		
20	PARIMANAM		✔		
21	PARIPALLY		✔		✔
22	POLAYATHODU		✔		✔
23	PUTHENTHURA		✔	✔	
24	PUTHIYAKAVU		✔		✔
25	THATTAMALA		✔		✔
26	UMAYANALLOR		✔		✔
27	VAVVAKKAVU		✔		✔

6.2 *Curves*

Curved portions in the selected stretch were analysed. Sight distance, superelevation and speed on the curves are the most important factors associated with curves. Sight distance in most of the locations was sufficient. In horizontal curves accidents occur when the centrifugal force is more than the direction and momentum of the force, which makes the vehicle move in a straight line instead of curved path. Under estimation of speed may be a contributing factor while moving through curved roadways, which necessitates speed adjustment and thus causes fatal and serious injury accidents, especially in heavy vehicles. Approaching sharp curves without realising the speed and the driver fails to decelerate traffic incidents may occur. Warning signs are necessary to warn the driver that he is approaching the curves and has to move with utmost care. Warning signs were absent in most of the curves, shoulder width is also insufficient.

6.3 Bridges

Three bridges are there on the list of hotspots. The carriageway width at these three locations is less than required and shoulder width is less for two bridges and no shoulder for one.

6.4 Traffic Characteristics

Traffic Volume. During Traffic volume count was taken by videographic method. Peak hour traffic volume is also counted as peak hour, traffic volume is nearing its capacity. A spot speed study was also conducted.

Operating Speed

Speed limits indicate the maximum and minimum speed a driver can move in a certain category of road, assuming that there are favourable traffic and weather conditions. The 98th percentile speed was 65 KPH and the 85th percentile speed, limit for speed regulation was 50 KPH. Drivers may cause accidents by engaging in the following behaviours: Driving too fast, driving 5–10 kmph over the speed limit is not over speeding; however, even traveling just a few kmph over the speed limit can have serious consequences. Driving too slowly can also be as dangerous as driving too fast. If the vehicle moves 5–10 kmph or slower than surrounding traffic, you are more likely to be in an accident. Failing to maintain a reasonable speed; drivers who fail to consider weather conditions, traffic conditions, and roadway conditions while driving may end up in an accident, even if they are not technically exceeding the speed limit.

From the detailed analysis it can be concluded that over speed, careless driving absence of warning signs are the main causes of accidents. The driver by knowing this hotspot region can reduce the speed of the vehicle and drive cautiously through the road and reducing the number of accidents. As smartphones become affordable for common people, they are starting to use them in all different parts of life. Nowadays, the use of smartphones with different mobile applications is common, especially in the transportation sector. These techniques combined with the right software can provide the user the location-based information, which can help in different ways.

7 SAAVADHAN -Mobile Application to Alert the Driver About Hotspots

"SAAVADHAN" is a mobile application developed to alert the users about the hotspot regions. The app mainly focuses on the safety of the user using it. It alerts people about the accident-prone zones ahead of them using a beep sound. Thus, the drivers

Fig. 11 Logo of mobile app

slow down the vehicle and drive safely through the particular region. This helps in reducing the number of accidents occurring in that region. Driving safely and comfortably is one of the key factors every user needs or desires. “SAAVADHAN” provides safety through its efficient working and properties. The application indicates to the drivers about the accident-prone zones ahead of them when the vehicle reaches within 200 m of the area. Thus, by slowing down the vehicle the user could safely go through the particular region. It efficiently works for the safety of the users (Figs. 11 and 12).

7.1 Features of the App

The application was developed in the expo platform. Expo is an open-source platform for making apps for Android, iOS, and the web with JavaScript. The main features of the mobile application are.

Accurate GPS Location

The application provides accurate GPS location. There is no delay in showing the respected hotspot region. The predicted location for a user is detected and spotted well by the app. It is a kind of helping tool for navigation.

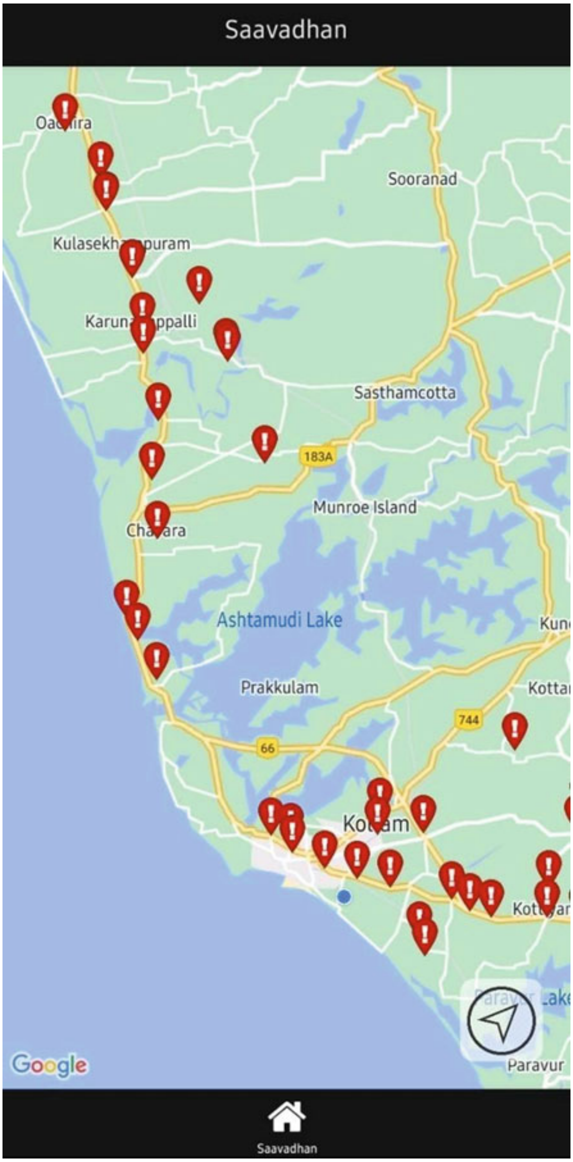
Alerts Only in the Direction of Driving

The application provides specific alerts in the direction of the stretches chosen. The alert is in the form of a beep sound.

Voice or Sound Alert When Approaching a Hotspot Region

It alerts the driver by a beep sound around 200 m advanced of the hotspot. It would be safer for the users as it alerts them before reaching the accident-prone zone.

Fig. 12 Screenshot of the page showing hotspots



Safe Drive

Safety during driving is one of the main features of the application. The beep sound alerts the drivers about the accident-prone zone which helps them drive safely through the respective region. It is also helpful during night hours when there is a possibility for the drivers to sleep while driving. The beeping sound helps them to be alert and thus avoid road accidents.

Offline Navigation

The application not only provides online alerts but also works offline. Even if there is any network interruption or data issues, the app works smoothly and effectively. Thus, helping the user in safe driving through hotspot regions.

7.2 Efficiency of the App

For the time being to test the efficiency of the app, a few test drives were conducted among a few young drivers. It worked satisfactorily. The mobile phone is an essential device nowadays and the beep sound from the device alerts the driver in the most efficient way. Usually, the reaction time of the driver is 2 s, the warning in advance of 200 m helps the diver to be more careful while approaching the hotspot region.

8 Results and Discussion

A detailed analysis of the characteristics of the accidents that occurred based on the reported accidents and a detailed investigation of the hotspots was done on the study stretch and the following conclusions were made.

- Number of accident cases increases as the number of registered vehicles increases day by day. Road safety measures implemented so far are not sufficient to prevent fatalities or loss of life due to road accidents
- 65% of the accidents were reported during the daytime as the number of vehicles was more in the daytime, 15% were after 9 pm even with less traffic volume.
- 55% of vehicles involved in accidents were two-wheelers and young male drivers with age group 18–29 were involved in more accidents.
- 74% of the accidents were severe with grievous injury or loss of life and more cases happened in sunny clear climates.
- 27 hotspots were identified in the stretch. From the detailed investigation of geometrical and traffic features most of the accidents occurred in straight stretches, and curves are due to overspeed, carelessness of the young drivers, absence of warning signs insufficient carriageway and shoulder width or no shoulders.
- To reduce the number of crashes due to overspeeding of young drivers, absence of warning signs an Android mobile application ‘SAAVADHAN’ was developed which alerts the driver 200 m in advance of the hotspot region by a beep sound. The working of the application was checked and found satisfactory.
- From local interactions and also from interaction with police officials, a number of minor accidents were unreported and the accident data from the police records alone are not sufficient to do a comprehensive analysis.

- The accidents due to driver negligence can be reduced by giving road safety education and through enforcement measures. Accidents due to overspeed can be reduced by providing optical treatments which can be applied to the road surface as markings, which creates a feeling or even illusion to the driver that he is going too fast.

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Visualising Blackspot Improvement at Nagpur

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Abstract Nagpur, the "Orange City" of the country is situated in the state of Maharashtra. Nagpur is one of the fastest economically growing cities and part of the "Smart City Mission". In this paper, the city of Nagpur has been identified as the test bed to understand and demonstrate the before and after scenarios of black spot improvement through 3-D modelling. In this regard, road crash data obtained from Nagpur Police in the form of First Information Reports (FIRs) was analysed to identify the black spots conforming to the protocol of the Ministry of Road Transport and Highways (MoRT&H). The geometric countermeasures deduced for two out of the 37 locations are illustrated by presenting 'before' and 'after' scenarios through 3-D Modelling with the primary objective of pictorially depicting the efficacy of the proposed intervention for two out of the 37 locations in Nagpur. The above form of pictorial illustration of the detailed Geometric Design Plan (GDP) for the above black spots will serve as an eye-opener for the relevant stakeholders in terms of undertaking steps towards the implementation of the suggested black spot remedial measures. It is clarified that the overall project termed **Intelligent Solutions for Road Safety through Technology and Engineering (iRASTE)** includes redesigning the road geometry as well as applying Artificial Intelligence (AI) technology to achieve its aim of reducing road fatalities by at least 50%, safety of pedestrians, and ease in traffic flow. However, this paper aims to highlight the importance of 3-D visualization for the representation of design interventions through which the policymakers, stakeholders and the general public alike can easily understand.

Keywords Transportation design · Road safety · Crash prevention · 3-D Visualization

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1 Preamble

In any city, all roads are safe until *they are not,* hence proper road designs are required. Road geometry plays an important role in ensuring the safety of all road users on the road network. To address the above, the city of Nagpur has been identified as the test bed to understand the demonstrate the 'before' and 'after' scenarios of a detailed geometric design plan (GDP) through 3-dimensional (3-D) Modelling in this paper. In this regard, road crash data obtained from Nagpur Police in the form of First Information Reports (FIRs) was analysed to identify the black spots conforming to the protocol of the Ministry of Road Transport and Highways (MoRT&H). While designing the above improvement plans at the identified black spots, the modal share was taken on board along with the functional classification of the road links in the form of arterial/sub-arterial/collector which carry huge volumes of traffic [1]. Further, the lanes leading to the markets are very narrow and also have unregulated parking which causes traffic congestion majorly during peak hours. Lack of proper road and street designs causes traffic-related issues which not only include congestion problems but also can lead to road crashes. Considering the above, it is envisaged to **develop remedial measures, *i.e. countermeasures* for the identified black spots**. In this context, 37 Black spots have been identified based on the analysis of the First Information Report (FIR) obtained from the Nagpur Police. After this, the geometric countermeasures are deduced for the above in Nagpur and two out of the 37 locations are illustrated by presenting 'before' and 'after' scenarios through 3-D Modelling with the primary objective of pictorially depicting the efficacy of the proposed intervention. The above form of pictorial illustration of the detailed Geometric Design Plan (GDP) for the above black spots will serve as an eye opener for the relevant stakeholders in terms of undertaking steps towards the implementation of the suggested black spot remedial measures. It shall be noted that under the larger umbrella of the study titled, *Project: iRASTE, i.e. Intelligent Solution for Road Safety through Technology and Engineering* the above-identified blackspots i.e. *crash-prone areas with a history of fatalities and several crashes* as well as the identification of the grey spots *i.e. locations vulnerable to becoming black spots have been identified* and presented in Fig. 1. However, in this paper, the engineering measures conceived for a couple of black spots have been illustrated through 3-D Modelling, and hence any discussion on the grey spots is beyond the purview of this paper.

2 An Overview of 3-D Modelling

3-D visualization is a decade-old technology/tool. However, this technology kept evolving to become hyper-realistic in representing anything in a three-dimensional form on a screen or paper and even in physical forms. The need for the most advanced 3-D visualization came from one simple concept–bridging communication gaps between clients and artists. One of the primordial purposes of visualization

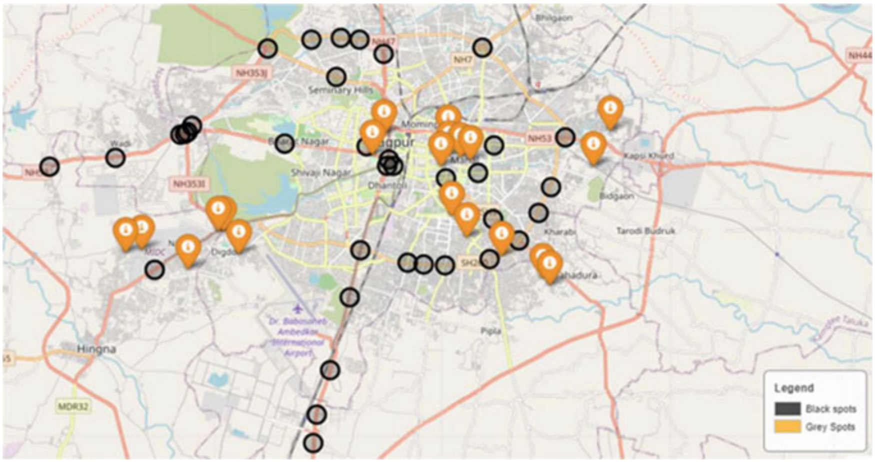

Fig. 1 Black spot locations (*Source iRASTE Dashboard*)

was communication. Because of that, visualization has continuously been evolving". [2]The very first 3-D modeling software was developed by Iwan Edward Sutherland at MIT in 1958 which revolutionized the *human–computer interaction.* Since then, a lot of 3-D modelling software has been developed. This method of visualization soon found its way into the architecture industry and then later into the film industry. The use of 3-D Modelling is also evident in developing a greater understanding of complicated designs in a much nontechnical manner for a layman to understand. In India mostly, 3-D modelling and rendering of walkthroughs are limited to architectural firms and some government departments, which require 3-D visualizations. Many large-scale projects such as highway design or a redesign of a long stretch of the street including intersections are mostly done in the 2-D format in Computer Aided Design (CAD) software, they hardly find their way to 3-D modelling and rendering. One of the most popular projects which used 3-D rendered walkthroughs is the Coastal Road Project in Mumbai, the video of the conceptual design circulated on various social media platforms made it very easy for even the general public to understand the proposal. In this regard, the study report of the Federal Highway Administration (FHA), Washington DC [3] lists the benefits that have been derived out of the 3-D visualization accomplished through the 3-D illustration of the benefits that accrued through the depiction of Road Safety Audit (RSA) projects:

- It enabled members of the RSA team to read the design more efficiently, especially those who are not well-versed in reading complex designs.
- The model helped in depicting all the details of the design both the horizontal features as well as the vertical ones by giving the idea of the surroundings, abutting land uses, heights of the buildings, etc.
- 3-D model provided a platform in the form of vantage points from which the proposal can be viewed as compared to 2-D.

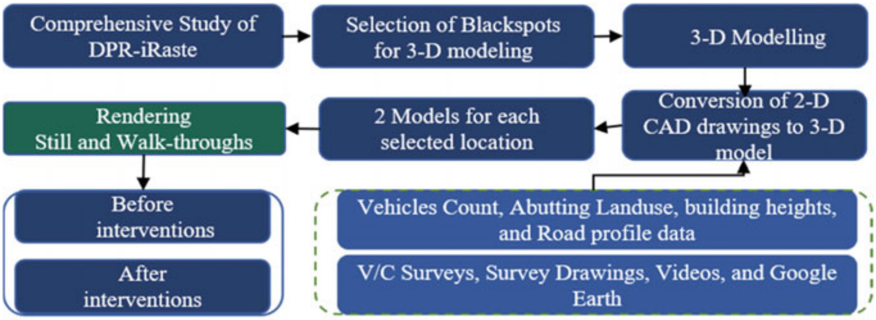

Fig. 2 Study methodology

- It helped in easily conveying design to all the stakeholders which include non-technical persons.
- In Virginia, the 3-D model helped visualize the impacts of proposed designs in sensitive areas."
- In Rhode Island, the 3-D model allowed for the review of locations that were not readily accessible to the RSA team [3].

Considering the above inherent benefits of the pictorial illustration of the 3-D Modelling, an effort has been made in this paper to illustrate the Detailed Geometric Design Plan (GDP) conceived for two out of the 37 black spots in the city of Nagpur through the above tool.

3 Methodology

The study Methodology devised to demonstrate the conceived GDP is presented in Fig. 2.

The 3-D models are developed for the existing and the proposed design by making a walk-through video by placing them side by side. These depict the interventions, some of which are also captured in still renders as shown in the succeeding sections.

4 Visualization of GDP Conceived for Black Spots

As mentioned earlier, the detailed geometric plan (DGP) deduced for two out of the 37 locations is illustrated by presenting 'before' and 'after' scenarios through 3-D Modelling is illustrated in this paper. This focuses on one Intersection (namely Jhansi Rani Square having five arms) and one midblock (namely, the road stretch from Telephone Exchange to CA road) in Nagpur city., Nagpur. The data deployed for 3-D modelling and walkthrough rendering for the two black spots encompassed 2-D CAD drawing depicting the existing physical survey plan and the proposed detailed GDP, Classified Volume Counts, Pedestrian Volume Counts, Stationary and mobile Videos, Google Earth, various [4–6] and Detailed Project Report (DPR) of Nagpur Metro.

4.1 Jhansi Rani Square 1

This intersection is located near the Sitabuldi area in Nagpur. It has five arms consisting of -AH-46, SH-255, and SH266, a road leading to Nagpur Railway Station, and one to Sitabuldi Metro station. The traffic count observed for a period of 12 h in both directions is 55,774 PCUs (74,871 vehicles) with an average speed of 31.9 kmph. The most preferred mode of transport in this area is two-wheelers (62%), and the overall modal split at this intersection is shown in Fig. 3. The average crashes at this location are around 32 before countermeasures and the number is estimated to decrease to 11 after the design intervention (which includes a host of countermeasures, traffic calming measures, etc.) which is a significant 66% decrease. The existing road geometrics is depicted in Figs. 4 and 5 whereas Fig. 6 presents the existing road geometry in the form of 2-D CAD drawings illustrating the existing situation. The proposed intervention is depicted in Fig. 7 which broadly encompasses the following:

- Channelizing islands and depressed medians provision with cement concrete bollards to provide safe pedestrian crossing.

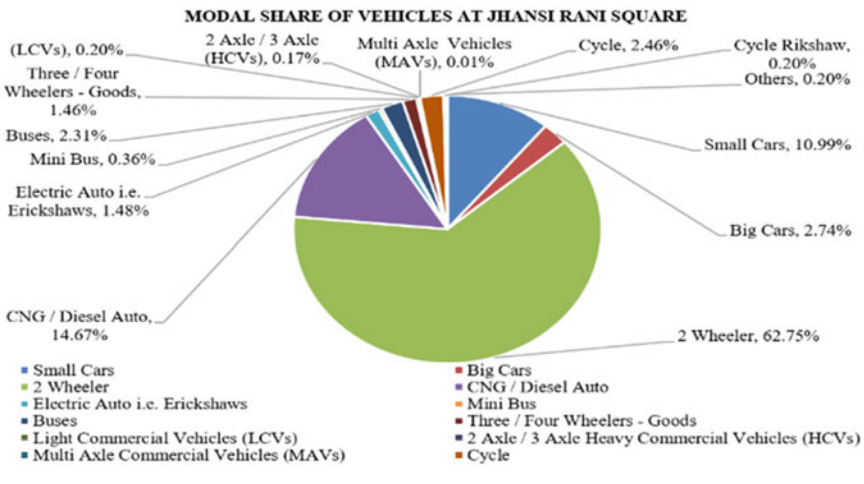

Fig. 3 Modal split at Jhansi rani square 1

Fig. 4 Absence of zebra crossing

Fig. 5 Absence of box marking at the crash prone location

Fig. 6 Existing geometric profile of the candidate intersection

Fig. 7 Proposed geometric design plan of the candidate intersection in 2-Dform

- Pedestrian crossing signals are to be installed at a distance of 45 m from the zebra crossing; the signal will be open for pedestrians for 20 s during the 60-s signal time.
- Transverse Bar Makings (TBMs) and rumble strips to be provided in both directions of travel. Provision of missing informatory road signs, retro-reflective pavement marking, edge delineators, and road studs.
- Provision of a speed table equipped with Blinking Solar Signals at free left turns on every approach.

Figure 8 presents the typical snapshot illustrating the existing conditions in the case of the Do Nothing, i.e. Business As Usual (BAU) scenario taken from the 3-D rendering video. On the other hand, Fig. 9 presents a bird eye view with interventions and similarly, Fig. 10 presents the typical view from the driver's seat depicting the proposed interventions whereas Fig. 11 presents the typical view of safety measures for one of the intersection approaches after interventions. Basically, the 3-D rendering video has been created by covering all 5 arms of the candidate intersection spanning a length of 250 m from the center of the intersection. As mentioned earlier, this kind of rendering thus helps in showcasing the 'before' and 'after' scenarios.

Fig. 8 Typical illustration of 3-D rendering of the ground conditions without intervention

Fig. 9 Typical illustration of 3-D rendering of the ground conditions with intervention

Fig. 10 3-D view from the driver's seat with intervention

Fig. 11 Typical 3-D view of safety measures on one of the intersection approaches after intervention

4.2 Telephone Exchange to CA Road

This is a mid-block location (on AH-46) which is located between two intersections-namely, Telephone Exchange Square and Chappru Nagar Square. The proposal spans a distance of about 1 km. This black spot intervention comprised the redesign of the entire stretch of the road including the above intersections. The traffic count observed at the midblock section during the 12-h covering both directions of travel is 40,835 PCUs (50,557 vehicles) with an average speed of 39.5 km/h. The most preferred mode of transport in this section is again two-wheelers (69%), and the overall modal split at this intersection is shown in Fig. 12. The average crashes at this location are around 37 before countermeasures and the number is estimated to decrease to 15 in the vent of the implementation of the above design intervention (like DGP, traffic calming measures, etc.) which is a significant 59% decrease. The existing traffic scenario and road geometrics at this midblock are presented in Figs. 13 and 14 respectively whereas Fig. 15 presents the existing road geometry in the form of 2-D CAD drawings illustrating the existing situation. Figures 16 and 17 present the typical snapshot illustrating the existing conditions in the case of BAU scenario taken from the 3-D rendering video. The proposed intervention is depicted in Fig. 18 which broadly encompasses the following:

- Bus bay provision on both sides of the highway after the intersection approach in Chappru Chowk.
- Channelizing the island with depressed medians to allow safe pedestrian crossing at both Chappru Chowk and Dr. Rajendra Prasad Chowk.
- Incorporation of traffic calming measures before reaching the Telephone Exchange metro station to reduce the travelling speeds where pedestrian volume is on the higher side.

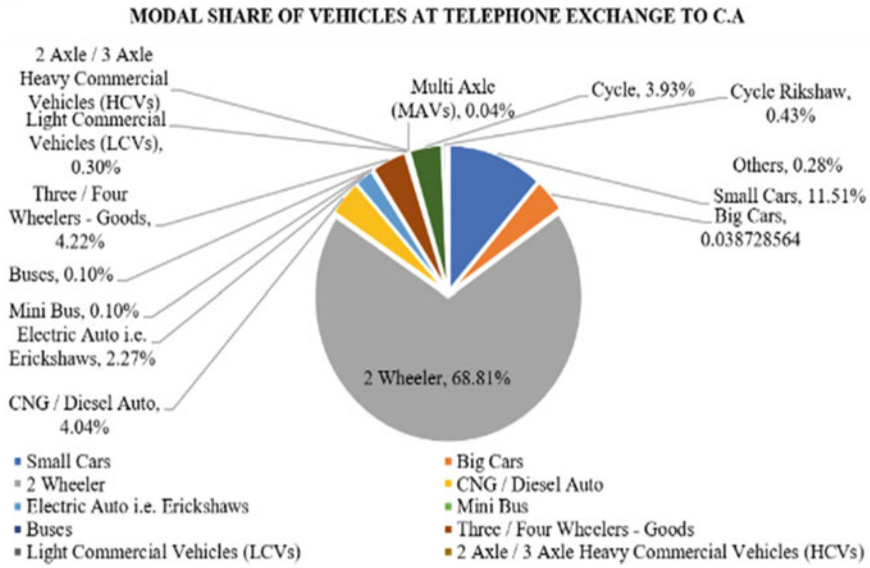

Fig. 12 Modal split on telephone exchange to CA Road

Fig. 13 **a** Absence of zebra crossing at the Rajendra Chowk **b** On street parking on the corridor: Major safety issue

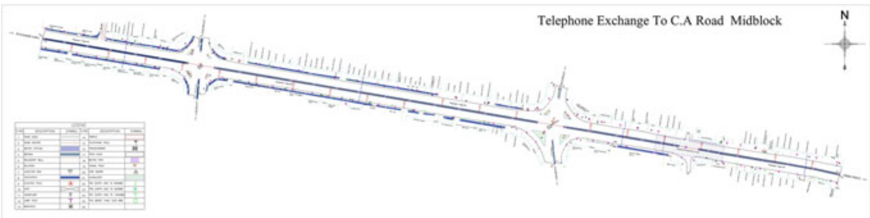

Fig. 14 Existing geometric profile of the candidate Midblock location

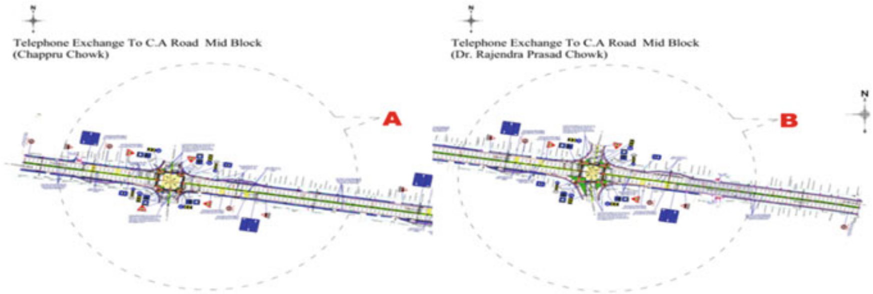

Fig. 15 Proposed geometric design plan of the candidate Midblock location spanning for 1 km in 2-D Form

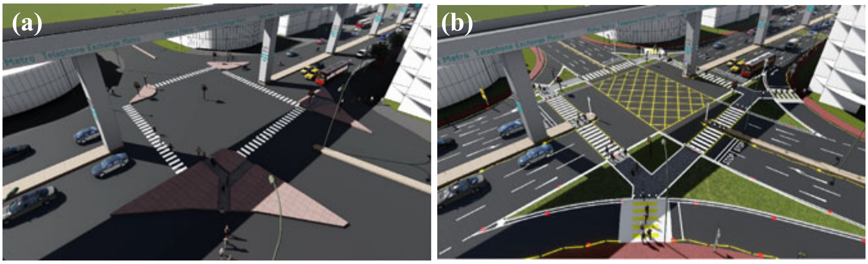

Fig. 16 a & b Typical illustration of 3-D rendering of the ground conditions without intervention on the candidate Midblock

Fig. 17 Typical illustration of 3-D rendering of the ground conditions without intervention on the candidate midblock at Rajendra Chowk

The images above depict the change in the level of understanding and interpretation of the proposal. The 2-D drawings consist of the technical data from the survey by using the proposed GDP drawings whereas the 3-D renders translate them into a realistic form that makes them understandable for the stakeholders and even the common public.

Fig. 18 Typical illustration of 3-D rendering of the ground conditions with intervention on the candidate intersection at Rajendra Chowk

5 Concluding Remarks and Future Scope

This study demonstrated the effectiveness of the 3-D model and its rendering helped in understanding its efficacy. Thus it can be inferred that a 3-D walkthrough can let the observer move around the road or the entire stretch and get a realistic idea of the proposal and thus provide angles that cannot not be accessed on foot due to running traffic and other safety concerns. Some inherent benefits derived through the study are listed:

- A 3-D walkthrough or even a still render gives the z-axis -*the missing vertical which* adds the height factor to the drawing and hence makes it easier to understand in terms of scale, details of the location and impact.
- This type of visualization can open the door to public participation as well, since this depicts the design in a realistic format, it becomes easy for the policymakers and stakeholders to better understand the proposal and thus give feedback or suggestions.
- Further advances in this field can also lead to the fusion of AR (Augmented Reality) and AI (Artificial Intelligence) into 3-D to predict and display future conflict points by using the CVC data and future projections.

In summary, this can be reckoned as an efficient tool for road development projects just like in other industries to showcase the design in a hyper-realistic format.

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Proactive Safety Assessment at Unsignalized T-Intersection Using Surrogate Safety Measures: A Case Study of Bhopal City

C. Noor Mohammed Parvez, Pritikana Das, and Dungar Singh

Abstract Every year, road accidents cost billions of dollars and damage millions of people around the world. Among all critical points, intersections pose special safety concerns because of the high probability of critical conflicts resulting from unsafe driver actions and maneuvers. As a result, assessing road safety, particularly at uncontrolled T-intersections in mixed traffic, becomes critical. Over the last few decades, Surrogate Safety Measures (SSM) have attracted a lot of study attention for analyzing road safety issues. These approaches are designed to be proactive, that do not rely on collisions, and require shorter observation time periods to create acceptable safety assessments. In this study, two uncontrolled intersections in Bhopal City were chosen as research areas for safety evaluation by adopting the Surrogate Safety Assessment approach. The study focused on crossing conflicts among right-turning vehicles and through traffic as they are considered severe among other conflicts. Post-Encroachment Time (PET) and conflicting vehicle speed of through traffic are used to determine critical conflicts. Further, Encroachment Time (ET) is taken as surrogate indicators to identify severity levels of right turning movements using the clustering technique.

Keywords Surrogate safety measures · PET · Critical conflict · Severity level

1 Introduction

Road safety has always been measured using accident data, which is essentially a reactive technique, despite the fact that this method has time and efficiency limitations. According to past literatures on traffic conflict techniques, employing surrogate safety data provides for a faster evolution of safety than using long-term accident data. At any given uncontrolled intersection, a large number of vehicle-to-vehicle and vehicle-to-pedestrian interactions could result n a crash. Road intersections are

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traffic merging points and hence are prone to accidents [7]. Within the intersection category, T junction accounts for the largest share of accidents, persons killed and injured [7]. As a result, assessing current road safety indications, particularly at unsignalized T-intersections in mixed traffic, becomes critical.

The majority of traffic safety assessments are based on the analysis of historical accident data, which is reactive in character; it's as if they're waiting for a road accident to happen before implementing their remedies. Furthermore, researchers suggested a proactive approach based on surrogate safety measures (SSMs) as a new method for assessing collisions at signalized and non-signalized junctions. The key benefit of this method is that it can help forecast the frequency of an upcoming road crash due to poor road geometry caused by the aforementioned variables, therefore serving as a more efficient and reliable proximate measure of traffic safety.

This study focuses on proactive safety evaluation of T-intersections in Bhopal city. Suitable SSM parameters have been identified from the literature to assess the safety at uncontrolled Intersections for right-turning conflicts. Surrogate safety indicators namely Post-Encroachment Time (PET), Conflicting Speed, and Encroachment Time (ET) are considered in this study. Percentage of critical conflicts observed based on PET and conflicting speed. Severity levels of safety measures have been defined using the clustering technique. The study outcomes will be useful for field engineers, planners and decision-makers to understand the present scenario and to provide safety appropriate safety measures.

Definitions and Terminology

1. Post-Encroachment Time (PET): Time-lapse between the end of encroachment of the turning vehicle and the time that the through vehicle actually arrives at the potential point of collision. The value is recorded in seconds [1].
2. Conflicting Speed: Speed of the through vehicle involved in the conflict event with a right-turning vehicle.
3. Encroachment Time (ET): Time duration during which the turning vehicle infringes upon the right-of-way of the through vehicle. This value is expressed second Allen [1].

2 Literature Review

In a country with a large population, such as India, traffic safety is still a reactive strategy. However, such an analysis is typically performed as an "after-thought" rather than proactively. Safety evaluation has historically been based on police-reported crash data in order to decrease crashes. The analysis of traffic crash data can be useful to understand the general pattern of crash occurrence and to identify the primary contributory variables that can be useful in implementing necessary countermeasures. However, in addition to the other limitations associated with the traditional technique, the non-availability of accident data and inaccurate information about the crash pattern and location are all too typical in developing countries. To overcome this drawback, some studies have suggested the use of short-term, indirect traffic safety measures that are 'proactive' in character and can be utilized instead of historical collision data for a more reliable and faster safety assessment [1]. As a result, a surrogate approach known as "conflict analysis" was created to overcome the paucity of previous accident and exposure data.

In a majority of research, the PET threshold was used, either at random or by taking into account the perception-reaction time associated with Stopping Sight Distance (SSD). Babu and Vedagiri [12] in their study recorded PET values and the speeds of their related conflicting through traffic to observe conflicts. Taking mixed traffic into account, Critical conflicts were discovered by Paul and Ghosh [8], Babu and Vedagiri [13] utilized critical speed, which was established using two surrogate safety indicators, speed of conflicting vehicles and PET. Pawar et al. [4] used PET as a surrogate safety measure to investigate the impact of intersection control measures (SSM). By collecting data from ten crossings, Paul and Ghosh [9] estimated a suitable PET threshold for classifying critical conflicts in a highly heterogeneous traffic environment. And correlated PET values with Crash Data for considering different classes of vehicles and observed PET threshold is 1 s. The intersections were ranked based on the cumulative number of PETs and accompanying crash data.

The critical speed parameter, defined by Paul and Ghosh [8], is derived using the idea of braking distance and is used to identify important conflicts. It was discovered that average PET values at various locations range between 2.13 and 3.16 s. Larger proportions of conflicts that are critical were observed when with right-turning HVs. The Required Deceleration Rate for every conflict is determined as v/2PET from braking distance. Based on these critical conflicts, PET and RDR are determined and ranked 3 T-Intersection. Babu and Vedagiri [13] proposed critical speed to identify critical conflicts which are calculated based on the braking distance concept for the particular critical value. They found that right-turning light motorized vehicles (LMVs) such as auto-rickshaws, cars, and minibuses are more vulnerable than large vehicles (buses and trucks) and 2W. Babu and Vedagiri [12] identify important conflicts, with the term critical speed has been proposed. The braking distance concept is used to establish the critical speed for a certain PET value. There is a substantial percentage of observed confrontations that are critical at the crossing, according to their findings. This shows that right-turning vehicle drivers are willing to take chances and accept smaller gaps in through traffic.

3 Study Methodology

The methodology of a research project outlines the complete work process and plan for achieving the research project's goal. It is a methodical process to achieve the goal of this study. The adopted methodology for the study is shown in Fig. 1. Various surrogate safety measures have been identified from past studies, utilized and tested. For this study, Post-encroachment Time, Difference in conflicting velocities, and Encroachment time were considered for safety assessment. PET is a quantitative

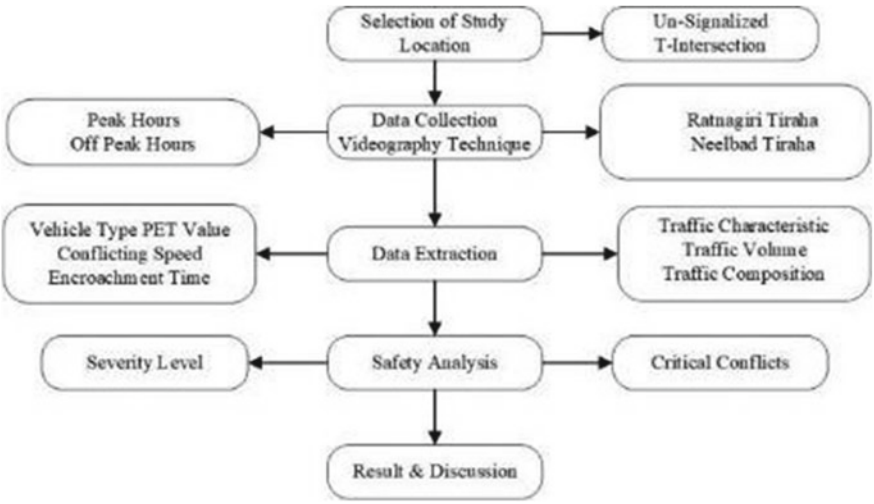

Fig. 1 Methodology flowchart

method for determining the state of a conflict. Because traffic indicators such as stop and yield signs are frequently absent at uncontrolled intersections, drivers have little control over their approaching speed. At intersections, the higher approaching speed of vehicles contributes to the severity of conflict that results in a collision. Because PET alone cannot judge, severity of a conflict, the approaching speed of conflicting through traffic, is considered in determining the severity and frequency of a conflict and evaluating the intersection's safety. The extracted data is used for analysis from which critical conflicts and clustering were obtained.

4 Data Collection and Extraction

The traffic data was obtained using a videography technique on a working day in October 2021 under fair weather conditions at the subject's un-signalized T-intersections. Road inventory and traffic volume details of selected intersections are also collected and shown in Table.1. Two uncontrolled intersections were selected based on having different geometric and traffic characteristics, including the presence of high-rise buildings near the location to capture data effectively, variable traffic demand at a different site to get more variations in safety indicators, vehicles travel at desired speed with less obstruction to flow, having both commercial and residential land use. Further, to track the movement of turning vehicles, the conflict area is divided into grids of 3.5 m × 3.5 m squares, with a lane width of 3.5 m, then overlaid with Kinovea software. Data extraction of 1 h (10 am−11 am) has been performed at selected study locations to evaluate safety in this study. The time delay between the offending vehicle (turning vehicle) leaving the conflict grid and the

Table 1 Road inventory and volume details at two intersections

Study location	No. of lanes in approaches and departures		Width of approaches/ departures (m)		Traffic volume (veh/hr)		
	Major	Minor	Major	Minor	Through	Right turn	Left turn
Ratnagiri Tiraha	6	6	3.5	3.5	2311	1126	512
Neelbad Tiraha	6	2	3	3.5	1609	477	493

conflicting vehicle (opposite through vehicle) entering the respective conflict grid is used to calculate PET values. The speeds of conflicting vehicles are also calculated by recording the time it takes to travel the distance between grids (three to four grids). PET values and the speeds of related conflicting vehicles along with the type of right turning and conflicting vehicles are noted. Similarly, ET is also extracted from video using Kinovea.

5 Data Analysis

5.1 General

PET and conflicting speeds of through traffic are used to determine critical conflicts for right turning and through traffic conflicts. Further, Encroachment Time (ET) are taken as surrogate indicators to identify the severity level of through and right turning movements respectively using the clustering technique.

5.2 Descriptive Statistics

The following Table 2 shows the descriptive statistics of the extracted data at three study locations namely Ratnagiri Tiraha, and Neelbad Tiraha. The aggregate mean, standard deviation, variance of the SSM indicators evaluated to assess the safety of 3-legged uncontrolled crossings in Bhopal City are shown in the descriptive statistics.

5.3 Distributions of Conflicts for All Right-Turning Vehicles with Through Traffic

For a conflict with such PET value and speed of the conflicting vehicle, when right-turning vehicle just left the conflict area, the conflicting through vehicle is at a distance

Table 2 Descriptive Parameters of SSM indicators at study locations

Parameter	N	Range	Mean	Standard deviation	Variance
Ratnagiri Tiraha					
PET	896	10.40	1.432	1.217	1.481
Conflicting speed	896	42.038	18.433	6.911	47.758
ET	1062	33.050	6.153	3.482	12.123
Neelbad Tiraha					
PET	404	18.00	4.07	3.47	15.546
Conflicting speed	404	39.586	20.329	6.571	43.178
ET	396	29.400	4.283	3.521	12.396

Note N indicates Number of observations

equal to PET times the conflicting vehicle's speed (PET × conflicting vehicle's speed). The conflict is not critical if this distance exceeds the stopping distance required for the conflicting vehicle's speed. Conflict is critical if the distance is shorter than the stopping distance required. To distinguish between critical and non-critical conflicts, the distance available is equated to braking distance [12].

Because opposing vehicle drivers have already reacted to a crossing manoeuvre, the perception of distance was neglected. The formula v2/2gf is used to compute the braking distance 'd,' where v is the opposing vehicle's speed in m/s, g is gravity acceleration in m/s^2, and f is the coefficient of friction between the road surface and tyre. The critical speed for that PET value is computed using PET* 2 gf, which is calculated by multiplying the available distance by the braking distance. Using this method, critical speeds for specific PET levels are computed using g = 9.81 m/s^2, and coefficient of friction = 0.35.

$$\text{Available distance} = \text{Braking distance}$$

$$\text{V x PET} = \text{V}^2/2\text{gf}$$

$$\text{Critical speed, V} = 2\text{gf} \times \text{PET}$$

In the current study, this concept was applied to assess the safety of an unsignalized intersection. To detect traffic conflicts, PET levels and speeds of linked conflicting through vehicles are utilized. Critical conflicts are calculated using the PET value, associated critical speed, and speeds of conflicting vehicles. If the conflicting speed is greater than the matching critical speed for a conflict with a defined PET value, it is a critical conflict.

The PET value of each conflict, as well as the speed of the opposing car, influence the outcome. This study considers PET values higher than 0 s and less than 6 s since many recorded conflicts with negative PET readings have slower speeds. At two crossings with PET ranges ranging from 0 to 6 s, a total of 896, and 404 conflicts at

Ratnagiri Tiraha, and Neelbad Tiraha were discovered. As indicated in Tables 3 and 4, the observed conflicts are segregated based on PET values that are categorized into 12 categories with a 0.5 s increment. The Lower Limit (LL) of PET for each of these categories is used to identify critical speeds. The distribution of these conflicts is also given in Tables 3 and 4, which is separated by the type of right-turning vehicle.

A brief summary of critical conflicts of critical conflicts at study sites is shown in Table 5. A total of 47.3%, and 19.06% of conflicts were found critical at Ratnagiri Tiraha, and Neelbad Tiraha, respectively. At Ratnagiri, Tiraha conflicts involving cars are found to be at a higher risk with 51.2% conflicts involving cars being critical. Whereas, at Neelbad Tiraha Two-wheelers are at higher risk with 24.66% conflicts involving Two-wheelers being critical.

5.4 Determination of Severity Levels Using Clustering Technique

Encroachment Time (ET) are chosen as safety indicators to define severity levels of right turning two intersections. Encroachment time was extracted from video-graphic data using Kinovea software as the time difference between entering and leaving time stamps in through traffic path by right-turning vehicles (as given in Eq. 1). Which indicates time spent by right-turning traffic in through traffic path [1].

$$\mathrm{ET(Rt)} = \mathrm{t_{exit}(Rt)} - \mathrm{t_{entry}(Rt)} \tag{1}$$

where ET(Rt) is Encroachment of Right turning (Rt) vehicle.

The severity of probable road crashes will increase with an increase in the value of ET for right turning. In order to classify the severity of probable road collisions based on the ET, all the values ET are grouped using the clustering technique Cluster analysis is the process of categorizing items based on data in the data set that describes their relationships. In this study, the K-means clustering is used as a clustering technique. K-means clustering is a well-known hard partitioning approach that is particularly useful for forming small clusters from large datasets. The k-means function divides the observed data into k mutually exclusive clusters and provides a vector of indices that indicate which of the k clusters each observation belongs to. After classifying the data, the silhouette index was used to validate the results of each clustering technique.

The cluster analysis and validation were carried out using MATLAB software. Finally, the range and threshold values of 3 clusters using the k-means clustering technique for ET are presented in Table 6. The last column of the table shows average silhouette index for all clusters. Further, three severity levels are developed based on obtained threshold values from K-mean clustering as Severity Level (SL) A, B, and C which denote Safe, Moderately Safe, and Unsafe conditions of road users, respectively, as shown in Table 7. The frequency of probable road crashes for unsafe

Table 3 Distributions of conflicts for right-turning with through traffic at Ratnagiri Tiraha

PET (sec)		Critical speed kmph	Ratnagiri Tiraha		Two-Wheeler		Three-Wheeler		LCV		Car	
LL	UL		Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)
0	0.5	0	0.00	0.00	15.55	15.55	1.90	1.90	3.13	3.13	7.27	7.27
0.5	1	12.4	17.00	18.90	26.06	22.15	3.80	3.36	2.35	2.24	10.51	8.50
1	1.5	24.7	28.30	24.61	16.00	3.47	3.47	0.22	4.36	0.00	13.09	5.37
1.5	2	37.1	20.25	3.58	9.28	0.11	0.78	0.00	1.45	0.00	4.25	1.12
2	2.5	49.4	10.51	0.22	7.83	0.00	0.89	0.00	1.34	0.00	4.14	0.00
2.5	3	61.8	9.96	0.00	3.69	0.00	0.67	0.00	0.45	0.00	1.68	0.00
3	3.5	74.2	4.59	0.00	2.46	0.00	0.45	0.00	0.34	0.00	0.78	0.00
3.5	4	86.5	2.80	0.00	2.68	0.00	0.45	0.00	0.00	0.00	0.89	0.00
4	4.5	98.9	2.91	0.00	0.78	0.00	0.00	0.00	0.00	0.00	0.22	0.00
4.5	5	111.2	0.89	0.00	0.67	0.00	0.11	0.00	0.00	0.00	0.34	0.00
5	5.5	123.6	0.67	0.00	0.45	0.00	0.00	0.00	0.11	0.00	0.34	0.00
5.5	6	135.9	0.45	0.00	0.45	0.00	0.00	0.00	0.34	0.00	0.11	0.00
				47.32	85.91	41.28	12.53	5.48	13.87	5.37	43.62	22.26

Table 4 Distributions of conflicts for right-turning with through traffic at Neelbad Tiraha

PET (sec)		Critical speed kmph	Neelbad Tiraha		Two-Wheeler		Three-wheeler		LCV		Car	
LL	UL		Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)	Conflicts (%)	Critical conflicts (%)
0	0.5	0.00	0.00	0.00	8.66	8.66	1.98	1.98	1.73	1.73	2.48	2.48
0.5	1	12.4	9.90	9.90	8.17	7.92	0.74	0.74	0.50	0.50	2.72	1.49
1	1.5	24.7	8.17	7.92	7.67	0.99	8.17	0.50	8.42	0.00	16.83	0.25
1.5	2	37.1	8.17	0.99	7.67	0.25	0.74	0.00	1.49	0.00	1.24	0.00
2	2.5	49.4	7.67	0.25	5.69	0.00	1.24	0.00	0.99	0.00	1.49	0.00
2.5	3	61.8	7.43	0.00	6.19	0.00	0.74	0.00	0.25	0.00	2.23	0.00
3	3.5	74.2	6.19	0.00	6.68	0.00	0.50	0.00	0.50	0.00	1.73	0.00
3.5	4	86.5	7.18	0.00	7.67	0.00	0.50	0.00	0.74	0.00	1.24	0.00
4	4.5	98.9	7.43	0.00	2.97	0.00	0.25	0.00	0.50	0.00	0.25	0.00
4.5	5	111.2	3.96	0.00	4.46	0.00	0.50	0.00	0.25	0.00	0.74	0.00
5	5.5	123.6	4.21	0.00	3.47	0.01	0.25	0.00	0.50	0.00	0.99	0.00
5.5	6	135.9	4.21	0.00	2.97	0.00	0.74	0.00	0.00	0.00	0.99	0.00
				19.06	72.28	17.83	16.34	3.223	15.8	2.23	32.92	4.21

Where, UL = Upper Limit of PET, LL = Lower Limit of PET, and LCV = Light Commercial Vehicle

Table 5 Brief summary of critical conflicts

Study location		Total	Two wheelers	Three wheelers	LCV	Car
Ratnagiri Tiraha	Conflicts	896	770	112	124	391
	Critical conflicts	424	370	49	48	200
	Percent of critical conflicts in total conflicts	47.3	48.05	43.75	38.7	51.2
Neelbad Tiraha	Conflicts	404	292	66	64	133
	Critical conflicts	77	72	13	9	17
	Percent of critical conflicts in total conflicts	19.06	24.66	19.70	14.1	12.8

cases based on their ET for selected intersections using K-means is presented in Table 8. The results from Table 8 show that the percentage of unsafe right-turn vehicles is higher at Ratnagiri Tiraha with 5.65% whereas at Neelbad Tiraha is 1.52%. So, the probability of crashes involving right-turning vehicles may be seen more at the former than at the later.

Table 6 Results of K-means clustering along with Silhouette Index

Study location	Parameter	Range		Thresholds
		Min	Max	
Ratnagiri Tiraha	ET	0.80	33.85	6.525, 12.5
Neelbad Tiraha	ET	0.48	29.88	5.68, 16.14

Table 7 Classification of encroachment time (in sec) for severity of right turning traffic

Study location	Threshold Values for ET using K-means clustering	Severity level	Safety of right-turning movement
Ratnagiri Tiraha	< 6.525	A	Safe
	6.525–12.5	B	Moderately safe
	>12.5	C	Unsafe
Neelbad Tiraha	< 5.68	A	Safe
	5.68–16.14	B	Moderately safe
	>16.14	C	Unsafe

Table 8 Percentage of vehicles fall under the unsafe case, i.e. when SL = C

Parameter	Ratnagiri Tiraha	Neelbad Tiraha
ET	5.65	1.52

6 Conclusion

This provided a study that developed a methodology for evaluating a surrogate safety indicator, PET, and conflicting speed to measure traffic safety at three-arm uncontrolled intersections. Further, severity levels to assess the safety right turning movement using ET.

PET threshold values are used to determine critical conflicts. However, at intersections with mixed traffic and varying speeds, relying solely on PET to assess safety is insufficient. As a result, conflicts are observed in the current study employing two surrogate indicators PET, and related conflicting vehicle's speed. To determine critical conflicts at intersections, the critical speed is recommended. The braking distance concept is used to find out the critical speed for a certain PET value. There is a substantial percentage of observed conflicts at the intersection are critical, according to the findings. This demonstrates that right-turning vehicle drivers are willing to take chances and accept short gaps in through traffic paths, which is unsafe. At Ratnagiri Tiraha conflicts involving cars are found to be at a higher risk with 51.2% of conflicts involving cars being critical. Whereas, at Neelbad Tiraha Two-wheelers are at higher risk with 24.7% of conflicts involving Two-wheelers being critical. This could be owing to their high proportion of volumes and high speed.

Further, namely Encroachment Time (ET) is used to define severity levels of right turning vehicles. Three unsafe severity levels were developed namely less unsafe, moderately unsafe, and highly unsafe using the K-means clustering technique. Among two intersections Ratnagiri Tiraha found it unsafe for right-turning traffic with 5.65% of vehicles falling under severity Level C, and at Neelbad Tiraha it is observed as 1.52% of vehicles falling under severity level C. The proposed approach provides a dependable method to identify unsafe crossings, and unsafe movements, and reduce accidents by executing preventive management measures that exist in developing nations. Several countermeasures like providing speed breakers, speed humps, and speed tables can be provided to reduce ET and result in safer movements. An increase in the proportion of unsafe right turning over the years may also indicate the importance of implementing traffic control measures.

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Ranking-Based Methodology for Prioritization of Critical Pedestrian Infrastructure in and Around the Market Area: A Case Study of Aminabad Market, Lucknow

Haroon Rasheed Khan, Mokaddes Ali Ahmed, and Manish Dutta

Abstract Market areas are one of the busiest urban areas that experience heavy pedestrian footfall. However, studies on assessing existing pedestrian infrastructure are limited to market areas. This study addresses the gap by ranking the roads in and around the market area based on factors related to the road environment so that critical pedestrian infrastructures can be identified for improvement. Aminabad, a market area in Lucknow, Uttar Pradesh, has been taken as the case study area. As a renowned market centre and a focal point for tourism and cultural activities, many populations use walking as a transportation mode for their daily commute into the market area. The results show that the border roads of the market area scored an excellent rank in terms of safety and mobility due to the presence of proper pedestrian facilities compared to the inner streets. Based on the result, market roads have been divided into three categories: first, whose condition is good and does not need any repair; second, which requires a little care; last, which are critical and need immediate action.

Keywords Pedestrian · Built environment · Walkability · AHP · TOPSIS · Market area

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1 Introduction

As one of the world's largest democracies, India is quickly advancing toward its goal of becoming one of the leading economic powers. As a result, urbanization has increased, and the number of people moving toward cities has also increased. Due to economic growth, vehicle ownership has also increased, leading to alarming traffic congestion on roads. However, to alleviate the heavy traffic congestion, roads are widened, and overpasses have been constructed, rather than trying to reduce the traffic on roads or to promote sustainable modes of transportation. To address the significant concerns of increased traffic, it is imperative to encourage walking as a preferred mode of transportation, at least for shorter trips, and public transit for longer distances. So, encouraging people to walk on foot might significantly cut down the number of trips being performed by automobiles in countries like India, where congestion is exceptionally high because of its dense population, shrinking spaces, and mixed land uses.

Walking is one of the most basic and eco-friendly modes of transportation; everyone uses it for their daily commute. However, the ease of convenience and comfort offered by motorized vehicles has led people to depend on these modes even for shorter trips, contributing to traffic congestion. Furthermore, pedestrians are the most vulnerable road users and are more likely to be involved in an accident than other road users. Although a considerable proportion of trips are performed on foot in India, it needs proper pedestrian facilities. Traffic management of vehicular traffic is usually the primary focus of highway design in developing countries, with minimal emphasis on pedestrian facilities, which is a significant concern. Also, to cope with the rising volume of vehicular traffic, roadways are being widened at the cost of pedestrian facilities. As a result, the lack of sufficient pedestrian infrastructure has caused pedestrians to utilize the carriageway designated for automobiles, which not only interrupts traffic but also threatens their safety. This problem must be addressed by improving the design of pedestrian amenities so that more people will choose to walk instead of driving.

Accordingly, this research paper examines the condition of the road environment in a market area regarding safety and mobility by assessing the various road environment factors or attributes that influence pedestrian movement. The objectives of this study are (1) to identify the gaps in the existing literature by performing a thorough assessment of the pedestrian walk environment and propose an appropriate methodology, (2) to identify the factors related to the road environment that influence the pedestrian movement in a market area, and (3) to prioritize the various roads of the market area based on the attributes of the existing road environment using AHP and TOPSIS. Section 2 presents a brief literature review on pedestrian walk environments and the various research work using the Likert scale for ranking. Section 3 explains the study's overall methodology, including a brief overview of AHP and TOPSIS. Section 4 describes the study area and the data collection techniques. Section 5 presents the result and analysis starting with a preliminary analysis and ranking various roads in the market area. Section 6 highlighted the findings, inferences, and future scopes that may provide necessary input to urban planners.

2 Literature Review

A range of methodologies, including both qualitative and quantitative approaches, have been described in the literature. It is necessary to review the existing literature to better understand the research problem.

Numerous studies have been conducted to prioritize transportation-related infrastructure projects [1, 2]. However, most of these investigations focused on road infrastructure development, maintenance, and rehabilitation [3, 4]. Only a few studies have been found in prioritizing transportation-related infrastructure [5, 6]. Prior research has shown that priority choices are driven mainly by several associated and competing decision factors, making it difficult for decision-making [3]. To address this, thorough consideration is needed to establish an appropriate set of regulating standards to overcome the abovementioned characteristics.

Multicriteria decision-making (MCDM) approaches, such as analytic hierarchy process (AHP) [7], fuzzy AHP [2], and analytical network process [8], have been found to be systematic approaches [9, 10]. The prioritizing of attributes has been accomplished using several different multi-attribute decision-making (MADM) techniques. Such methods include grey relational analysis (GRA), analytical hierarchy process (AHP), elimination and choice expressing reality (ELECTRE), technique for order preference by similarity to ideal solution (TOPSIS), relative to an identified distribution integral transformation (RIDIT), etc. Among these, TOPSIS, one of the most extensively adopted MADM methods [11], is used in this study along with AHP. The TOPSIS method uses a derived scalar quantity for evaluation, giving an edge over other MADM approaches. Scalar value estimation is the ability to use a simple mathematical equation to compare the relative performance of the best and worst alternatives [12].

Researchers looked at how sidewalks, crosswalks at intersections, and midblock crosswalk affects the pedestrian level of service [13]. Few studies have developed a qualitative approach that considers pedestrian safety, security, continuity, comfort, and convenience. In some studies, pedestrian level of service was assessed using other factors such as traffic volume, surface quality, and obstructions. Few studies have focused on the safety considered by the adjacency to traffic flow and the level of segregation from the traffic. Most of the existing studies are based on user perception. Another similar kind of work has been found to assess the factors that would promote walking using people's perception of identified factors [14]. Accordingly, a 9-pointer Likert scale questionnaire was constructed to evaluate user perception. Aesthetics and amenities, signage and street furniture, personal safety, and separation from traffic flow are some attributes included in the survey. Analysis and assessment of road segments were conducted using the analytical hierarchy approach [15]. Very little information was found in the existing literature that prioritized road environments for pedestrian movement using expert opinion-based data.

Limited research work has been found on the pedestrian-built environment in the country. Assessment and prioritizing attributes based on expert opinion would assist in creating a pedestrian-friendly built environment. Road type, land use, average building frontage/setbacks, and road connectivity are the essential attributes found in the literature. The attributes are further divided into sub-attributes to determine the relative weights and effect of the attribute on the road environment, which are discussed in Sect. 4.

3 Methodology

A detailed methodological framework for the study is shown below in Fig. 1.

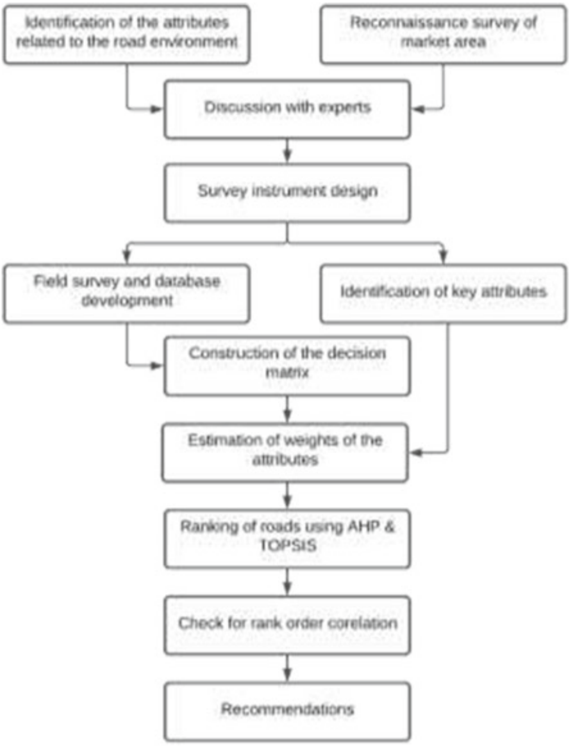


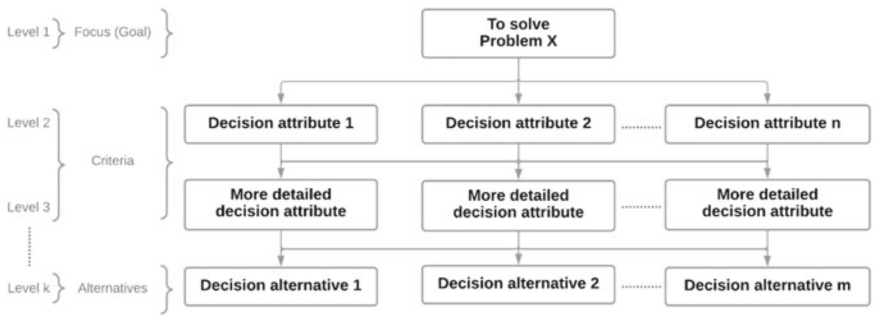
Fig. 1 Methodological framework

3.1 Theoretical Background of Analytical Hierarchy Process (AHP)

In 1980, Saaty introduced the analytical hierarchy process (AHP), which has been widely used in most MCDM-related problems [16]. Educational institutions, engineering firms, government agencies, industrial sectors, and a variety of other sectors are using the AHP for decision-making [17]. Due to its simplicity, ease of use, and high versatility, it is widely applicable. The process of analytical hierarchy process is discussed in detail below.

Step 1: Model development

The hierarchy is established by dividing the issues into a series of interconnected decision points.



Step 2: Construction of pairwise comparison matrix among all the available attributes for that specific problem and assign the weights among the attributes using the fundamental scale given by Saaty.

$$A = \begin{array}{c} \\ 1 \\ 2 \\ 3 \\ \vdots \\ n \end{array} \begin{array}{c} \begin{array}{ccccc} 1 & 2 & 3 & \dots & n \end{array} \\ \begin{bmatrix} w_1/w_1 & w_1/w_2 & w_1/w_3 & \dots & w_1/w_n \\ w_2/w_1 & w_2/w_2 & w_2/w_3 & & w_2/w_n \\ w_3/w_1 & w_3/w_2 & w_3/w_3 & & w_3/w_n \\ \vdots & & & & \vdots \\ w_n/w_1 & w_n/w_2 & w_n/w_3 & \dots & w_n/w_n \end{bmatrix} \end{array}$$

Step 3: Calculation of relative weights of all the attributes using the formula using Eq. (1)

$$\begin{aligned} W_{A_1} &= \frac{A_1}{\sum A_1 + A_2 + A_3 + \ldots + A_n} \\ W_{A_2} &= \frac{A_2}{\sum A_1 + A_2 + A_3 + \ldots + A_n} \\ \vdots \quad & \vdots \\ W_{A_n} &= \frac{A_n}{\sum A_1 + A_2 + A_3 + \ldots + A_n} \end{aligned} \tag{1}$$

Step 4: The consistency check, using the consistency index formula as shown in Eq. (2).

$$ConsistencyIndex(CI) = \frac{\lambda_m - n}{n - 1} \tag{2}$$

Step 5: The overall priorities were derived by aggregating the relative weights of various attributes as obtained from Step 3 to get the final decision of the problem, which is used as a rating for multiple decisions (or selections) to achieve the most general objective of the problem.

3.2 Theoretical Background of Technique of Order Preference Similarity to the Ideal Solution (TOPSIS)

Hwang and Yoon introduced TOPSIS in 1981 to help select the best option based on a limited set of criteria. The typical TOPSIS technique attempts to choose alternatives with the longest distance from the negative ideal solution and the shortest distance from the positive ideal solution simultaneously [18]. TOPSIS is a well-known MCDM approach that has received much attention from academicians and practitioners. The detailed process of evaluating TOPSIS is discussed below.

Step 1: Objective determination and identification of pertinent evaluation attributes

Step 2: Construction of normalized decision matrix as shown in Eq. (3)

$$r_{ij} = \frac{x_{ij}}{\sqrt{\sum_{j=1}^{m} x_{ij}^2}} \tag{3}$$

Here, x_{ij} and r_{ij} are original and normalized scores of the decision matrix, respectively.

Step 3: Check for the relative importance weights of different attributes with respect to the objective, such that the summation of all the weights should be equal to one.

$$\sum_{j=1}^{n} w_j = 1$$

Step 4: Construction of weighted normalized decision matrix using formula as shown in Eq. (4).

$$V_{ij} = \mathrm{w}_j * r_{ij} \tag{4}$$

Here, w_j is the weight for j criterion.

Step 5: Determination of positive ideal and negative ideal solutions using Eqs. (5) and (6).

$$A^+ = \left\{V_{ij}^+, \ldots.., V_n^+\right\} Positive\,ideal\,solution \tag{5}$$

Here, $V_{ij}^+ = \left\{max(V_{ij})|i \in I; min(V_{ij})|i \in I'\right\}$

$$A^- = \left\{V_{ij}^-, \ldots.., V_n^-\right\} Negative\,ideal\,solution \tag{6}$$

Here, $V_{ij}^- = \left\{max(V_{ij})|i \in I; min(V_{ij})|i \in I'\right\}$

Step 6: Calculation of separation measures for each alternative using Eqs. (7) & (8).

$$S_j^+ = \sqrt{\sum_{i=1}^{n} (V_{ij} - V_j^+)^2} Positive\,ideal\,solution \tag{7}$$

$$S_j^- = \sqrt{\sum_{i=1}^{n} (V_{ij} - V_j^-)^2} Negative\,ideal\,solution \tag{8}$$

Step 7: Evaluation of relative closeness to the ideal solution using Eq. (9).

$$C_i^* = \frac{S_j^-}{S_j^+ + S_j^-} \tag{9}$$

Step 8: Based on the obtained value, sort the value of C_i^* in decreasing order and rank it accordingly.

3.3 Spearman's Rank Correlation Coefficient

In this research, two different ranking methods have been used to determine the ranking of various roads in the market area. So, deciding the final rank order derived from the specific ranking methods might be challenging. Therefore, a statistical test known as Spearman's rank correlation coefficient (ρ) has been evaluated to determine the relationship between rank-order results.

$$\rho = 1 - \frac{6\sum d_i^2}{n(n^2 - 1)} \tag{10}$$

Here, d_i = difference in the paired ranks.

n = number of roads in the market area.

4 Study Area and Data Collection

4.1 Area of Study

This research aims to ascertain the pedestrian-built environment towards the factors affecting their safety and mobility regarding the prevailing pedestrian amenities in Lucknow, the capital of Uttar Pradesh. As the capital city, Lucknow has an intriguing demographic mix. It not only attracts a large number of tourists but also generates a great deal of business. As a renowned market centre and a focal point for tourism and cultural activities, many populations use walking as a transportation mode for their daily commute into the market area. Despite the high volume of pedestrian travel, the city lacks adequate pedestrian infrastructure. Most of the existing pathways have been encroached on by hawkers or blocked by poles or vehicles parked on the sidewalks; this compels pedestrians to use the carriageway, which is already congested due to high vehicular traffic. An on-street survey based on existing attributes of the road environment was conducted in one of the central market areas of Lucknow (Aminabad) to ascertain the existing condition of the walking environment that affects the safety and mobility of pedestrians. Figure 2 shows the street view of the Aminabad market.

4.2 Data Collection

After an intensive analysis of the relevant literature, a list of factors related to the road environment was compiled. From that list, a set of the most significant factors relevant to the typical Indian road context has been chosen. The data collection is

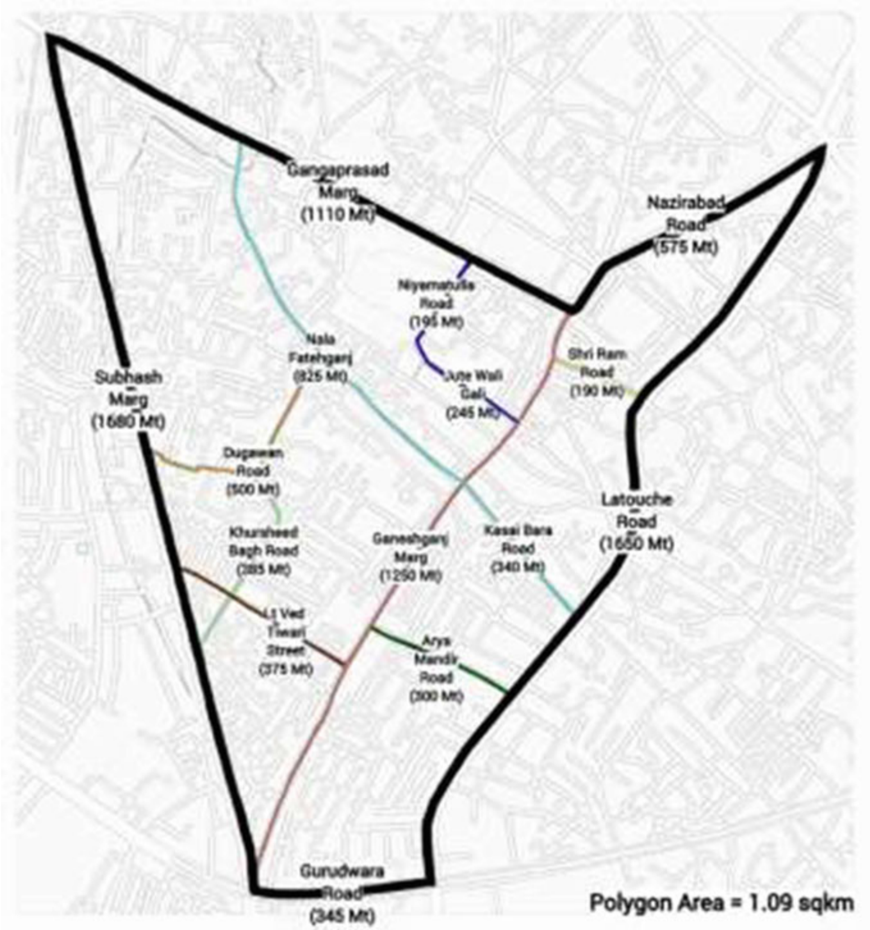


Fig. 2 Street view of Aminabad market

an auditor-based survey on a 5-pointer Likert scale on those attributes of the road environment. After reviewing the existing literature, this Likert scale was designed, but after collecting the field data, it was realized that most of the attributes fit in a 3-pointer Likert scale. So, we didn't push it to the limits to fit our data into the 5-pointer Likert scale, and we opted for the scale as it is, either a 4-scale or 3-scale. So, based on the existing literature and experience gained from the reconnaissance survey, a list of attributes and sub-attributes was created, and the process of selecting the attributes and sub-attributes is called survey instrument design. Two auditors have been assigned to collect the primary data of the market area based on the predefined attributes of the existing road environment. After collecting data, the irrelevant attributes are removed in the first stage of data screening and sorted according to our objectives. Entropy Weight Method (EWM) has been used to evaluate each attribute's weight related to the road environment.

5 Results and Analysis

5.1 Preliminary Analysis

Preliminary data analysis has been presented in this section. The market area of Aminabad has been segmented into smaller parts by selecting the various roads, as shown in Fig. 2. Five significant roads surrounding the market area and ten minor roads inside the market area are selected for the study. The inner roads have been selected based on two parameters–(1) the most famous roads, and (2) one end of the road should be connected to any of the border roads of the market area except for Jute Wali Gali, which has been selected due to its high popularity. The area of Aminabad is 1.09 km^2, and the road lengths, along with the road markings of the various roads, are presented in Table 1.

A customized Likert scale has been designed for the study area as per the various attributes of the road environment to assign the weights for sub-attributes. A table with attributes, sub-attributes, their respective scales, and the reason for assigning weights has been presented in Table 2.

Table 1 Road lengths of the market area

Road no	Road name	Road length (m)
1	Subhash Marg	1680
2	Gurudwara Road	345
3	Latouche Road	1650
4	Nazirabad Road	575
5	Gangaprasad Marg	1110
6	Shri Ram Road	190
7	Kasai Bara Road	340
8	Arya Mandir Road	300
9	Niyamatulla Road	195
10	Jute Wali Gali	245
11	Ganeshganj Marg	1250
12	Lt Ved Tiwari Street	375
13	Khursheed Bagh Road	385
14	Dugawan Road	500
15	Nala Fatehganj	825

Table 2 Likert scale of different attributes

Attributes	Sub-Attributes	Likert-Scale	Reason
Road type (As per IRC)	Arterial Road	4	Higher-order Road got a higher Likert Scale [due to better safety and mobility] (because of the wider carriageway; proper traffic lights, and signals; appropriate markings, etc.)
	Sub-arterial Road	3	
	Collector Street	2	
	Local Street	1	
Land use (As per study area)	Residential	3	Higher the traffic areas, lesser the Likert scale (Due to poor mobility and safety)
	Mixed Land Use	2	
	Commercial	1	
Average building frontage/ setbacks (As per national building code of India, 2011)	Wide (more than 9.0 m wide)	3	Likert Scale is directly proportional to the Frontage length
	Moderate (6.0 m to 9.0 m wide)	2	
	Narrow (less than 6.0 m)	1	
Road connectivity (No. of roads connected)	–	–	It will be a numerical value, as the number of roads connected to a particular road

5.2 *AHP-TOPSIS-Based Prioritization of Roads*

EWM has been used to evaluate the respective weights (W_j) of the attributes based on the inputs collected from the field survey using Eq. 11 and the weights are shown in Table 3 The result shows that land use got the highest weight among all the attributes as it is one of the significant parameters of road environment, whereas road connectivity has the least weightage.

$$W_j = \frac{1 - E_j}{\sum_{j=1}^{m}(1 - E_j)} \tag{11}$$

Table 3 Weights of attributes of road environment

	Road type	Land use	Average building frontage/setbacks (m)	Road connectivity
W_j	0.2529	0.2622	0.2561	0.2288

Here, E_j = entropy measure.

Based on the AHP and TOPSIS scores, the prioritization of roads has been evaluated as shown in Table 4. A larger value of AHP or TOPSIS scores for a specific road indicates that the road has been ranked higher based on existing road environment parameters. The result shows that the border roads got the higher rank due to the presence of desirable pedestrian facilities as compared to the inner streets, except Ganeshganj Marg, which brought the second rank due to the presence of a better road environment for pedestrians. The inner roads have got a lower rank due to the unavailability of the proper pedestrian facilities. Spearman's rank correlation has been carried out to check whether the two rankings conform with each other. The correlation coefficient has been found to be 0.996 which is an indication of excellent conformity.

These results lead to several key insights. Firstly, the minor roads inside the market area have received a lower rank due to the poor road environment for pedestrian movement. Secondly, the border roads have got a higher rank due to the availability of proper pedestrian amenities that will help pedestrians to roam in the market area easily and safely. Thirdly, Shri Ram Road, Kasai Bara Road, Arya Mandir Road, Niyamatulla Road, and Jute Wali Gali, these five roads that require immediate attention as they got a low order rank by both the ranking methods. These are the inner streets of the market area having less width and narrow building frontage (>6 m), comparable to the major roads. Moreover, the frontage area has been encroached on by shop owners or hawkers or electric poles, due to which building frontage/setbacks are reduced, leading to traffic jams and safety concerns. Out of the four road environment attributes that are used in this study–road type, land use, and road connectivity cannot be changed. However, average building frontage/setbacks can be adjusted to provide a better road environment for pedestrian movement in the market area. Fourthly, those roads that rank between six and ten will also require minor interventions to enhance overall safety and mobility.

6 Conclusion and Future Scope

The development of pedestrian infrastructure facilities is cost-intensive, like other urban infrastructure. In such cases, urban regions need a proposal for pedestrian infrastructure improvement based on existing road environment data and plan the infrastructure development in a phased manner. Therefore, a new methodological framework has been proposed in this research work which can be used as a tool to build a logical ranking order for market areas to prioritize their improvement needs. Existing road environment attributes are used in the proposed framework to characterize the walking environment of the Aminabad market in Uttar Pradesh. Based on the ranking done by AHP and TOPSIS, Shri Ram Road, Kasai Bara Road, Arya Mandir Road, Niyamatulla Road and Jute Wali Gali are the roads, which require immediate intervention to improve pedestrian safety and mobility.

Table 4 Ranking of roads based on road environment characteristics

Road no	Road	AHP score	TOPSIS score	AHP rank	TOPSIS rank	Remarks
3	Latouche Road	0.83	0.7072	1	1	Good condition Does not require any intervention
11	Ganeshganj Marg	0.785	0.6549	2	2	
5	Gangaprasad Marg	0.721	0.532	3	4	
1	Subhash Marg	0.712	0.6027	4	3	
2	Gurudwara Road	0.673	0.4924	5	5	
4	Nazirabad Road	0.587	0.3511	6	6	Fair condition Requires a little intervention
15	Nala Fatehganj	0.417	0.278	7	7	
12	Lt Ved Tiwari Street	0.379	0.1808	8	8	
13	Khursheed Bagh Road	0.372	0.1598	9	9	
14	Dugawan Road	0.368	0.1491	10	10	
7	Kasai Bara Road	0.353	0.1051	11	11	Critical condition Requires immediate intervention
8	Arya Mandir Road	0.345	0.0825	12	12	
6	Shri Ram Road	0.342	0.071	13	13	
10	Jute Wali Gali	0.327	0.0241	14	14	
9	Niyamatulla Road	0.319	0	15	15	

This research is one of the unique efforts to prioritize roads in a typical Indian environment (market area) based on the existing road environment attributes and demonstrates an application of MCDM techniques like AHP and TOPSIS. Overall, these results might serve as basic suggestions for Indian planners and stakeholders. Although the conclusions are case-specific, the methods exhibited may be used in other cities with comparable road environment conditions. In this study, only road environment attributes have been considered to evaluate the safety and mobility of the market area, but some other attributes like factors related to road traffic can also be incorporated to get a clearer understanding. This work can be escalated by taking other factors related to pedestrian safety and mobility and then the overall market area can be ranked. Similarly, different market areas of the city can be ranked for comparison. The overall rank of a city may be obtained on the aggregation of the ranks of the different market areas.

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Machine Learning Categorization Algorithms for Traffic Conflict Ratings

Pushkin Kachroo, Anamika Yadav, Ankit Kathuria, Shaurya Agarwal, and Mahmudul Islam

Abstract This paper provides a review of unsupervised and unsupervised categorization algorithms using safety surrogate data to predict the severity of traffic conflicts using processed traffic video data at conflict sites as well as human ratings of the conflicts.

Keywords ML categorization · Safety surrogates · Traffic conflict

1 Introduction

This paper presents a review of Machine Learning (ML) algorithms that attempt to solve the categorization problem. There are two types of algorithms for categorization: supervised and unsupervised. In the supervised algorithm, the input and output data is used to train the categorization algorithm to minimize the norm of the error difference of the data output from the model output for the same given input. In the unsupervised case, we use the data input and perform the categorization without the use of the output data. In general, the unsupervised algorithms are used when the output data is not available.

Our study is on the topic of traffic safety where we attempt to categorize the severity of a traffic conflict in terms of categorical levels. In turn, the traffic conflicts are a surrogate safety measure for crashes. Hence, categorization of the traffic conflicts has a relationship to the probability of actual crashes and their severity [1, 2].

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Fig. 1 Safety surrogate data collection

In this paper, we use the various categorization algorithms for traffic conflict severity using the field data and human-derived categorization for supervised learning, and using direct field data for unsupervised learning.

The data was collected in the field at various intersections of different types using fixed cameras as well as using drone videography overhead. The video data went through image processing to obtain vehicle trajectories, which were further processed for obtaining the variables used in the algorithms presented in this paper. Human-trained agents were used for categorizing the traffic conflicts for use in training the supervised algorithms. Figure 1 shows one of the locations from the data collection site.

1.1 Background

Machine Learning (ML) has been studied and deployed in many applications for many decades starting from one of the early publications on the topic [3] and followed by the first learning rule in [4]. These lead to the development of neural networks. Modern ML also uses techniques from multivariate statistics for pattern recognition [5, 6].

Machine learning has been used in many applications in transportation for a while [7] and it is still an actively pursued area [8]. It is also being used in transportation safety area [9]. In this review paper, the authors show what work has been done in using ML for crash predictions.

1.2 Research Gap and Contribution

Traffic accidents can qualify to be categorized as rare event phenomenon [10–12]. This can be seen by dividing the time variable into small increments such as in a few seconds or minutes and then looking at how many intervals involve accidents versus how many have no accidents.

Since in any contiguous data collected on data, most of the data will have no accidents, it is very difficult to build estimation of accidents based on that. Hence, researchers have developed Safety Surrogate Measures (SSM) for accidents such as Time to Crash (TTC) and its various modifications, namely, time-exposed time to collision (TET) and time-integrated time to collision (TIT), post-encroachment time (PET), delta-V (ΔV), relative speeds, accelerations, etc. These SSMs have been used in the models for the prediction of traffic crashes as well as their severities [1, 2].

1.3 Problem Statement

The current ML algorithms concentrate on crash predictions. In this current paper, we use ML techniques to connect some available SSMs to the ratings that have been given by human raters after being trained and watching the same videos that have been processed using software to perform video processing and then obtaining the SSMs automatically. Hence, the aim of this paper is to review the categorization ML algorithms and show how they can be used to mimic human rating method.

2 Algorithms

The general framework of this paper is shown in Fig. 2.

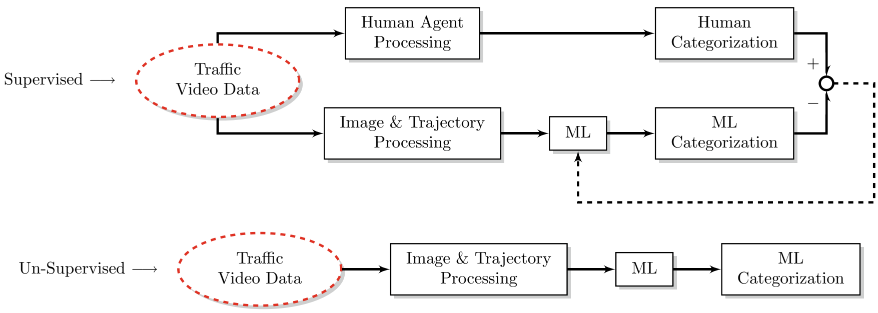


Fig. 2 ML cataegorization algorithms for ratings

2.1 Supervised Algorithms

In this paper, we have used three regression models that fall under the category of supervised algorithms, namely: (1) simple linear regression, (2) multiple linear regression, and (3) logistic regression. The data set we used for testing the models contains various variables such as vehicle type, time-exposed TTC (TETTC), time-integrated TTC (TITTC), TTC Min (TTCM), speed of the vehicles, conflict type, and crash severity ratings. As we tried to predict the crash severity ratings of the incidents, we used the rating variable as the dependent variable for all the models throughout the manuscript.

Simple linear regression tries to predict the quantitative response of the dependent variable (Y) based on a single independent variable (X). The equation for simple linear regression is given as

$$Y \approx \beta_0 + \beta_1 X \tag{1}$$

Here, we fit the model using the TETTC and the models estimated the coefficients as $\beta_0 = 0.34$ and $\beta_1 = 0.15$. The residual standard error for this model is 0.485 and the $R^2 = 0.043$. The regression line for simple linear regression is shown in Fig. 3, which tells us that with the increase of TETTC, the crash severity ratings also increase.

Multiple linear regression uses at least two variables to predict the dependent variable. If we have n distinct independent variables, then the equation for the dependent variable (Y) will be

$$Y \approx \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n \tag{2}$$

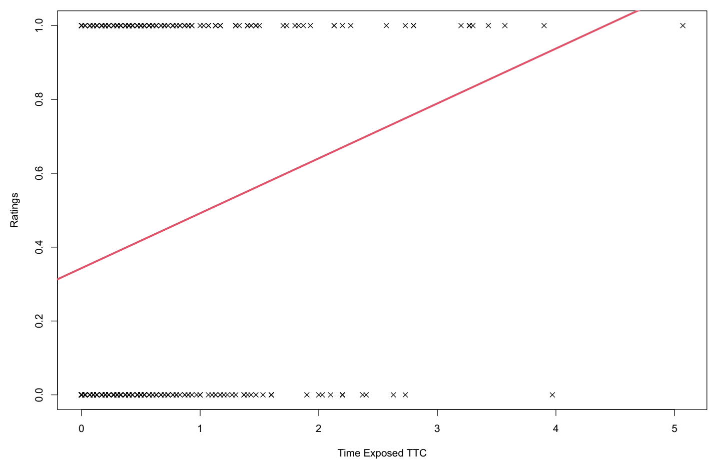


Fig. 3 Regression plot for ratings on time exposed TTC

We used TETTC, TITTC, and the difference between the speed of the vehicles as independent variables to model the multiple linear regression algorithm. After fitting the models, we find the values for the coefficient as $\beta_0 = 0.32$, $\beta_1 = -0.49$, $\beta_2 = 0.145$, and $\beta_3 = -0.003$. The residual value for the model is 0.45 and $R^2 = 0.2$. The correlation between the dependent and independent variables is given in Table 1. Logistic regression predicts the probability of an event occurring based on the independent variables. The probability of whether an event will occur or not can be modeled as

$$p(X) = \beta_0 + \beta_1 X \tag{3}$$

For logistic regression, we used all the variables (vehicle type, vehicle speed, conflict types, TETTC, TITTC, and TTCM) to estimate the probability of the severity ratings of a crash. The prediction of this model is given in Table 2, which shows the model predicts the crash severity ratings at an accuracy of 81%.

Support vector machine is a supervised learning framework that classifies the data using a hyperplane. We have used a linear hyperplane for the classification of the data. We used TETTC and TITTC variables to classify the crash severity ratings. First, we split the data into training and testing as a ratio of 7:3. The training data set contains 70% of the data used for the model's training, and the rest 30% is used for validation. The SVM classification plot is shown in Fig. 4. The trained SVM model is tested on the testing data set. The result shows that the model predicts with an accuracy of 69%. The prediction value is given in Table 3.

An artificial neural network (ANN) is an interconnection of nodes that can map the outputs with the inputs without any information on the relationship among the variables. ANN is typically made of one input layer, one output layer, and one or more hidden layers. In our case, we used vehicle types and speed differences in the input layer and the ratings in the output layer. We used one hidden layer with two

Table 1 Co-relation between dependent and independent variables for multiple linear regression

	TETTC	TITTC	Speed diff.	Ratings
TETTC	1	0.933	0.008	0.209
TITTC		1	−0.207	0.325
Speed diff.			1	−0.188
Ratings				1

Table 2 Prediction made by the logistic regression model

	Predicted value	
Actual value	False	True
0	235	47
1	49	162

Table 3 Prediction made by the SVM model

	Predicted value	
Actual value	0	1
0	76	37
1	8	26

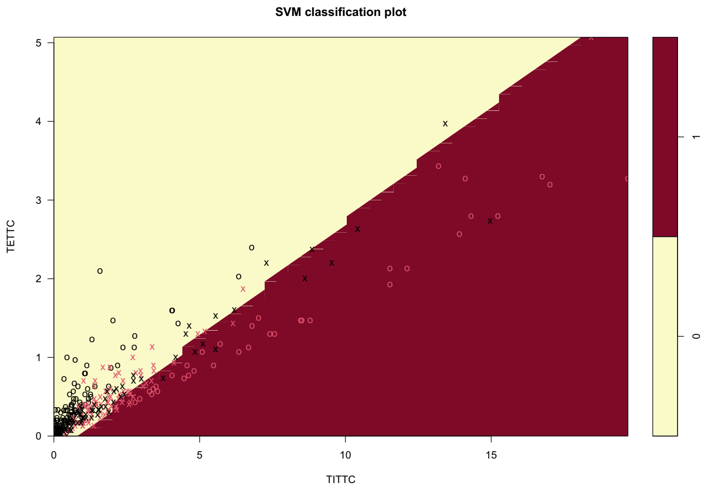
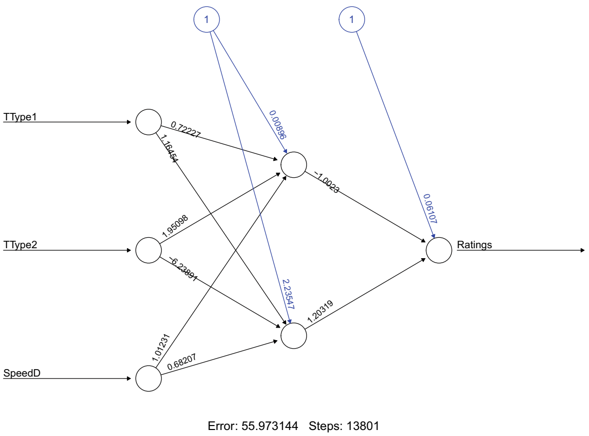
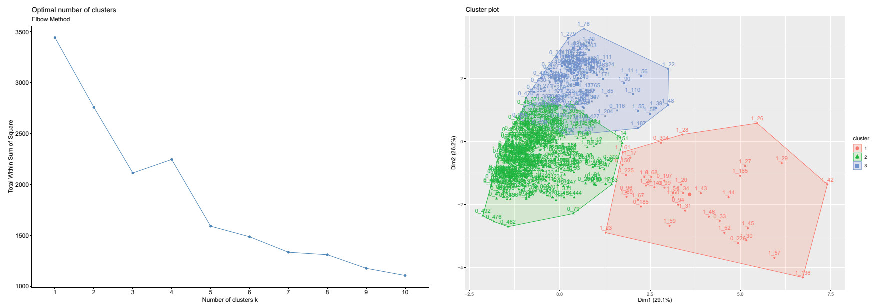


Fig. 4 Classifying the data using support vector machine

Fig. 5 Structure of the artificial neural network





(a) Estimating optimal number of clusters using elbow method

(b) Clustering the observation into three different groups

Fig. 6 K-Means clustering on all the variables

nodes. After training the model, the weights are correctly tuned, shown in Fig. 5. This model can predict the crash severity ratings with an accuracy of 64%.

2.2 Unsupervised Algorithms

K-Means Clustering is an unsupervised learning algorithm that tries to find a model which closely maps the inputs with the outputs without any levels. We used all the variables in the data set apart from the ratings to cluster all the observations. First, we used the elbow method to estimate the optimal number of clusters that encloses all the observations. From Fig. 6a, we can see that the optimal cluster number should be 3. Based on this, we cluster all the observations in three clusters, shown in Fig. 6b.

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A Fuzzy Logic Approach on Pedestrian Crossing Behaviour at Unsignalized Intersection

M. Manoj, Vivek R. Das, and Nitin Kumar

Abstract The pedestrian behaviours on roads affect their safety to a greater extent. Of many behaviours exhibited by pedestrians, crossing speeds, gender and gaps accepted and rejected to cross the road are of major concern. In this paper, the study is made on behaviour of pedestrians at unsignalized T intersection at Channasandra within the capital city of Karnataka, Bengaluru by generating a fuzzy model. The data were collected at the study location using a videographic survey for a duration of one hour during the peak hours and the above-mentioned behaviours were extracted and subjected to analysis. For modelling, gaps accepted and crossing speeds of pedestrians were given as inputs in MATLAB's fuzzy logic toolbox by framing a set of rules, and after defuzzification the choice of pedestrians to cross the road will be obtained from the model.

Keywords Pedestrian behaviours · Gaps accepted · Crossing speeds · Fuzzy logic · Unsignalized

1 Introduction

In recent times there has been a drastic increase in the number of accidents on roads caused due to vehicle–pedestrian collisions, as per MoRTH "Road Accidents in India" report of 2020, accidents corresponding to pedestrians in the year 2020 stood at 57,763 and that of death toll was 23,483 in India alone. With the increase in

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population around the globe, the general trend followed is that people settle in the urban areas. Due to this the capacity of roads in urban areas have been decreased. There is also poor planning in terms of safety provisions for non-motorized road users. The lack of space available in completely developed cities is also one of the reasons. On the other hand, people are reluctant to follow basic rules and try to risk their lives by not using the provided pedestrian facilities. These problems have been a common sight in the capital city of Karnataka, Bengaluru. Thus, this study is intended to examine the behaviour of pedestrians at unsignalized intersections. For this purpose, a T intersection was selected in the area of Channsansdra, Bengaluru. Descriptive analysis and fuzzy model were developed from the data extracted in the above-mentioned intersection. The concept of fuzzy logic which was developed by Lofti Zadeh, analyses the input data using approximate and human language [1]. The final model gives the choice behaviour of pedestrians using the input data of gaps accepted and crossing speeds.

2 Literature Review

Most of the previous studies mainly focus on pedestrian demographic characteristics (such as age and gender) and how these characteristics influence road crossing behaviour. Such studies have focused on detailed experiments to find out the effect of age on road crossing decisions with the effect of vehicle distance or speed of vehicle [2, 3]. Khan et al. [4] studied pedestrian behaviours for three different activities in the city of Karachi, Pakistan. This paper also observes the effect of sidewalks and street encroachments on pedestrians. Statistical Analysis was done using the Fisher exact test with a 95% confidence level. The results were given on all three activities of pedestrians considered namely, Road crossing behaviour, Street pedestrian behaviour, Sidewalk pedestrian behaviour, and finally on encroachments. Dıaz [5] study gives the theory of pedestrian behaviour and their intentions to violate the rules for which data was obtained from self-rating methods from the sample of pedestrians and the evaluated using statistical analysis method EQS. Lapses, errors, and violations of different gender and age groups were reported. Chai et al. [6] used fuzzy logic to examine differences in cognition for age and gender traits at jaywalks and signalized crosswalks by observing pedestrian movements captured by videography survey. Fuzzy sets and rules were defined to model the decision–making process affected by signal timing, pedestrian group, upcoming vehicles, and ongoing pedestrians. Further, improvements can be made to refine and extend the applications of the fuzzy logic method. This paper also suggests to conduct Questionnaires surveys to further understand the behaviour, perception, and attitude of road users. Dutta and Vasudevan [7] studied the effect of the waiting time of pedestrians on the rolling gap acceptance at unsignalized intersections in heterogeneous urban traffic environments in six sites in Kanpur, India. In India, pedestrian guidelines are mentioned in IRC—103 [8]. The analysis was done using the survival analysis approach. The result showed the various factors that affect the rolling gap acceptance by pedestrians.

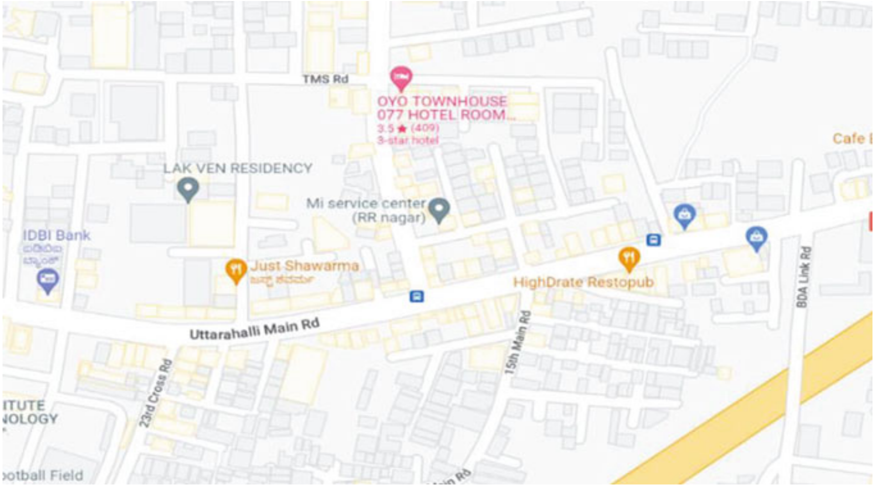

Fig. 1 Map of selected intersection (Google image dated 15.06.2022)

3 Methodology

3.1 Site Location

An unsignalized intersection was selected in Channasandra area within the city of Bengaluru, Karnataka. This location is comprised of 3 arms meeting together forming a T-junction. This intersection was selected due to more pedestrian footfall observed during the reconnaissance survey because of the presence of educational institutes and commercial activities near the intersection. Figure 1 shows the map of the location selected. Approach A connects to Mysuru Road, approach B connects to Uttarahalli, and approach C connects to the interior of RR Nagar.

3.2 Data Collection

Data was collected from the selected site location using a videographic survey with the support of a DSLR camera. The survey was conducted for a duration of one hour during peak hours, in this case the peak hour was 9 am to 10 am. The camera was placed on the terrace of a high-raised building at the intersection in such a way that the view from the camera covered the entire area of the intersection road area. Figure 2 shows the setup of camera at study area.

Fig. 2 Camera setup at Channasandra junction

Table 1 Details of Channasandra junction

Pedestrian flow	210
Vehicle count	3316
Crosswalk marking	Yes
Presence of sign boards	No
Presence of median	No

3.3 Data Extraction

The data such as pedestrian volume count, crossing speeds of pedestrians, and gaps accepted and rejected were extracted from the survey conducted through videographic survey based on the genders. Also, the few components in the vicinity of the study intersection, such as crosswalk markings and presence of sign boards, were noted. Table 1 shows a few details of the intersection. Male and female pedestrians percentage was 62% and 38% respectively of the total pedestrian volume count of 210. The average speed was found to be 1.34 m/s and 1.13 m/s for male and female pedestrians, respectively.

3.4 Descriptive Statistics

The quantitative summary of statistics obtained from the easyfit software is represented in this section. The sample data were subjected to descriptive statistics analysis for two inputs namely gaps accepted and crossing speeds of pedestrians. The mean values from the sample data are shown for minor and major crossing along with the deviation of data from its mean values. Also, the value of skewness and excess kurtosis for gap and speed data is mentioned in the following Tables 2 and 4, respectively.

Minimum, median, and maximum values along with some other percentiles of the sample data of gaps accepted and crossing speeds are tabulated in Tables 3 and 5. From this, the average crossing speeds are 1.32 m/s and 1.08 m/s for males and females, respectively. The minimum time gaps are accepted by males when compared to females. The average time gap accepted is 5.66 s and 5.44 s for males and females, respectively (Table 4).

Table 2 Descriptive statistics of gap

Statistic	Minor crossing		Major crossing	
	Male	Female	Male	Female
Range	16.73	7.99	11.29	11.05
Mean	5.52	5.28	5.9691	5.68
Variance	9.28	3.31	10.011	7.41
Std. deviation	3.05	1.82	3.164	2.72
Coef. of variation	0.55	0.34	0.53007	0.48
Std. error	0.37	0.30	0.55079	0.52
Skewness	2.04	0.45	0.87648	1.26
Excess kurtosis	5.64	−0.02	−0.05431	1.41

Table 3 Percentile statistics of gap

Percentile	Minor crossing		Major crossing	
	Male	Female	Male	Female
Min.	1.39	1.92	2.36	2.6
5%	2.10	2.67	2.40	2.72
10%	2.44	2.91	2.57	2.89
25% (Q1)	3.55	3.95	3.27	3.77
50% (Median)	5.13	5.22	5.56	4.49
75% (Q3)	6.45	6.76	8.26	6.73
90%	9.05	7.83	10.80	9.91
95%	12.38	8.90	13.26	12.35
Max.	18.12	9.91	13.65	13.65

Table 4 Descriptive statistics of speed

Statistic	Minor crossing		Major crossing	
	Male	Female	Male	Female
Range	2.59	1.11	1.11	1.22
Mean	1.12	1.57	1.57	1.36
Variance	0.12	0.13	0.13	0.17
Std. deviation	0.34	0.36	0.36	0.41
Coef. of variation	0.31	0.23	0.23	0.31
Std. error	0.04	0.06	0.06	0.07
Skewness	3.07	0.19	0.19	0.05
Excess kurtosis	15.84	−1.58	−1.58	−1.41

Table 5 Percentile statistics of speed

Percentile	Minor crossing		Major crossing	
	Male	Female	Male	Female
Min.	0.64	0.55	1.09	0.76
5%	0.78	0.65	1.11	0.76
10%	0.81	0.75	1.15	0.80
25% (Q1)	0.90	0.81	1.25	0.93
50% (Median)	1.08	0.9	1.56	1.39
75% (Q3)	1.24	1.01	1.91	1.73
90%	1.43	1.16	2.04	1.96
95%	1.59	1.27	2.12	1.97
Max.	3.23	1.29	2.20	1.98

3.5 Distribution Data

The data were subjected to distribution fittings in the easyfit software for both inputs (Gap Accepted and Crossing Speed) that are planned to be used as inputs in the MATLAB fuzzy logic. For the gaps accepted data there were four different distributions of which Dagum (3P), Gen. Extreme Value, and Lognormal (3P) have similar curves with varying equation parameters and changes in skewness and kurtosis. The distribution data of the gaps accepted is shown in Table 6.

Similarly, for the data regarding crossing speeds of pedestrians, there were three different distributions with Johnson SB distribution repeating itself for both male and female pedestrians in the major crossing section. All the three distributions are smooth curves similar to normal distribution curves with varying equation parameters and changes in skewness and kurtosis. Cauchy is a leptokurtic distribution. The distribution data of speed is shown in Table 7.

Table 6 Distribution data of gap

Arm type	Gender	Distribution	Kolmogorov Smirnov		Parameter	
			Statistic	Rank		
Minor crossing	Male	Dagum (4P)	0.08462	1	$k = 0.55762$ $\alpha = 3.4679$	$\beta = 4.8467$ $\gamma = 1.1466$
	Female	Gen. extreme value	0.07561	1	$k = -0.13449$ $\sigma = 1.666$ $\mu = 4.5142$	
Major crossing	Male	Gen. Pareto	0.09544	1	$k = -0.24169$ $\sigma = 4.9208$ $\mu = 2.0061$	
	Female	Lognormal (3P)	0.12465	1	$\sigma = 0.7852$ $\mu = 0.99376$ $\gamma = 2.0893$	

Table 7 Distribution data of speed

Arm type	Gender	Distribution	Kolmogorov Smirnov		Parameter	
			Statistic	Rank		
Minor crossing	Male	Dagum (4P)	0.0565	1	$\kappa = 0.51445$ a $= 4.201$	$\beta = 0.60585$ $\gamma = 0.592$
	Female	Cauchy	0.09432	1	$\sigma = 0.07323$ $\mu = 0.89102$	
Major crossing	Male	Johnson SB	0.10274	1	$\gamma = 0.13447$ d $= 0.3052$	$\lambda = 0.95338$ $\xi = 1.1403$
	Female	Johnson SB	0.07474	1	$\gamma = 0.03976$ d $= 0.46971$	$\lambda = 1.2862$ $\xi = 0.72888$

3.6 Fuzzy Logic Model Development

Fuzzy logic is an easy yet effective tool that suits well for the analysis of data that involves randomness and vagueness. In this paper, pedestrian choice behaviour for crossing the road is modelled in MATLAB'S fuzzy logic with two variables, gaps accepted and speed of crossing as input parameters. Gaussian distribution had a very similar match with the distribution curves obtained from the easyfit software, tabulated and explained in earlier sections. Thus, for both inputs, Gaussian membership function was defined and a triangular membership function was defined with No (decision is not crossing) and Yes (decided to cross) as output. A set of rules were framed as tabulated in Table 8 against which fuzzy logic analysis for the data was done. The results were obtained in the form of surfaces and rules and are shown for minor and major crossings for both genders from Figs. 3, 4, 5 and 6.

Table 8 Rules for evaluation

Input 1 gap	Input 2 speed	Output choice
Low	Low	No
Low	Medium	No
Low	High	No
Medium	Low	No
Medium	Medium	Yes
Medium	High	Yes
High	Low	No
High	Medium	Yes
High	High	Yes

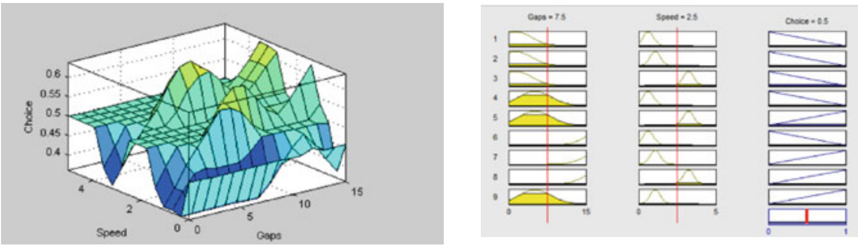

Fig. 3 Minor crossing—male

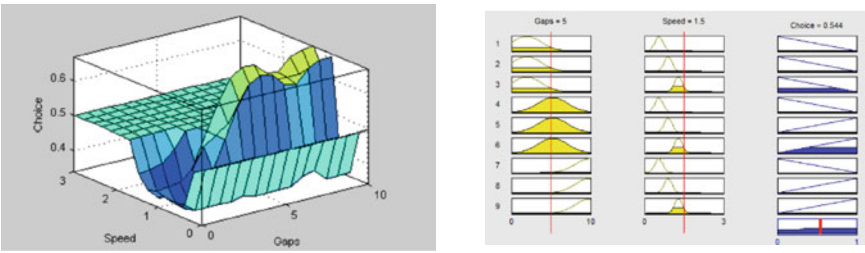

Fig. 4 Minor crossing—female

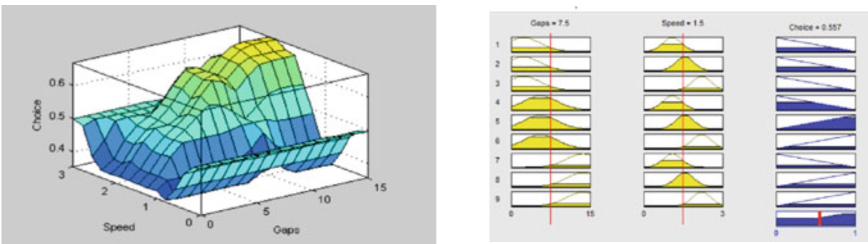

Fig. 5 Major crossing—male

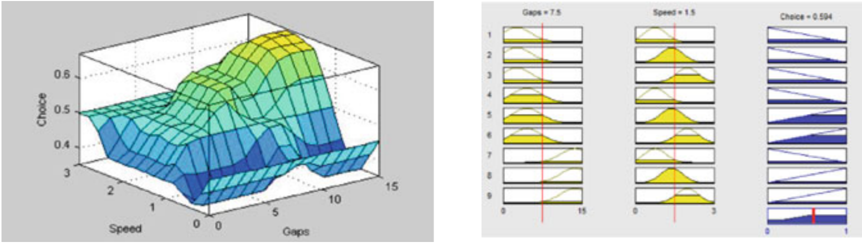

Fig. 6 Major crossing—female

4 Conclusions

- The average time gap accepted by pedestrians in seconds was 5.66 s and 5.44 s for males and females, respectively.
- The average crossing speed of pedestrians was found to be 1.32 m/s and 1.08 m/s for males and females, respectively. Thus, it is clear that the walking speed of males is found to be faster compared to that of females.
- The distributions for the gap accepted data as obtained from easyfit were Dagum (4P), Gen.Extreme Value for minor crossing corresponding to males and females, respectively, and Gen Pareto and Lognormal (3P) for major crossing corresponding to males and females, respectively.
- The distributions for the crossing speed data as obtained from easyfit were Dagum (4P), Cauchy for minor crossing corresponding to males and females, respectively, and Johnson SB distribution for both males and females in major crossings.
- Four models were developed each for minor crossing and minor crossings for two genders. These models help in the study of the choice behaviour of pedestrians at unsignalized intersections.

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A Review on Surrogate Safety Measures Using Extreme Value Theory

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Abstract Over the past few decades, the observed accident data has served as the primary source for road safety analyses. To avoid the shortcomings of crash data, traffic conflict techniques have been promoted as an alternative approach to analyze safety from a wider perspective than relying solely on crash data. Still, the application of surrogate safety measures, and the validity of conflict techniques is a great concern. The Extreme Value Theory (EVT) offers Peak Over Threshold (POT) and Block Maxima (BM) approaches to provide a robust modeling framework for relating surrogate safety measures (SSMs) to crash frequency. This study is a comprehensive review of the development and use of EVT in combination with surrogate measures for safety evaluation. A number of formulated research questions on the use of EVT for SSM analysis are also identified and discussed in a precise manner. The key finding of the study demonstrates that bivariate extreme value modeling approaches with a combination of surrogate measures, time to collision (TTC), and Post Encroachment Time (PET), are more useful to estimate safety than uni-variate extreme value modeling approaches with individual safety measures. Finally, the authors identified several research gaps to assist researchers and practitioners with recommendations for potentially useful avenues of future research.

Keywords Surrogate safety measures · Extreme value theory · Peak over threshold · Block maxima

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1 Introduction

Road safety is a matter of great concern in developed countries with lane-based traffic conditions and developing countries with non-lane traffic conditions, where rapid growth of motorized vehicles is coupled with inefficient roadway infrastructure to control outcomes. Annually, approximately 1.3 million people die, and around more than 50 million suffer non-fatal injuries globally, World Health Organization [19]. Numerous factors associated with such a scenario are mainly inefficient driving behavior, poor road infrastructure, and lack of management. Over the years, in traditional road safety analysis, several approaches have been proposed, such as before-after studies, Elvik [6]; identification of black spot programs, Ahmed et al. [1]; statistical modeling and road safety audits, Meuleners et al. [12]. These methods primarily rely on past crash data, statistical modeling, and skilled and experienced field observation. This type of study is known as a reactive technique, this begins based on existing crash data and focuses on identification of the high-risk area, which implies that a significant amount of accidents are to be recorded to study road safety.

A reactive approach has significant limitations, such as poor data quality and a lack of accident data. According to research by several previous researchers [2, 15, 18], in many cases, accidents that do not result in serious injury are not reported from the site. As a result of the aforementioned issues, it is necessary to offer an alternative safety evaluation approach.

Safety evaluation using traffic conflict technique (TCT) applying different SSMs is another alternative approach to the crash-based events method. It is a safer approach for resolving all the above-described difficulties. Initially, General Motors Corporation Laboratories adopted the TCT Perkins and Harris (1967) to characterize the interaction between two road users in traffic collision conditions and estimate the probability of a crash. traffic conflict analysis is a proactive approach to safety evaluation that does not solely rely on actual crash data. However, with the additional implementation of an image processing system and sensor-based technology for video-graphic data extraction and driver behavior characteristic data using a driving simulator and naturalistic driving study.

The validity of SSMs is typically accessed by correlation with actual crash data frequency, typically evaluated by regression analysis models [11, 20, 21]. Furthermore, some non-crash-based and more advanced approaches have been suggested by researchers in the evaluation of road safety, such as automated road safety analysis, extreme value theory, and the causal model approach [16, 17].

The extreme value theory offers two approaches, Block Maxima (BM) or Peak Over Threshold (POT), to examine the suitable threshold for surrogate safety measures. Application of BM and POT approaches is becoming increasingly important in conflict techniques approach in recent years. Firstly, Songchitruksa and Tarko [17] developed EVT based method to analyze road safety as an alternative to the classical regression analysis without relying solely on crash data. Some of the existing study applications for extreme value theory have been discussed nicely in Tarko,

2018. This study provides a comprehensive review of the recent development of the extreme value theory. The study objective is as follows:

1. To understand the state of the art of EVT and its applications in conflict techniques;
2. To discuss the current problems on the application of EVT in the road safety analysis;
3. To identify research gaps for promising future research directions of SSM and their applications in EVT safety.

The organization of this article is as follows: The first step in identifying relevant studies is to discuss the research aims and questions. The second step is to use a search technique, inclusion and exclusion criteria, and gather relevant literature. In order to identify limitations and research gaps, the current literature was further segmented into different and thoroughly discussed. Finally, the study concluded with a review of the results and suggestions for further research.

2 Methodology

In order to provide a thorough summary of the current research-related evidence, a systematic literature review (SLR) is frequently conducted. It is a systematic procedure that can be used to synthesize earlier research done by academics, researchers, and practitioners. The objective of SLR is to reduce occurrence bias in a thorough literature search across multiple databases and provide an answer to a predetermined, focused research question [13]. This study follows "The Preferred Reporting Items for Systematic Reviews and Meta-Analyses Protocols" (PRISMA) framework for analyzing previous studies.

2.1 Search Strategy

For identification of relevant literature, the following scientific databases were searched; Science Direct, Scopus, and Google Scholar. The author uses different keywords, with their combinations also used to ensure comprehensiveness. These are as follows: "conflict technique", "surrogate measure", "Extreme value theory", and "safety–critical event".

The snowballing approach was immediately applied after a first screening to locate additional pertinent missing papers. As mentioned, this methodological approach is designed to address all predefined research objectives. The total search strategy found 452 relevant studies for further analysis. The data extraction process was completed using SLR inclusion and exclusion criteria. Further relevant studies were exported into an Excel spreadsheet, and a two-step procedure was used to include pertinent systematic reviews:

Table 1 Eligibility criteria for selecting relevant literature

Sn. No	Basic criteria	Decision
1	Research presented and published in conference proceedings and peer-reviewed journals	Inclusion
2	Research defines and elaborates on key concepts, surrogate safety measures (SSM), extreme value theory (EVT)	Inclusion
3	Research is establishing the relationship between conflict and crashes	Inclusion
4	Inaccessible articles and review papers	Exclusion
5	An article that is duplicated among database search results	Exclusion
6	The study of work that is not based on original research	Exclusion
7	Studies based on surrogate safety in medical science	Exclusion
6	Grey literature and dissertations	Exclusion

1. All papers that were initially retrieved had their titles and abstracts independently reviewed by two reviewers. Discussion was used to try to reach an agreement in case of disagreement.
2. Two impartial reviewers carried out full-text screening on a few chosen systematic reviews. When there were disagreement, discussion was used to try to reach an agreement.
3. Which extreme value theory approach Block Maxima or Peak Over Threshold, can estimate the probability of conflict efficiently?

2.2 Eligibility Criteria

The application of eligibility criteria guarantees ensures that the literature used in the systematic review is relevant to the study, producing more accurate, objective, and significant results. However, a systematic selection procedure that applies the eligibility requirements criteria to the search results in a way that minimizes the possibility of selection bias would be beneficial. Several eligibility criteria were applied in order to find the pertinent literature, as shown in Table 1.

2.3 Quality Assessment

The quality evaluation has been done in multipart and corresponds to various objective studies. The following criteria were used to assess the quality assessment of the systematic literature review.

1. Are the review's inclusion and exclusion criteria sufficiently described?

2. Is it likely that the literature search included all relevant studies on surrogate safety assessment?
3. Did the chosen publication use blind reviewers to rate the study's validity and quality?
4. Was the type of extreme value theory mentioned in the literature described adequately?

2.4 Study Selection

The authors found 450 records in total using three online databases: Science Direct (80), Scopus (62), and Google Scholar (308). Apart from these database sources, the authors included two studies from other known sources. Initially, the duplicate types of literature in the database were excluded, and 350 articles remained, after which 180 were excluded during the title and abstract screening process. Furthermore, an eligibility assessment was carried out on potentially 213 full-length texts, in which 130 articles were excluded from the study because their content was irrelevant or the application of extreme value theory was not well defined. Finally, **10** full-length articles were included in the narrative synthesis. Time restrictions were not taken into account in this study. Studies must be written in English and published in peer-reviewed journals and conference proceedings, including those that describe the application of EVT in surrogate safety assessment. However, the systematic literature did not include "Dissertations" or "Grey Literature". The characteristics of the final selected studies are summarized in Table 2, and the literature mapping is described in Fig. 1.

3 Discussion

3.1 Extreme Value Theory

Extreme value theory (EVT) is a branch of statistics dealing with the stochastic behavior of extreme events and deviations from the median of probability distributions. The three extreme value distributions (Gumbel, Frechet, and Weibull) can each be used to explain the behavior of maxima for a single distribution, Fisher and Tippett [8]. The key feature of the extreme value theory is to predict rare events that are larger and less frequent than previously observed events [5]. Normally, this extreme behavior is rare and unobservable in the data collection period. Extreme value theory in transportation engineering was initially applied by Hyde and Wright [10] to estimate road traffic capacity. Henceforth et al. (2006) estimate crash probability based on specific crash proximity measures. Extremes sampling methods are frequently a source of serious concern. There are typically two types of extremes from various sample approaches.

Table 2 Summary of the literature on extreme value theory

Sn No	Author name, year	Country	Road geometry	SSM measures	Type of conflict	Modeling techniques
1	Zheng et al. [20]	China	Freeway	PET ≥ 0 s	LC	BM, POT
2	Farah and Azevedo's [7]	USA	RH	TTC ≥ 0	CC	BM, POT
3	Zheng et al. [21]	China	Freeway	PET ≤ 0 s	LC	POT
4	Borsos et al. [3]	Belarus	SI	TTC ≤ 0 s T2 ≤ -2 s	CC	BM, POT, KS
5	Songchitruksa and Tarko [17]	USA	SI	PET ≤ 0 s	CC	BM
6	Chauhan et al. [4]	India	SI	TTC < 2 s DRAC > 3.4 m/s2	RE	POT
7	Goyani et al. [9]	India	UI	PET ≤ 0 s	CC	BM, KS
8	Zheng and Sayed [23]	Canada	SI	TTC ≥ 0 PET ≥ 0	CC	BM, POT, PC, KRC
9	Jonasson and Rootzén (2014)	Sweden	Urban Road	TTC ≤ 0 s	RE	BM
10	Zheng et al. [22]	Canada	Highway	LPM > 1	LC	BBM

Note SI—Signalized Intersections, UI—Unsignalized Intersection, CS—conflicting speed of vehicle, MADR—Maximum Available Deceleration Rate, KS—Kolmogorov–Smirnov Test, RH—Rural Highway, LC—Lane change, RE—Rear End, CC—Crossing Conflict, UBM—Uni-variate Block Maxima, BBM—Bivariate Block Maxima, UPOT—Uni-variate Peak Over Threshold, BPOT—Bivariate Peak Over Threshold, PC—Pearson's Correlation, KRC—Kendall's Rank Correlation, SRC—Spearman Rank Coefficient, Light-GBM—light gradient-boosting machine, THW—end of the passing maneuver, HC—Head on Collision, PCC—Pearson correlation coefficient, AIC—Akaike Information Criterion, RL—Reinforcement Learning

3.1.1 BM Approach

Block Maxima (BM) approach observations are grouped into fixed intervals over time and space, a block of minima or maxima is treated as extremes. This shows the extreme values occurring at maximum over a period of time, such as an hour or a year. Whereas extreme event samples follow a Generalized Extreme Value (GEV) distribution, the samples are distributed in different blocks in a certain time interval, considering the largest and smallest threshold in each sample block. As BM approaches, R-largest order statistics are typically preferred as the majority of blocks have enough observations. Block maxima approach mainly focuses on the behavior of independent random observation distributions.

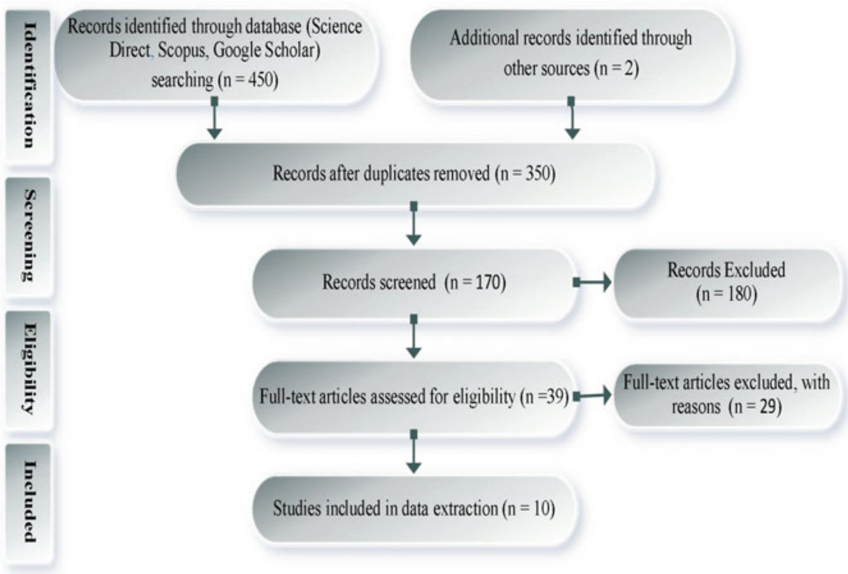

Fig. 1 Flow chart of the inclusion articles in the review *Source* Liberati et al. (2009)

$$\mathrm{Mn} = \max\{X_1, X_2, \ldots X_n\}$$

where X_1, X_2,…X_n is, X_n is a series of independent random variables with the same distribution function $F(x) = Pr(X_i \leq x)$. M_n is the process maximum over n time units of observation. When $n \to \infty$, the function M_n will converge to GEV distribution [20, 21]. The GEV function is shown below in Eq. 1.

$$F(x) = exp\{-1\left[1 + \xi\left(\frac{(x-\mu)}{\sigma}\right)\right]^{-1/}\xi]\} \quad (1)$$

The three parameters are defined as location parameter (μ) $-\infty < \mu < \infty$, scale parameter (σ) $\sigma > 0$, and shape parameter (ξ) $-\infty < \xi < \infty$.

The distribution of the function is defined as per the value of shape parameter, ξ, Frechet distribution $\xi > 0$, Weibull distribution $\xi < 0$, and Gumbel distribution $\xi = 0$.

3.1.2 POT Approach

Pickands [14] treats the Peak Hour Threshold (POT) as an extreme value if the related measurement exceeds the optimal predetermined threshold, where random observations {X1, X2, …, Xn} are identically and independently distributed within the Generalized Pareto Distributions (GPD). For a suitable threshold, the GPD is as follows:

$$G(x; \mu; \sigma; \xi) = [1 - (1 + \frac{\xi}{\sigma}(x - \mu) - 1/\xi]\} \quad (2)$$

where

$X - u$ = exceeds the threshold conditional on $X > u$, $\sigma > 0$ = scale parameter, $-\infty < \xi < \infty$ = the shape parameter.

4 Literature Review Summary

The extreme value theory was developed in the early twentieth century by Fisher and Tippett [8]. The authors distributed the limiting frequency of extreme values across larger and smaller sample sizes. The concept of EVT provides the possibility of estimating the likelihood of extreme events for a relatively short period of observation. EVT is a method for analyzing safety quickly that is similar to SSMs. The approach produces a group of models that make it possible to extrapolate from frequently occurring events, like traffic conflicts, to less frequently occurring events like crashes [23]. In addition, EVT provides a single dimension to assess the severity of surrogate events and identifies actual crashes [17, 20].

EVT use in analyzing traffic safety has grown significantly in recent years. Initially, Songchitruksa and Tarko [17] proposed the EVT (Block maxima) method to evaluate the safety of right-turning movements at 18 signalized intersections for an 8-h duration by SSM indicator post encroachment time. The proposed method evaluation findings showed a correlation between model estimates and crash data. Henceforth, analyzing the safety of freeway, authors, Zheng et al. [20] used two approaches, BM and POT, to evaluate lane change maneuver behavior at several freeway spans. Studies done by Zheng et al. [20] and Borsos et al. [3] found that the POT approach performs better than the BM for short interval data in terms of data estimate accuracy and reliability. On the contrary, for long interval data, the BM approach yielded better accuracy than the POT approach, Farah and Azevedo [7].

Moreover, analyzing the lane change behavior, Zheng et al. [21] proposed a shifted Gamma-GPD model and parameters such as threshold μ and shifted value δ to estimate crashes. They compared the Bayesian approach to the conventional maximum likelihood estimation approach. However, the application of naturalistic driving in estimating near-crash safety continuum, the study by Jonasson and Rootzén (2014), used the bivariate BM approach, safety measures TTC, in combination with a number of explanatory variables (max speed, min distance, right lane marking, etc.) to estimate a crash probability. Farah and Azevedo [7] used the BM and POT approaches to evaluate head-on collisions during passing maneuvers on two-lane rural highways. For freeway safety, PET and length proportion of merging (LPM) were used by Zheng et al. [22] to categorize the seriousness of events in merging areas. The authors proposed combining a bivariate extreme value model with a bivariate GPD. The findings demonstrated that the estimates of crashes from the bivariate model were more precise and accurate than those from traditional uni-variate models. Additionally, Zheng and Sayed [23] compared bivariate (BGP and BGEV) and uni-variate (UGP & UGEV) extreme value models and found that conflict, estimated by bivariate extreme value model (GPD), performs better than bivariate GEV and conventional uni-variate extreme value models. Borsos et al. [3] used TTC_{min} and T_2 and showed that the conflict estimated by TTC_{min} is more accurate, whereas SSM indicator T2 overestimated the precise crash probability. Zheng and Sayed [23] observed that bivariate extreme value GPD models outperform traditional uni-variate extreme value models for safety analysis. Moreover, for carrying out safety analysis in mixed traffic conditions, Chauhan et al. [4] used the POT approach to determine the threshold for conflict probability. They found the probability of conflict TTC threshold less than 2 s and DRAC threshold greater than 3.4 m/s^2. Furthermore, Zheng et al. [20] adopted a PET threshold of less than 1 s for defining the likelihood of crash, whereas Goyani et al. [9] selected both lower and upper limits (−6, 6) and used the PET threshold level to estimate the critical conflict (−1, 1).

5 Conclusions

Traffic conflict techniques have been widely used in recent years to analyze the safety of different traffic facilities (Intersections, Mid-Blocks, and Highways) in real time and provide remedial measures before and after safety analysis. This study reviewed several existing studies on the application of EVT in the field of road safety. This approach can predict the probability of a crash consistently and reliably without being dependent on crash data. The EVT approach is applied to estimate the collision risk for uni-variate and bivariate models with the generalized extreme value in the BM approach and the generalized Pareto distribution in the POT approach. The study highlights the different traffic conflicts defined by different types of indicators. However, PET and TTC have shown promising results in previous literature, both PET and TTC are good indicators for near-crash rear-end, lane change, and crossing collisions. In summary, early work argued that the bivariate extreme value model is more useful for estimating safety than the uni-variate model. EVT (POT) approach's overall performance is more reliable and practical than EVT (BM) approach. However, the researchers argued that more comparative research is required before a firm conclusion can be drawn regarding which technique is superior. The main challenge of Extreme value theory is varying degrees of validity. These groups were determined by analyzing crashes based on historical data and 95% Poisson confidence intervals. However, previous literature has shown that the accuracy of both categories is quite difficult. Underlying conflict process is independent and uniformly distributed, and time stationary. Existing research is typically constrained in terms of data collection regarding traffic conflict observations, study sites, and time intervals that may not adequately cover a variety of traffic. Furthermore, extended observation periods and more sites are also required to address this issue. The multi-variate EVT method may be of interest to improve EVT performance by combining multiple indicators. Aside from that, the implications of the EVT approach in safety analysis are primarily done in homogeneous traffic conditions, where traffic follows a lane-based pattern. Further research is needed to take into account non-lane-based traffic patterns in heterogeneous environments. Future research requires the application of EVT in naturalistic driving datasets to prove more relationships between driver characteristics and accident causes and provides a comprehensive crash analysis. More research must include the influence of geometric and environmental factors on threshold heterogeneity.

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Examining the Evasive Behaviour of Pedestrians to Measure Their Degree of Vulnerabilities at Unsignalised Intersections

George Kennedy Lyngdoh, Aakash Bhardwaj, Manish Dutta, and Suprava Jena

Abstract Pedestrian safety has become a significant problem in today's world, and pedestrian casualties have escalated in developing countries where there are no special provisions for the movement of such vulnerable road users. The interaction between pedestrians and vehicles must be prioritised and requires extended study. Therefore, surrogate safety measures (SSMs) come in handy to find the relative spatial and temporal measures of road users under conflict. This study's primary objective is to identify evasive action-based pedestrian safety indicators that are best suited for predicting pedestrian behaviour under mixed traffic conditions. The pedestrian's step frequency and lateral deviation are proven useful in measuring evasive actions based on the pedestrian's trajectory data. Furthermore, an expert-based analysis is undertaken to evaluate which evasive action-based parameters are most appropriate in identifying the severity of pedestrian safety. Hence, it was found that the lateral deviation has a more significant potential influence on severity identification than step frequency.

Keywords Pedestrian safety · Surrogate safety measures · Trajectory data · Un-signalised intersection

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1 Introduction

Pedestrians, also known as "vulnerable road users," are particularly prone to severe injuries and fatalities when they encounter vehicular collisions. Pedestrian safety at junctions is a significant problem that must be addressed effectively. Cyclists and pedestrians are the most vulnerable road users, accounting for 26% of all road accident-related deaths worldwide(WHO) [1]. Pedestrians accounted for 13.8% of road accident fatalities in India in 2017, adding to 20,457 deaths (MORTH) [2]. The heterogeneity in the traffic conditions where multiple vehicles of different dynamic and static characteristics share the same road with pedestrians contributes to the high crash rate. As seen on Indian roads, the loose lane discipline and the poor yielding behaviour of both vehicles and pedestrians further worsen the situation. Traditional road safety methodologies in India are based on accidents reported in the police's First Information Reports (FIR). These records are very basic, and pedestrian safety analysis using such data of varying quantity and quality will not produce accurate results. Therefore, surrogate safety measures (SSMs) are necessary for safety analysis for a developing nation like India to overcome such problems.

2 Literature Review

The pedestrian-vehicle interaction is very dynamic in nature since it depends on numerous factors and conditions that occur when the movement of a pedestrian and a vehicle are in conflict [3]. A traffic conflict is "… an event involving two or more road users, in which the action of one user causes the other user to make an evasive manoeuvre to avoid a collision" [4]. The nature of this conflict may differ based on the geometry of the road where it takes place, the type of intersections, any legal restrictions, and the requirements of the driver and pedestrian [5]. According to Fu et al. [6], the willingness of the driver to yield and the aggressiveness of the pedestrian are just two of the numerous variables that might affect this conflict. It may result in instances where the driver makes an unexpected judgement, while the pedestrian has already considered the driver's opinion, leading to a conflict or even an accident. An important factor to consider when assessing pedestrian behaviour is the average walking speed. For instance, Goh et al. [7] compared the crossing speed of pedestrians at both signalised and unsignalised crosswalks, and it was found that the speed is higher in signalised crosswalks. Knoblauch et al. [8] found that on wider roadways, pedestrians frequently have higher crossing speed. The evasive behaviours of pedestrians in conflict situations have been investigated by numerous researchers. Malkhamah et al. [9] reported that evasive action is taken by both drivers and pedestrians to avoid conflict at a Pelican crossing. Tageldin et al. [10] compared two surrogate safety indicators to examine evasive action-based indicators and temporal proximity indicators in assessing the seriousness of pedestrian conflicts in heterogeneous traffic situations. Pedestrian evasive behaviours, such as rushing, running, or stopping, were used by Medina et al. [11] to categorise pedestrian conflict severity levels. Traditionally, field observations and manual measurements have been used to gather pedestrian data, which are time-consuming, labour-intensive, and costly. Computer vision algorithms were successfully used in numerous pedestrian data collection to automatically detect and track pedestrian and vehicular movement in videos and to conduct a thorough safety analysis of pedestrian behaviour [12, 13].

3 Methodology

Historical data of accidents and SSMs have been extensively used to characterised vehicle–pedestrian interaction. The present study aims to identify evasive action-based parameters to find the dependency of interaction behaviour of pedestrians. Therefore, it is critical to select the best indicator to indicate evasive actions of road users. Figure 1 shows the flowchart of safety analysis adopted in this study.

3.1 Data Acquisition

Videographic survey was conducted to collect data from a T-intersection in Silchar, Assam, India. The camera was installed at a height from a nearby building to obtain a clear view of the intersection. Recording was done for two hours every day for five days during peak hour traffic. Table 1 contains information on the junction, including vehicle and pedestrian volumes.

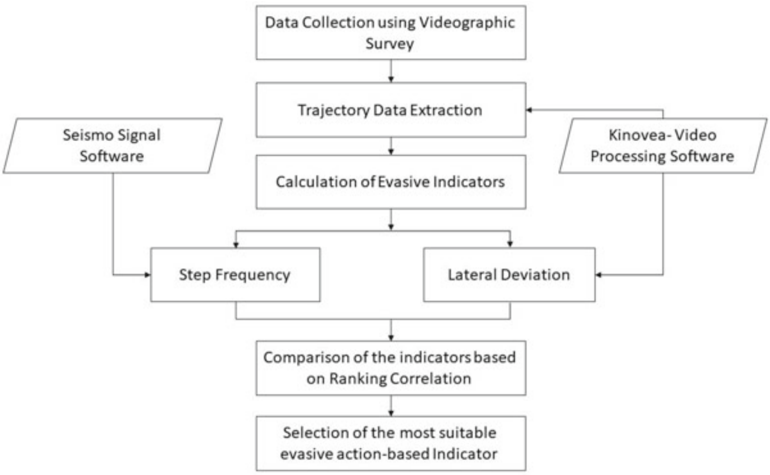


Fig. 1 Proposed methodology of the study

Table 1 Data for the observed intersection

Intersection	Total video duration (hr)	Pedestrian volume (ped/hr)	Vehicular volume (veh/hr)
T-intersection	10	410	1800

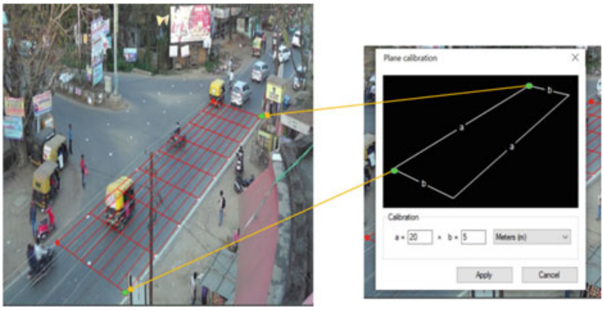


Fig. 2 Plane calibration using the perspective grid

3.2 Data Extraction

The camera data is analysed using an open-sourced, semi-automated tracking software, Kinovea, to extract trajectory data. The following steps are involved in trajectory-based data acquisition:

- Coordinate transformation: A perspective grid of a specified length is created on the video using a homography matrix. Figure 2 depicts a snapshot of the video calibration utilising the perspective grid.
- Obtaining trajectory data: The track path option was used to select the appropriate vehicle and pedestrian to track both road users. Any noise in the trajectory data is filtered using the Butterworth filter. Care is taken while setting the feature tracker to reduce errors as much as possible.

4 Results and Discussions

4.1 Analysis of the Selected Evasive Action-Based Indicators

Step Frequency. It is the number of times a foot touches the surface per unit time [14]. Seismosignal software is used to calculate the step frequency profile based on the speed profile of the pedestrian obtained from the trajectory data, as shown in Fig. 3. The speed profile is used as input for the software.

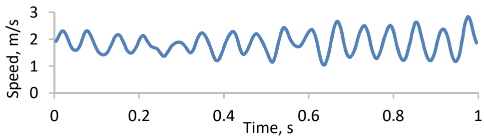


Fig. 3 Speed profile of pedestrian

After analysing all the extracted speed profiles of pedestrians, it has been observed that the maximum number of power amplitude peaks lies in the frequency range of 0.5–2.5 Hz. Figure 4 shows the power spectrum profile of the trajectory data.

Lateral Deviation. Pedestrians change their paths, along with step frequency, while crossing the street to avoid collisions. For pedestrians, two sorts of route adjustments were observed: one to avoid collision and the other to shorten the trip distance. The first type of route variation was seen before the collision zone, while the second type was observed after the collision zone [15]. The change in path is characterised by the lateral deviation obtained during trajectory data analysis. Figure 5 shows the lateral deviation (LD) calculation assuming that the pedestrian's ideal path is perpendicular to the origin when the pedestrian decides to cross. The 85th percentile of the maximum lateral deviation was calculated as shown in Fig. 6 and is equal to 1.18 m, which indicates evasive behaviour.

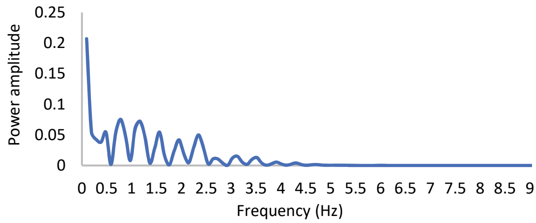


Fig. 4 Power spectrum profile

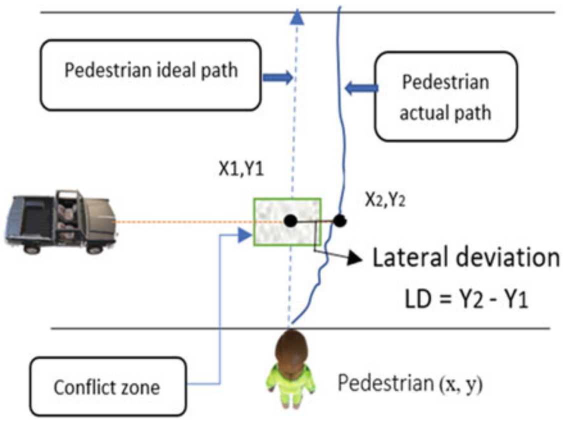


Fig. 5 Lateral deviation in the pedestrian path

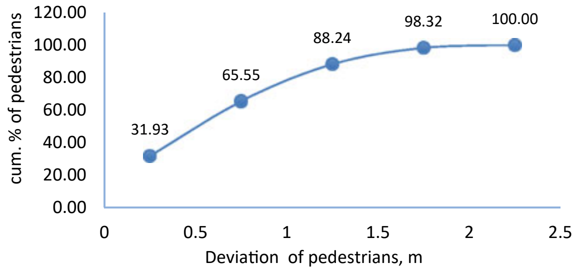


Fig. 6. 85th percentile maximum lateral deviation graph

4.2 Analysis Based on Expert Evaluation

To see if the estimated evasive indicators were effective in indicating conflict severity, the ranking of two traffic safety experts was used to compare the evasive indicators. The experts involved in traffic safety studies were asked to classify these conflicts based on severity. Using Sayed and Zein's severity criterion, the experts were asked to rate pedestrian evasive behaviours within each category [16].

Consistency Test. Cohen's kappa coefficient was used to assess the level of agreement between the two experts [17]. The agreement between the two experts was determined using the classification of both experts, as shown in Table 2 for the pedestrian step frequency dataset and Table 3 for the pedestrian lateral deviation dataset.

In Eq. 1, the kappa coefficient is defined as follows:

Table 2 The outcomes of two safety experts' agreement for the pedestrian step frequency dataset

Expert 1	Expert 2			
	low	Medium	High	Total
Low	4	3	0	7
Medium	3	3	3	9
High	0	2	7	9
Total	7	8	10	25

Table 3 The outcomes of two safety experts' agreement for the pedestrian lateral deviation dataset

Expert 1	Expert 2			
	low	Medium	High	Total
Low	4	4	1	9
Medium	3	3	1	7
High	2	1	6	9
Total	9	8	8	25

$$k = \frac{P - P_e}{1 - P_e} \tag{1}$$

where

P = % agreement observed,

P_e = % agreement expected by chance alone.

The value of kappa, i.e. k, was determined to be 0.34 and 0.28 for pedestrian step frequency and lateral deviation, respectively. Equation 2 is used to compute Kappa variance, shown as follows.

$$var(k) = \frac{1}{N} \times \frac{\sum_j Pj^2 - \left(\sum_j Pj^2\right)^2}{\left(1 - \sum_j Pj^2\right)^2} \tag{2}$$

where

N = number of cases in total,

j = 1 to n = 3 categories of classification,

P_j = proportion of all jth category assignments.

Table 4 summarises the findings of the kappa test for evasive action indication. Under the hypothesis of no agreement beyond chance and using the central limit theorem, the k/√var(k) may be roughly distributed as a standard normal variation [17]. It can be seen that k/√var(k) is greater than the Z-value at a 95% significance level (Z = 1.96). Hence, there is a statistically strong agreement between the two experts.

Correlation of Ranking. The experts' ratings and the ranking based on evasive indicators are compared using the Spearman rank correlation. The ranking correlation evaluates the intensity of the ranking of various conflicts with respect to the researched indicators. Equation 3 is used to derive this correlation, and the results are shown in Table 5.

$$r = 1 - \frac{6\sum d^2}{n^3 - n} \tag{3}$$

where

Table 4 Results of consistency test between experts

	Evasive indicator	
	pedestrian step frequency	Pedestrian lateral deviation
P	0.56	0.52
P_e	0.34	0.33
k	0.34	0.28
Var(k)	0.0204	0.0203
k/√var(k)	2.35	1.97

Table 5 Spearman rank correlation coefficient based on experts' rankings

Spearman's rank correlation coefficient	Step frequency	Lateral deviation
Expert 1	0.7088	0.7177
Expert 2	0.6680	0.7861

d = difference between the ranks of observations,

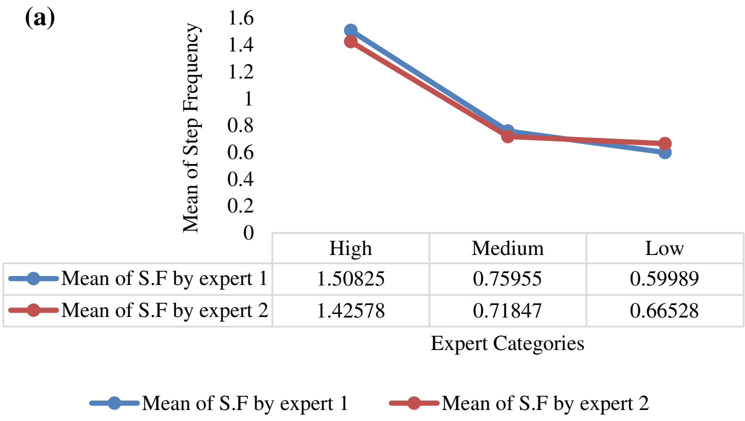
n = number of conflicts.

Severity Trend. The efficacy of the proposed indicators is examined in this section. The first stage in comparing severity measurements is to examine the trend along the expert-assigned categories. In general, the expert categories' means exhibited a decreasing tendency from the most severe, i.e. high, to the least severe, i.e. low. The profile in Fig. 7 shows a declining pattern, indicating that the severity ranked by experts is highly correlated with step frequency and lateral deviation.

5 Conclusions

This study aims to create a framework for studying pedestrian interactions with vehicles at unsignalised junctions. The significant findings are discussed below:

- This study examines pedestrian step frequency and lateral deviation as the evasive action-based indicators based on the literature gap. The selected indicators are effective and dependable in recognising conflicts at an unsignalised junction under mixed traffic situations.
- It was discovered that the greatest number of power amplitude peaks are in the range of 0.5–2.5 Hz, indicating the pedestrian step frequency during evasive action. To validate the frequency range obtained from the power spectrum profile, 25 pedestrian profiles were chosen randomly, and their step frequencies were manually calculated. The manually calculated step frequency range is 0.64–2.27 Hz. Both values are nearly identical, indicating better accuracy of the result.
- The estimated evasive indicators were compared to the ranking of two experts to see if they effectively indicated conflict severity. Only 25 pedestrian trajectories were randomly employed because a larger number of pedestrian trajectories from the research area might complicate the expert rating procedure. The lateral deviation is shown to be more suitable for severity identification than step frequency.



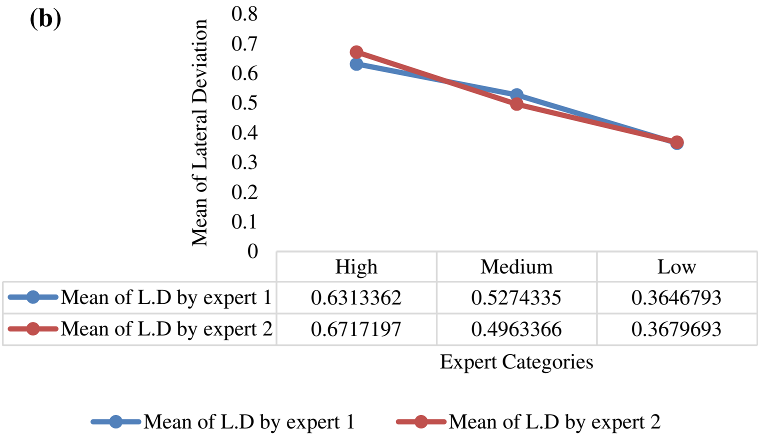


Fig. 7 **a** Severity trends of expert categories for step frequency. **b** Severity trends of expert categories for lateral deviation

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Exploring PageRank Algorithm and Voronoi Diagrams for Dynamic Network Partitions Facilitating Feedback Linearization-Based Control

Saumya Gupta, Pushkin Kachroo, Shaurya Agarwal, and Kaan Ozbay

Abstract This paper explores a novel approach to dividing a traffic region (network) into sub-regions for efficient traffic control among the areas. The macroscopic flow diagram (MFD) in each of these sub-regions, referred to as sub-MFD, can then be used to determine the macro-state of that sub-region and subsequently design controllers. The region division is based on the theory of complex networks. We exploit the inherent network characteristics through the PageRank centrality algorithm to identify the most significant nodes in the traffic network. We use these significant nodes as the seeds for a Voronoi diagram-based partitioning mechanism of the network. A feedback linearization-based controller is then presented, which controls the traffic flow between the sub-regions. A case study is performed for the Manhattan area in New York City to demonstrate the network partitioning approach; the control approach is demonstrated through a toy example containing two sub-regions.

Keywords Macroscopic Flow Diagram (MFD) · Sub-MFD · Traffic flow control

1 Introduction

Efficient traffic control of large-scale urban transportation networks remains a big challenge for researchers. Challenges include uncertainty in user behavior, accurate estimation of origin–destination (OD) trip matrices, and a reasonable estimate of a network state. Traditionally, the developed control algorithms focused on individually controlling each link and signalized intersections. There are multiple prob-

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lems associated with the existing network control strategies. Firstly, it is complex to model traffic dynamics at each link and intersection on a large-scale urban network. Secondly, these models or simulation scenarios are challenging to calibrate due to the high amount of stochasticity involved in traveler behavior and network parameters. Thirdly, even if we develop a realistic model or simulation scenario, many input/output variables associated with the control design make the implementation infeasible. Issues such as controllability and observability of the network come into play as well [1–3]. In addition to link-level control, a macro approach to managing traffic flow between regions is also needed in large-scale networks. Researchers have been exploring the idea of performing perimeter control of an area based on an aggregation of traffic conditions in a network. This approach helps in reducing the complexity of the modeling and control design, as well as in the deployment of traffic control strategies.

Aggregating traffic states over a region has recently been of active interest to researchers due to the desire to efficiently manage traffic in larger regions. The concept of the Macroscopic Fundamental Diagram (MFD) has been around for a while but has only recently started to gain significant traction and attention from researchers. Godfrey presented the idea of MFD through his research in 1969, where he reported the network-wide relationship between observed speed and density [4]. Herman and Prigogine [5] presented the idea of aggregation of traffic variables for a city, where they consider moving vehicles and the fraction of vehicles that are stopped to estimate aggregated traffic conditions in a traffic network. Proof of the existence of the MFD using analysis of experimental data from downtown Yokohama, Japan, was presented in [6]. The authors further made a case for the MFD relation between average speed and densities over the network of a certain class and provided some analytical treatment as well in [7]. A generalized MFD that uses the inhomogeneity of the traffic has been proposed in [8].

As an example of the importance of MFD and its usage in practical problems, [9] used it for the model predictive perimeter control of an area partitioned as two aggregated regions. Geroliminis et al. also developed an optimal perimeter control problem for two-region urban cities by utilizing the MFD concept in [10] and solved it by using a model predictive control approach. Another study based on using a model predictive control is [11]. Daganzo developed an adaptive control approach to improve urban mobility and decrease congestion in [12] by observing and controlling aggregate vehicular accumulations in neighborhoods. Ekbatani et al. also exploited the idea of a network fundamental diagram (NFD) to improve mobility in congested conditions by applying gating techniques using feedback control [13]. Although the concept of MFD is promising and has a lot of potential, its existence is not guaranteed. A well-defined MFD exists only under certain conditions, which were discussed in [14].

However, due to the large network size and variations within the network, aggregated variables sometimes do not truly represent the entire network. Recent research indicates that link density heterogeneity is very crucial in determining the shape and scatter of MFD [15]. Moreover, heterogeneity can also cause hysteresis loops and degradation of network performance [16]. An MFD is expected to be well-defined under the condition that the network is homogeneous with similar link properties. However, in reality, large-scale urban networks are expected to have various congestion levels. Now, in order to exploit the usefulness of MFD in designing macroscopic control strategies, it becomes essential to subdivide the network into smaller regions for the existence of sub-MFDs in each of these subregions. The first attempt in this direction was made by Ji & Geroliminis in [17], focusing on clustering of the network based on spatial congestion distribution for a given time period.

Contribution: This paper aims to explore the inherent network characteristics to create effective sub-division algorithms. We present a novel methodology to divide an urban network into sub-networks, where an MFD determines the macro state of those sub-networks. The region division is based on the theory of complex networks [18]. We exploit the inherent network characteristics through the PageRank algorithm [19] to identify the most significant nodes in the traffic network. We then use these significant nodes as the seeds for a Voronoi diagram-based partitioning mechanism of the network [20, 21]. A feedback linearization-based controller is then presented, which controls the flow among these sub-regions. A case study is performed for the Manhattan area in New York City, and results are provided through simulations.

2 Background

2.1 Complex Networks

Cyber-physical systems are becoming the backbone of modern society. These systems have a very high level of connectivity and the study area of *complex networks* has been developing to answer many questions in this field. Many introductory survey papers [18, 22] and several books exist that provide details of the theory and applications of complex networks [23–25]. Different types of networks that come under this study area include computer networks, social networks, economic networks, biological networks, and transportation networks. This paper uses the theory of complex networks to perform dynamic aggregation of regions so that MFD-based control strategies can be applied to the overall network. Specifically, we use the page rank algorithm to identify the most important nodes in a transportation network. This can be performed based on the structure of a directed graph. It can also be performed on a weighted directed graph where the weights represent the dynamic traffic conditions, for instance, in terms of time-varying traffic densities, on the links of the graph.

2.2 PageRank Algorithm

Google uses the PageRank algorithm to rank web pages in their search results [19]. This algorithm measures the importance of web pages by counting the number and quality of links. Next, we will provide some basic terminologies and discussion on the subject.

Consider a non-weighted network ($\mathcal{N}$, $\mathcal{E}$), where $\mathcal{N}$ is the set of nodes and $\mathcal{E}$ is the set of edges, such that

$$\mathcal{N} = \{a, b, c, d, \ldots\}, \mathcal{E} = \{(a, b) \mid \forall \text{ pair of connected nodes}\}$$

Now the basic node centrality/significance index can be defined in terms of the degree of the node. For instance, the degree of node a is representative of the direct number of links it has and is defined as

$$\text{degree } (a) = |\{(a, b) \in \mathcal{E}\}|, \tag{1}$$

where $|.|$ is the degree operator giving the total count of valid connections of the node a.

Another idea immediately following the degree centrality concept is that node centrality is proportional to the cumulative degree of its direct neighbors. Mathematically speaking:

$$\text{nodecentrality } (a) = \sum_{(a,b)\in\mathcal{E}} \text{degree } (b), \tag{2}$$

The underlying argument in the PageRank algorithm is that the weight distribution of each connection is not uniform. Rather, it is inversely proportional to the number of other connections that the neighboring nodes have. The following recursive formula defines PageRank centrality:

$$\text{PageRank } (a) = \alpha \sum_{(a,b)\in\mathcal{E}} \frac{\text{PageRank } (b)}{\text{degree } (b)} + \frac{1-\alpha}{n} \tag{3}$$

where $0 < \alpha < 1$ is the damping parameter, and n is the total number of nodes.

2.3 Voronoi Diagrams

Voronoi diagrams provide a computational geometric method to divide a region into subregions based on distance from a given set of seed points [20, 26]. The problem with the Voronoi diagram is as follows:

[Voronoi Problem] Given $\Omega \subset \mathcal{R}^n$, and $s_i \in \Omega, i \in \{1, 2, \cdot, N\}$, then find $v_i, v_i \subset \Omega, i \in \{1, 2, \cdot, N\}$, such that

$$\bigcup_{i=1}^{N} v_i = \Omega, \quad v_i^0 \cap v_j^0 = \emptyset \quad \text{and}$$
$$v_i = \{x \in \Omega \mid d(x, s_i) \leq d(x, s_j), \forall j \neq i\}$$

Here, the subscript on the regions such as v_i^0 means the interior of the set, and the function $d(x, y)$ is the distance between the points x and y in the metric space.

3 Main Algorithm

The flowchart for the algorithm for the overall subdivision, aggregation, and region feedback control is shown in Fig. 1.

The task of creating subregions based on the PageRank algorithm can be performed once in a static setting or can be repeatedly applied as the traffic in the system changes, which can produce changing boundaries of various regions in a time-varying fashion. The static map data (link lengths, etc.) can be used as an input in the static setting to perform the PageRank algorithm and obtain the most critical nodes of the directed graph (digraph). Alternately, in a dynamic environment, one can create a weighted digraph, where the weights on the links are obtained via traffic densities. This would give a changing set of the most critical nodes as the traffic conditions change. After obtaining the nodes, we apply the Voronoi algorithm to divide the overall region into subregions. Once the regions are obtained, we can use MFD for each region in order to develop a control law on the simplified network model.

4 Case Study

Once the subregions have been established with their corresponding MFDs, we write down the conservation flow dynamics for traffic densities for each subregion being its state variable. This provides us with state space control dynamics where control variables are also used to indicate how traffic control has to be performed. For instance, if

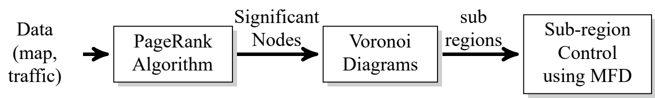

Fig. 1 Main algorithm flowchart

(a) Top Nodes (b) Top Nodes (c) Subnetworks

Fig. 2 Manhattan, New York area divisions

the perimeter control in the region has to be performed using a ramp control [27] or a gating mechanism, a corresponding control variable will be present in the dynamics. Once the dynamics are established, one can design a feedback control law to satisfy some design requirements, such as some asymptotic performance or optimality. An example of dynamics using MFD is given in [10].

We consider the Manhattan road network for the application of the proposed approach and divide it into sub-networks based on the PageRank algorithm. The Manhattan Open Street Map network was downloaded from *www.openstreetmap.org*. Which was then further parsed and processed to extract the valid nodes (intersections) and their latitude-longitude values. The total number of valid nodes in the data for the Manhattan road network was 446, 415.

We next analyze the data using *NetworkX* package in *python* and apply the PageRank algorithm to obtain the ten most significant nodes. Figure 2a shows the top significant nodes marked on the Manhattan road network. Sets of nodes that were very close and overlapping were considered as one.

Figure 2b shows the markings of nodes on a Google map. It is pretty interesting to note that most of these nodes are close to the major entry bridges/tunnels into the Manhattan area. We next apply the Voronoi diagram method to divide the network into sub-networks, using these significant nodes as seeds. The divided regions are shown in Fig. 2c.

The subdivided network can now be represented as a weighted digraph to generate a state space representation of finite dimensional traffic dynamics for the entire network. The directed graph is shown in Fig. 3. In fact, the output of the Voronoi diagram can lead to more features, such as extra subregions, depending on the local geography and traffic density distribution. For instance, in this case, the waterway further divides the nodes 2 and nodes 5. Each MFD region should have a low variance in its internal traffic density distribution. Hence, the region boundary design can be performed by combining the results of the Voronoi algorithm, geographical features,

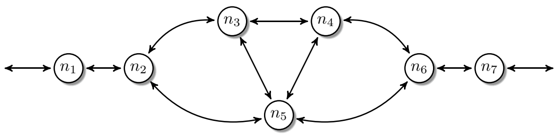

Fig. 3 Network digraph

and traffic density variance. We can modify the digraph based on these additional changes.

We now develop the dynamics for the system for the traffic moving on the digraph in Fig. 3. Control variables can appear at multiple places in the equations depending on what actual mechanism is available in the system. For instance, congestion pricing might be used to control the inflow into a toll road/bridge; other control mechanisms may include ramp metering or adaptive signalized intersections. Control objectives can be created based on the desired values of the MFD variables in the system dynamics.

5 Control Design for Two Region Case

An example of dynamics using MFD is given in [10]. Borrowing the dynamics from there, we have its labeled digraph and equations in (4) (Fig. 4).

$$\begin{aligned}
\frac{dn_{11}(t)}{dt} &= q_{11}(t) + u_{21}(t)M_{21}(t) - M_{11}(t)\\
\frac{dn_{12}(t)}{dt} &= q_{12}(t) - u_{12}(t)M_{12}(t)\\
\frac{dn_{21}(t)}{dt} &= q_{21}(t) - u_{21}(t)M_{21}(t)\\
\frac{dn_{22}(t)}{dt} &= q_{22}(t) + u_{12}(t)M_{12}(t) - M_{22}(t)
\end{aligned} \tag{4}$$

The region r_1 has n_1 total number of vehicles which is a sum of n_{11}, the number of vehicles in r_1 with destination in r_1, and n_{12}, the number of vehicles in r_1 with destination in r_2. The variables for region r_2 are defined analogously. The variable q_{ij} indicates the trips starting from region i with destination in region j. The variable

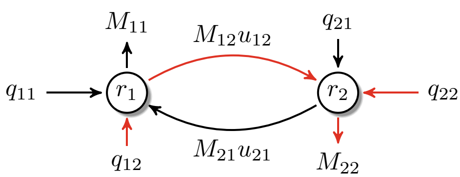

Fig. 4 Two region network

M_{ij} indicates the trips ending in region i with origin in region j. The control variables $u_{ij} \in [0, 1]$ indicate the fraction of the flow M_{ij} they allow to go through. The flow variables M_{ij} are related to n_i, n_{ij}, and $G_i(n_i)$ where $G_i(n_i)$ is the MFD variable indicating the average flow in a region as a function of the total number of vehicles (the product of the average density in a region and the total lane length in the region) in that region. These relationships are noted in Eq. (5).

$$n_i(t) = n_{ii}(t) + n_{ij}(t), i \neq j \ , \quad \text{and} \quad M_{ij}(t) = \frac{n_{ij}(t)}{n_i(t)} G_i(n_i) \tag{5}$$

In order to design control laws to try to achieve desired flows obtained by maintaining critical densities (or number of vehicles) in each region, we combine Eq. (4) with Eq. (5) to obtain

$$\frac{d}{dt}\begin{bmatrix} n_1(t) \\ n_2(t) \end{bmatrix} = \begin{bmatrix} q_{11}(t) + q_{12}(t) - M_{11}(t) \\ q_{21}(t) + q_{22}(t) - M_{22}(t) \end{bmatrix} + \begin{bmatrix} M_{21}(t) & -M_{12}(t) \\ -M_{21}(t) & M_{12}(t) \end{bmatrix} \begin{bmatrix} u_{21}(t) \\ u_{12}(t) \end{bmatrix} \tag{6}$$

We rewrite Eq. (6) using matrix notation which is implied by that equation as

$$\frac{dn(t)}{dt} = Q(t) + M(t)u(t) \tag{7}$$

Equation (6) is decoupled by the control law:

$$u(t) = M^{-1}[-Q + v] \tag{8}$$

where the vector v has components v_1 and v_2 that are chosen as

$$v_i(t) = -K_i(n_i(t) - n_{id}) \tag{9}$$

In order to make the vector $n(t)$ with components $n_i(t)$ follow a desired vector of number of vehicles in each region given by the vector n_d which has components n_{id}, we choose positive values for k_i the control gains. This control is designed to ensure $n(t) \rightarrow n_d(t)$ as $t \rightarrow \infty$. Figure 5 shows the simulation results for two regions. Values of K_1 and K_2 are fixed at 1 and 2, respectively. The simulation results show that both the sub-regions achieve the desired state (number of vehicles) by using the proposed control law. Error term also goes to zero as $t \rightarrow \infty$.

5.1 Hierarchical Control

The MFD framework automatically lends itself to a natural hierarchical control structure. Each MFD region should have a low variance traffic density distribution for the MFD to be valid in that region. So, at the higher level, MFD can be used to control

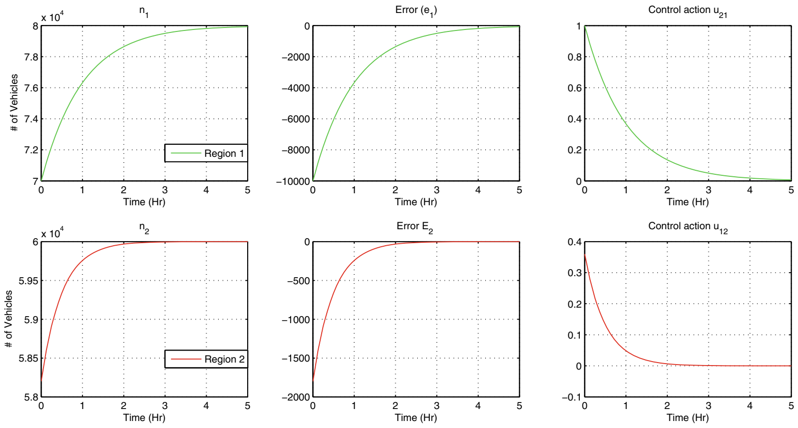

Fig. 5 Simulation results

an inflow, for instance, at a bridge to the entire region. Similarly, at a lower level, the traffic signals and ramps inside the region can be used to maintain a smooth flow, creating a uniform traffic density in that region.

6 Conclusions

This paper explored a new approach for creating subregions for an area for traffic control based on the application of the PageRank algorithm from the complex networks theory to identify important nodes, followed by the application of the Voronoi diagram algorithm. The approach then used MFD for each subregion and used feedback control design on the simplified network model. An example problem was studied illustrating these steps applied to that problem, followed by a simulation performed in a two-region network that uses a novel feedback linearization control for perimeter control. The results presented in the paper are preliminary and require a large-scale case study to validate the outcomes. Furthermore, we did not perform a check to validate the existence of MFDs and sub-MFDs in the regions, which could be a potential future research direction.

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Meta-Analysis of the Methodologies Used for Road Accident Costing and Conceptualizing Framework for Road Accident Compensation

Adil Ata Azmi and Sewa Ram

Abstract Road accidents have long-term negative consequences for society and have significant economic impacts, there are several techniques for valuing road accident-related expenses. Those from lower socioeconomic groups are more likely to be involved in accidents. In India, there is a wide range of accident scenarios at the state and city levels. About half of India's union territories and states (45%) have a fatality risk that is higher than the national average. Majority of developed and emerging countries are focusing on road safety to reduce fatalities. In India, approximately 1.5–1.6 lakh people die each year as a result of vehicle accidents. As a result, it requires immediate attention in order to take the essential steps to improve the deteriorating situation by exploring and implementing the best method which can value the detrimental impacts of road accidents. This study will try to collate the different methods of road accident costing in literature, based on their advantages, disadvantages, and applicability, and will formulate a framework base, on the most suitable method for developing countries, with a focus on India. This paper will also address the key component of traffic accidents and compensation.

Keywords Meta-analysis · Methodologies · Accidents · Costing · Compensation

1 Introduction

The human population is ever-increasing and so is the need to travel, As the world's population grows, so does the number of automobiles on the road. As per [1], India has about 300 million vehicles. The number of automobiles has increased at an exponential rate, WHO [2, 3]. As per [4], a road accident is defined as an "Unplanned event in a chain of planned or controlled events". While Deleon et al. [5] have defined

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a road accident as error with sad consequences. Most, if not all road accidents are a consequence of human error and come with a human cost attached to them [5]. As per the stats of World Health Organization (WHO), road accidents claim the lives of an estimated 1.3 million people each year, worldwide. Non-fatal injuries are estimated to affect 20 million to 50 million people worldwide, with the majority of them becoming disabled [2]. According to WHO projections for India, for the year 2019, 1,54,732 deaths and 4,37,396 injuries were reported annually [2]. According to Ministry of Road Transport and Highways (MoRTH), 1,50,785 people were killed and 4,94,624 people were injured in India in the year 2018 [6]. As per [7], India has the highest number of fatalities due to road traffic accidents as compared to any other country, which also confirms the finding by [8, 9]. Another estimate predicts that, in India, the deaths due to road accidents are expected to cross the mark of 250,000 by the year 2025 if no immediate, preventive measures are taken [10]. Road accidents and deaths vary by age, gender, month of the year, time, and even social background in India. The 30–59-year-old age group is the most vulnerable. Males are more vulnerable to traffic accidents than females [10]. Accidents are also more likely to occur during times of extreme weather and during working hours. Although mobility due to transportation systems has made our daily lives easy but it does attach the cost of lives lost as a result of traffic accidents. Study by [5] referred to vehicles as "High-velocity moving lumps of metal" and their impacts as "Weapons of mass destruction" [11]. The deaths due to road accidents cost India about 1.5 lakh lives annually, which is rarely discussed [6]. Even if they are discussed, their scale and impacts are very less, and due to this reason, road accident deaths can also be referred to as a "Hidden Pandemic" as per [12]. The research paper is focused to draw attention to this "Hidden pandemic". This research will highlight different methods used for road accident costing and will try to formulate the best suitable method for Indian conditions. The next section, of the research paper, discusses the methodology, followed by, the detrimental socio-economic effects of road accidents, and economic models that are currently used with the pros and cons in detail in the subsequent sections. This research paper is an attempt to highlight the gaps in the literature and to come up with a framework to address the research gap in the Indian context. This paper will also address the important aspect of compensation. Figure 1 shows the death rate and number of vehicles over the years.

2 Research Methodology and Data Collection

Various research papers were collected relevant to the present research. Research papers were collected by searching journals from open-source web databases with keywords like road accidents, costing mechanism, compensation, meta-analysis, methodology, accidents, etc. It included published journals, reports by national and international organizations, thesis, and books. The keywords were searched from different academic open sources like Scopus, TRID, WOS, PubMed, etc. This paper is based on a thorough review of more than 304 such articles that were initially

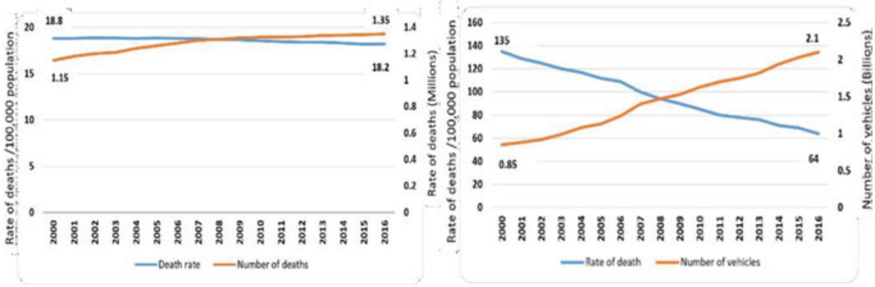


Fig. 1 The first graph shows the number of death and death rate across the globe while the second graph is a relation between the number of vehicles and the death rate worldwide [2]

identified, out of those articles 290 were selected and duplication of the article was avoided, out of the 290 articles 166 were not directly related to the topic, and out of 124 relevant articles, 11 journals could not be assessed and the permission for full text was denied, majority of them were case studies from China. A total of 107 articles from the literature were short-listed after reading the full texts. 95 articles have been mentioned in the reference section and 63 of them have been cross-referred in this article, as and when required. The flowchart for shortlisting the works of literature has been shown in Fig. 2. Figure 3 shows the year-wise publication included in the article while Fig. 4 shows the publishers, whose articles are included in the research paper. The bibliometric network of the keywords has been shown in Fig. 5, along with the domains from where articles were included in the research paper like Road Safety, Traffic Engineering, Economics, Medical, Civil Engineering, Accident Analysis & Injury Prevention, Social and Applied Science.

Conventional analysis with systematic literature review (SLR) was done using PRISMA analysis, [5, 9, 13, 14] the methodology followed was to read the literature,

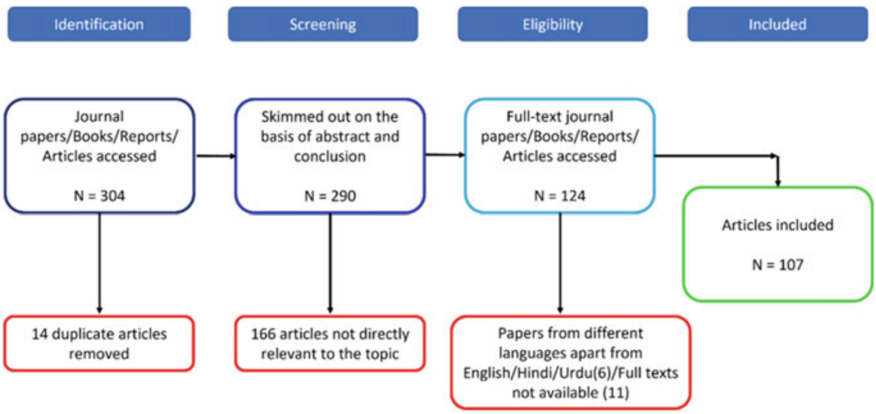


Fig. 2 Flowchart showing how the literature were shortlisted

Fig. 3 Year-wise publication of the articles

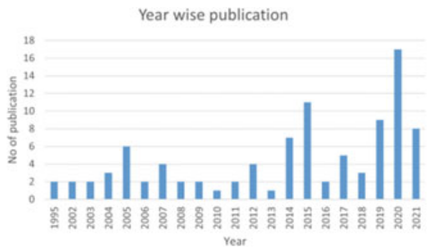
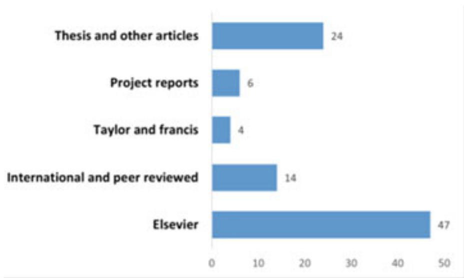


Fig. 4 Publisher of articles included in the research paper



highlight the key points and add them to an Excel spreadsheet, along with the author's name, Journal/conference name, year of publication, etc. Based on this, papers with similar conclusions were clubbed together and the results were produced. On the basis of this, the gaps in the previous research have been pointed out along with the future scope of work. Figure 6 shows the country-wise contribution of the research paper to the present article.

3 Accident Scenarios

3.1 Accident Scenario Across the Globe and in India

There have been a lot of cases where the accident victim's life was completely devastated after an accident because of his or her dependence on others and restricted mobility. In case the victims succumb to a fatal injury then it is the family members of the deceased who have to cope through phases of trauma and mental torment. Ultimately, it causes huge monetary loss to the nations, and then there are some losses that are intangible that cannot be gauged precisely in monetary terms. About 70% of the victims who lost their lives in road accidents are from developing countries and, out of them, pedestrians account for 65% of the victims, with minors accounting for 35% of those killed on the highways [15]. Based on reports WHO 2018 report [2], road accidents were among the top 10 (at position 8) causes of deaths globally in 2016. Road injury was the number one cause of death due to "Injuries". Research by

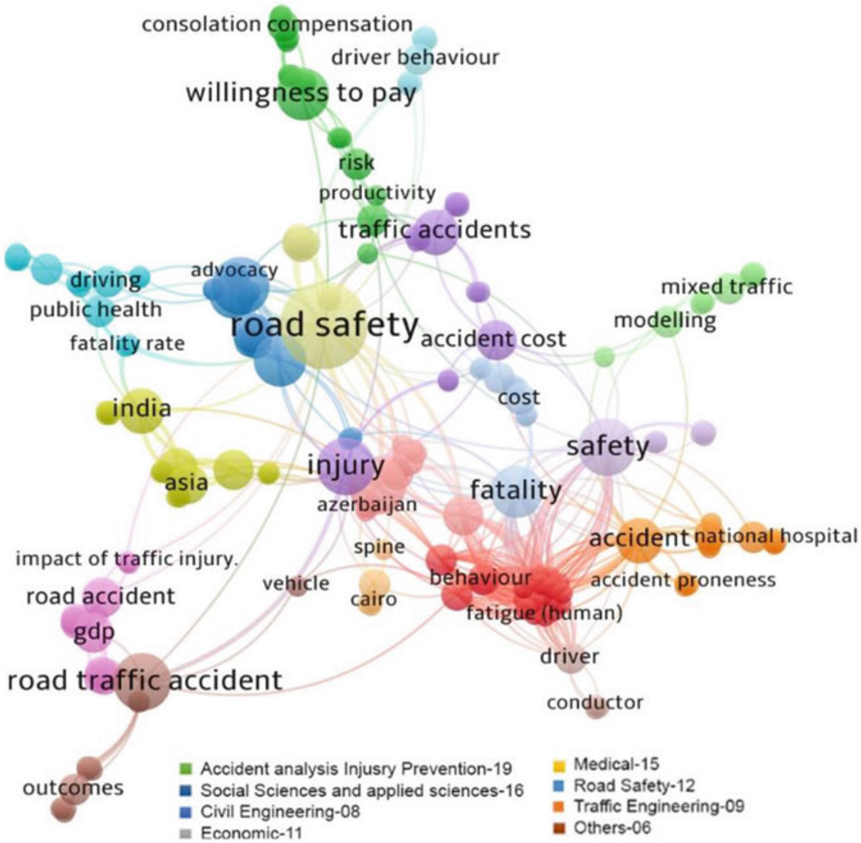


Fig. 5 Analysis of keyword co-occurrence network and domains from where the papers are included

Fig. 6 County wise contribution of the research papers

[16] suggests that the number of people killed in automobile accidents each year (last 5 years) is estimated to reach 5 million, while the count of people injured could be as high as 50 million [16]. Traffic crashes are also expected to rise from the ninth to the fifth largest cause of death by year 2030, resulting in about 2.4 million deaths per year unless quick action is taken [16]. Road accidents result in a significant amount of economic loss both on a personal and national level [17] Apart from the safety issue, it creates health, social and economic problems as well, not only for the victim but also for the family members of the victim [18, 19]. The majority of these fatalities occur in developing nations (74%) and undeveloped countries (16%), which is both a public health and socioeconomic development issue [20]. In India, every hour, there are 15 fatalities and 53 injuries due to road accidents [18]. While in many countries, the situation is improving, the situation in India is actually worsening as claimed by Sahu [18]. Tunnels are one of the most vulnerable accident locations, the drivers often become so focused while entering the tunnel that they completely miss out on the information that are provided by the signages and- markings. The rate of accidents is highest at the beginning of the tunnel and it diminishes as one proceeds further in the tunnel [21]. It has been pointed out that apart from tunnels intersections are the most vulnerable points of conflict, and the severity of accidents increases as and when the volume of vehicles and the angle of collision increases [22–26]. Research favours the notion that the identification and treatment of black spots is one of the options for lowering accident costs [27, 28]. Analysing the road accident pattern by mapping the previous accidents can significantly help in future planning and designs of roads that will help in reducing accidents and the costs attached to it [13].

3.2 Types of Road Accidents and Losses Attached to Them

A road accident death is defined in India as "death(s) occurring within 30 days of a road traffic accident". Fatal accidents, serious injury accidents, major injury accidents, grievous injury accidents, minor injury accidents, and no injury accidents (Property damage only accident) are the six types of road accidents [6]. "A fatal accident happens when one or more people are killed as a result of the accident within 30 days of the accident occurring" [6]. "A serious injury accident is defined as one in which a person is detained in a hospital as an "in-patient" or one in which a person sustains any of the following injuries, whether or not he or she is detained in a hospital: fractures, concussions, internal injuries, crushing, severe cuts and lacerations, or severe general shock requiring medical treatment and the victim requires ICU admission", "A major injury accident is defined as either a person being detained in a hospital as an "inpatient," or if any of the following injuries are sustained whether or not he or she is detained in the hospital: fractures, concussions, internal injuries, crushing, severe cuts and lacerations, or severe general shock requiring medical treatment but not ICU admission. The grievous accidents include serious and major injury accidents, While the "Minor accident is one in which there are no deaths or serious injuries but a person is slightly injured. This will be an injury of minor nature such as a cut, sprain, or bruise, where only first aid is required and does not require hospitalization". A damage-only accident, also known as property damage only (PDO) accident, occurs when no one is hurt but the vehicle and/or property are damaged. Many academics have previously stated that while road accidents cause economic loss on a personal and national level, the effects of road accidents also have physical and emotional ramifications [17, 18, 29, 30]. Psychiatric symptoms and illnesses are common after grievous or minor traffic accidents, according to Richard, Bridget, and Robert. Apart from safety issues, accidents have become health, social and economic problem as well [18, 19]. An effective strategy to save life losses due to road accidents is by providing emergency medical services to the victims, however, too many centres often reduce the quality of service and it rather increases inefficiency in the system [31]. Thus, it is important to provide timely medical service within the "Golden hour". 15–44 years are the most productive age group [2]. Men have 2.5 times more chances of an accident as compared to women [24, 32]. In all sorts of accidents, the 16–25 age group bears the greatest negative impacts of a road accident [11, 30, 33–37]. The most venerated age group is 30–59 years old [10]. In low-income nations, the probability of fatality from road accidents is three times higher than in high-income countries [31]. Ironically, countries with a poor safety record and greater death rates place a low monetary value on victims, whereas countries with a better safety record place a higher monetary value on victims [38]. Because of a lack of statistics, the economic loss caused by accidents in poor nations is frequently overlooked [29]. The lower-middle-class section of society reports the maximum number of fatalities, and this statement is true for all countries. The social class of the parents has a strong relationship with their children meeting with an accident [39, 40]. To assess the economic losses due to accidents, five major studies have been conducted in India previously apart from various other research papers on this topic. A brief of each of the previous studies has been given below.

3.3 Major Economic Studies Conducted in India as a Case Study

The first major study was conducted in the year 1982, sponsored by the World Bank as the road user cost study. This was the first large road user cost study, and it was based on data from the city of Delhi. Medical bills, legal fees, property damage, insurance costs, and lost output due to death were all included in the costs. The cost of road accidents was projected to be 0.29% of GDP. The Human capital (HC) cost method was used. In the year 1995, Transport Research Laboratory (TRL) did a study under the name of Overseas Road Note 10 for costing road accidents in developing countries. It did surveys in four countries across the world, including India. Suggested a method for calculating accident costs for developing countries. The methodology was loosely based on the HC method with a slight modification, and recommended it as a method to be used in developing countries. In India, the Accident Cost Study was sponsored by the Ministry of surface transport (MOST), which was titled Research Scheme R-79, in the year 2000. It used Gross Output Approach to calculate the value of road accidents. Data coverage included Insurance data from 16 cities across India and one government hospital data. The economic cost of road accidents in India was estimated to be 0.69% of GDP. Around the same time in the year 2000, another study was conducted as a part of the Sundar Committee of India. The study was outsourced to Tata Consultancy Services (TCS), which was the Working group estimate. It is estimated a loss of around 3% of the country's GDP or about $550,000 million in 2000, or $22,08,021 million in 2022, taking into account previous inflation rates. Till today many evaluations of road accident costs are made based on this report, along with using the wholesale price index. The latest work on road accident costing in India (was done by S. B. Paul, Transportation Research and Injury Prevention Program (TRIPP), Indian Institute of Technology Delhi (IIT-D), and Delhi Integrated Multi-Modal Transit System (DIMTS) [41]. 20 state capitals were included in the case study (2 trauma centres per city were surveyed). 250 samples from each city hospital were collected, overall, 5000 samples were collected. Calculated the cost of fatal accidents in India as ₹9.1 million per fatality, which is the social cost of an accident.

4 Review of Methodologies for Road Accident Costing

4.1 Timeline of the Methodologies Used for Road Accident Costing

A timeline of the various methods for road accident costing has been given in Fig. 7. It is evident that studies on traffic accidents and their costs did not become sensitive until the 1960s. The most common and most widely used method was the HC method. In the 1970s, it got criticism from various economists as they claimed that its mechanism is inconsistent with the cost–benefit analysis (CBA) principles, in 1982, first road user study was conducted by WHO and it used the HC method. In the 1990s, the developed countries began shifting toward the willingness to pay (WTP) method as they found the method to be more theoretically sound. In 1995, Transport Research laboratory (TRL) conducted another study and suggested a mechanism for the calculation of the cost of road accidents, which was loosely based on the HC method. In 1995, various other methods were explored and in the early 2000s, the researchers suggested that socioeconomic factors should also be considered while calculating the cost. In the 2020s, hybrid methods are being used to assess the cost of road accidents although the number of case studies are limited as it is a fairly new method.

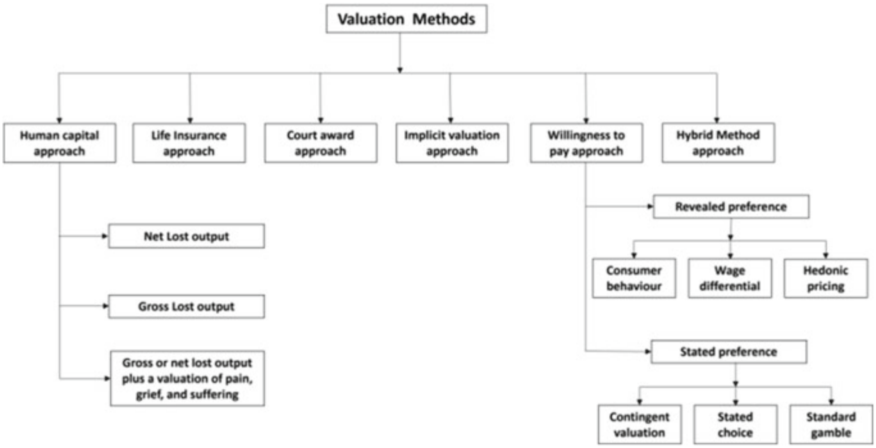


Fig. 7 Flowchart of the various methodologies for the evaluation of road accident cost

4.2 Costing

The values of economic losses come out to be different as different studies have taken different parameters under consideration along with the different methods of calculating the road accident cost. Some of the most common methods that are most commonly used across the globe are namely: human capital approach (HC), the life insurance approach, the court award approach, the implicit public sector valuation approach, the values of the risk-change approach or the willingness-to-pay approach (WTP), and the hybrid method. The different methods used across the globe have been shown in Fig. 8. Hills and Jones-Lee [42] suggest that only two road traffic accident costing procedures, the Human Capital (HC) method and the Willingness to Pay (WTP) method, appear to be directly relevant among these methods. When examining a country's wealth, the HC approach (Gross Output method) is appropriate.

If social welfare is a concern, the willingness to pay method is more suited. The Gross Output technique is recommended by TRL (Transport Research Laboratory, UK) to cost road accidents in developing nations like India. Professionals working

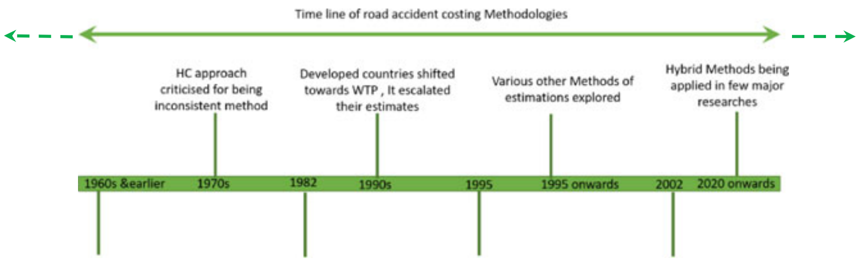


Fig. 8 Timeline of road accident costing methodologies

in this field, such as E. Hauer, believe that putting a monetary value on human life is unethical [43]. Miller [44] divided the losses resulting from traffic accidents into the following categories as shown in Fig. 9 [45–48]. This method (HC) assumes an arbitrary value for the loss of grief and pain. Economists also argue that this is not an appreciable method to gauge grief loss and pain. It is more suitable when the priority is national welfare and macroeconomic parameters are to be considered. In the 1970s, HC method was criticized for not being consistent with CBA (Cost–Benefit Analysis) principles. But it is one of the easiest and most widely used methods for developing countries. The majority of the studies conducted in the Indian context have used this method. The victim's future consumption is subtracted from the gross output value in the net output approach. It can also be stated that the cost of an accident is equal to the "gross production" amount less the discounted worth of the victim's consumption. Because the future consumption of persons killed in road accidents is deducted, the Net output model has a more conservative economic cost to society. The approach was originally used to evaluate the total net increase to a country's stock of wealth created through production during an accounting interval, rather than to calculate the economic loss due to accidents. This method is used for accident cost calculation in limited studies, it is difficult to find studies in the Indian context that uses net output method. The life insurance technique calculates the total cost of real resources and the price that average people are ready to pay to guarantee their own lives or limbs. It is quite commonly used but not an advisable method to gauge economic losses because not everyone gets themselves insured by insurance companies. The poor section of the society that is often the most affected by road accidents does not actually have insurance and thus this approach has its limitations. Another drawback of this method is that data availability is a challenge as insurance companies do not provide it as an open-source. Eighty percent of accident victims in India are uninsured [12]. The court award approach uses some rationales to value the lost life, and then that amount of money is given to the victim; in the case of a fatal accident, the victims' relatives can file a case. It is considered an indicator of the cost that society associates with the fatality, or the value that society would have placed on its prevention. Aside from these charges, the cost of real resources is added to this sum to arrive at the cost of an accident. This method is an amalgamation of court awards and other prevalent accident costing methodologies. The verdicts are often used to compensate or calculate the economic losses due to road accidents for future cases of similar intensities. The implicit public sector valuation method tries to put a monetary value on the implicit costs and values of accident prevention in safety legislation or in public sectors. The implicit public sector valuation method employs a set of implicit values to determine the worth of human life.. The willingness to pay method, also known as the value of risk change approach method, seeks to determine how each investment in road safety reduces the likelihood of an accident. It represents society's readiness to pay to avoid the deaths, injuries, and property loss that result from traffic collisions. Individual and societal willingness to pay can both be calculated by this method. It can be done using the stated preference (SP) and revealed preference (RP) methods. There are several methods under the SP and RP methods, and some of the most well-known ones are listed below. In the Contingent

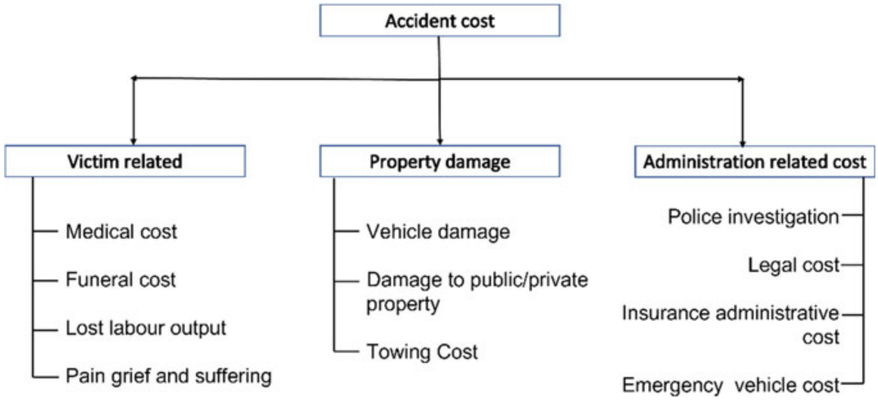


Fig. 9 Components of road accident cost (HC method) as per miller

valuation method or bidding game survey, respondents are asked their willingness to pay for a certain public good in a given situation. He revealed preference technique attempts to analyse the individual's or society's behaviour towards risk and includes a set of questions to gauge the same. Hedonic pricing is used to increase or decrease in benefits of road users, by asking them a bundle of questions. This bundle can have various possibilities of choices. In standard gamble, respondents are given a scenario that which their involvement in a hypothetical road crash results in them having a choice of treatment based on their risk for their outcome. Wage differential method tries to look into how people behave when provided with extra money in exchange for a riskier job. Consumer behaviour tries to look into how the user decides to buy or not to buy based on the price, i.e., a similar technique is said to assess the road safety and price attached to it (Table 1).

Table 2 shows a matrix between the various methodologies of road accident calculation and the parameters that are considered for accident costing in the various literatures, the green colour shows the commonly used parameters, the yellow colour is designated for the lesser-used parameters, while the red colour symbolizes the parameters that are rarely considered.

5 Compensation

Once a road accident has taken place, it is important to look into the compensation aspects, in order to stabilize the victim, and in case of a fatal accident a "fair Compensation" needs to be calculated in order to stabilize the affected family members of the diseased. As discussed earlier, road accidents may result in acute cognitive disorders in addition to emotional distress and often cause substantial problems in everyday life. In India, the problem is even more grievous as the whole family gets involved when a road accident is recorded, which completely destabilizes the whole family

Table 1 Comparison table of the various methodologies used across the globe

Method	Advantage (s)	Disadvantage (s)	Data required	Mathematical model	Stakeholder	Widely used in
Human capital approach	Easiest and most widely used method to be used More suitable when the priority is of national welfare and macroeconomics is to be considered	Not an advisable method to gauge grief loss and pain Assumes an arbitrary value of pain and grief Widely criticized by economists after the 1970s for not being consistent with (Cost-benefit analysis) CBA principles	Property damage, administrative cost, medical cost, lost output	Cost-benefit analysis Multi-criterion decision models	Victim Government body Employer Hospitals	Asian and other developing and under developed countries
Life insurance approach	Based on the valuation by the insurance company and insurer, the victim gets compensation. Generally, the quickest method for receiving compensation	Not everyone gets themselves insured The poor section of the society, that forms a major portion of the victims, does not get covered, i.e., about 80% [10]	Insurance policy, FIR, an estimate of vehicle repair, indoor patient documents, other related medical bills, death certificate in case of a fatal accident	Model dependent on stochastic and rarely deterministic statistical models	Victim Insurance firm Police Hospitals	Developed and developing countries

(continued)

Table 1 (continued)

Method	Advantage (s)	Disadvantage (s)	Data required	Mathematical model	Stakeholder	Widely used in
Court award approach	Does not work on fixed rules or formulas, works on guidelines, which can favour the victim Uses past verdicts to decide the valuation of the road accident cost	Often takes too long to settle cases Due to long durations, the victims/their representatives become disheartened and opt out of court	Extent of damage, income of the diseased, might consider "Future prospects"	It may or may not be based on some previously discussed mathematical models or simply the amount the plaintiff seeks	Victim Relatives of victim Police second party Insurance companies Government	Almost every country
Implicit valuation approach	Focuses on looking into the implicitly placed values on accident prevention	In the valuation of the cost of a road accident, it has a limited use	The extent of damage, maintenance cost, depreciation cost, and service life	Simply subtracts the explicit costs from the total cost/revenue and then adds that as cost of implicit component of road accidents	Victim insurance firm	European and American countries

(continued)

Table 1 (continued)

Method	Advantage (s)	Disadvantage (s)	Data required	Mathematical model	Stakeholder	Widely used in
Willingness to pay approach	The best method to gauge the intangible factors like the value of pain and grief It is the most theoretically sound method for the valuation of the human cost	Complex method to be applied. Respondents may mention a price for which they are unwilling to pay	Damage extent to target group, stated and revealed preference method to assess the level of pain and suffering and other intangible parameters	Probit model, Multi-criterion decision models	Victim Employer Government hospital	Australia, European and American countries
Hybrid method approach	Combines two methods to get the best of both worlds Eliminates the disadvantages of HC and WTP methods	No major disadvantages have been found as of now Very limited studies have been done as it is a fairly new method (2020)	Property damage, administrative cost, medical cost, Lost output. Stated and revealed preference method to assess the level of pain and suffering and other intangible parameters	Uses a combination of above methods Multi-criterion decision models and probit models	Victim government body Employer Hospitals Insurance companies	Limited studies across the globe mainly from Asian countries

Table 2 A matrix of the methodologies and the parameter used for accident costing

Parameters / Methods	Victim related costs	Property damage	Administration related costs	Family-related costs	Future prospects	Opportunity costing	Value of life	Differential wage	Impairment specific productivity losses
Human capital cost	Commonly used	Commonly used	Commonly used	Lesser used	Rarely used	Rarely used	Commonly used	Rarely used	Rarely used
Life insurance approach	Commonly used	Lesser used	Lesser used	Lesser used	Lesser used	Rarely used	Commonly used	Rarely used	Rarely used
Court award approach	Commonly used	Lesser used	Lesser used	Lesser used	Lesser used	Rarely used	Lesser used	Lesser used	Rarely used
Implicit valuation approach	Lesser used	Lesser used	Lesser used	Lesser used	Rarely used	Rarely used	Lesser used	Rarely used	Rarely used
Willingness to pay method	Commonly used	Lesser used	Lesser used	Lesser used	Lesser used	Rarely used	Commonly used	Lesser used	Rarely used
Hybrid method for valuation	Commonly used	Commonly used	Commonly used	Lesser used	Rarely used	Rarely used	Commonly used	Lesser used	Rarely used

Commonly used parameters Lesser used parameters Rarely used parameters

especially if the victim is the sole bread earner. Compensation is often a sum of money that compensates only the direct losses completely neglecting the effects on "opportunities lost", ambitions, and loss of quality of life [41, 48–52]. Compensation is often a frustrating and lengthy process, and people settle for a derisory amount just because they become fed-up with the process, many a time it requires interrogation and re-recording of the accident which further causes mental torment to the victim or their family members, which often degrades the overall health of the diseased and its dependent family members [52, 53, 54–58]. It's not easy to put a monetary value on life, it's hard to capture the dynamic nature of costing and compensation with a single statistical or mathematical model. Kenneth R. Feinberg a compensation expert quotes and emphasizes that it's difficult and debilitating to assign value to life [59, 60]. It was based on the experience of, the Special master, who himself along with his team assigned a monetary value to the death of each 9/11 victim in the USA. He suggested that the value of life equals victims earning over a work-life plus economic loss due to being unable to work and add to that the value of pain and suffering. The formula on paper seems a bagatelle, but in practice, it isn't. There isn't something like "One size, fits all" when it comes to costing and compensation [61–63]. India needs to make its unique model that incorporates suitable parameters and hands out compensation that stabilizes the victims and their family members. It has also been found that the acknowledgment of the pain and suffering that the victim or family members are equally important. The compensation should not be too less that it is unable to support the family members, nor it should be too much that one treats it as a lottery. It has been found that in the case when compensation is too much, people often fake their case in order to get that compensation amount [64]. As in the current scenario if one searches for a road accident compensation mechanism in India then either a court case will open up or the Motor accident claim tribunal (MACT) guidelines will appear, which again are mere guidelines and not fixed formulas or rules so that one can easily find the amount of compensation. Thus, a strong mechanism needs to be developed for accident compensation, especially for developing countries like India.

6 Results and Discussion

6.1 Limitations and Gaps in the Previous Studies

Currently, an updated version of the study titled, "Evaluation of Road Accident Costs" conducted by TCS in 1999 is being used to often decide compensation. This study has few drawbacks and thus the estimates won't be accurate. Like life expectancy, medical expenses, and valuing the loss of livability had some discrepancies in them, and thus the estimates are not justifiable. As per [55] still "Very few studies have been carried out in India and it is lacked in area coverage and accuracy" [55]. The court award method takes a lot of time and suggests that valuation should be done keeping in view the "Future prospects" of the diseased. But often the valuation is not accepted by the victim or the family members of the victims as it is believed to be quite low by the litigants. Many a times when the verdict is announced half of the people involved in the case are dead. Limited efforts have been made to account for the differential in wages of the different people in the society. Some other social factors that arise in Indian families like loss of opportunities have not been accounted and they still need to be incorporated in the compensation models. Among many methods, there is a debate about which method is most suitable for India. Monetary calculations of pain and grief are still in question and are rarely addressed. None of the methods so far has scientifically analysed the "lost opportunity costs". Dependent family members are often ignored while handing out the final compensation.

6.2 Proposed Conceptual Framework

As discussed in the previous sections of this paper, there are various methods that are used to estimate the cost related to road accidents that can be further used for estimating compensation to the accident victims. There have been a few limitations to each of these methods. This research proposes a conceptual framework for estimating purely the compensation portion for road accident victims that can be used to evaluate the conceptual compensation amount. As pointed out earlier, this framework is developed keeping in view the Indian conditions into the picture so that a "Fair compensation" should be given in a short span of time. This research proposes a name to this compensation equation, i.e. "Ramadilian Equation". The flowchart of the conceptual framework has been given in Fig. 10.

$$\begin{aligned}&\text{Ramadilian equation for compensation} =\\&\text{Restitution cost (C)} + \text{Victims related cost (X)}\\&+ \text{Property damage cost (Y)} + \text{administrative cost (Z)}\end{aligned}$$

where

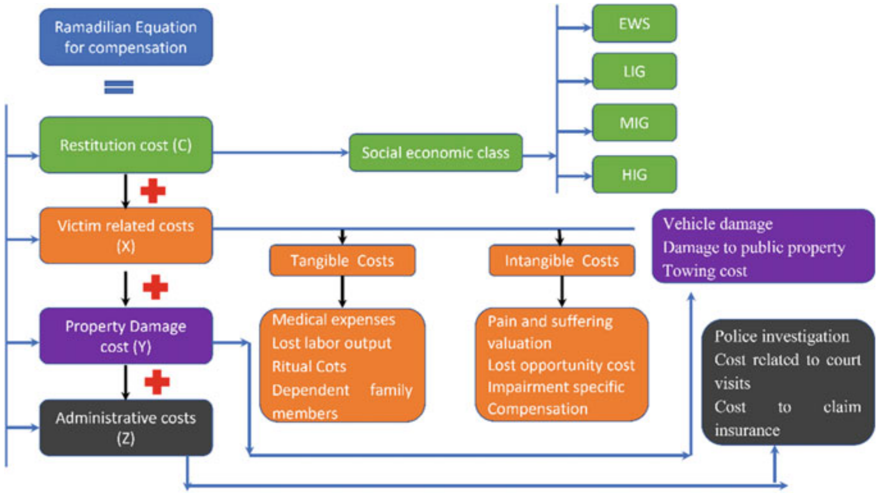


Fig. 10 Conceptual flowchart of the proposed framework

Restitution cost (C) = Minimum amount of money that should be given to the victim so that they can get their personal and medical expenses covered (especially recommended for low-income and daily wage earners). Irrespective of their fault as this paper believes, saving a life is the ultimate goal. This amount is currently ₹ 50,000/(At max) for grievous accidents and ₹ 2,00,000 (At max) for fatal hit and run cases as per MoRTH guidelines 2022. This paper also proposes that this amount should be inflated as per the inflation rates and at least the medical expenses should be covered for the victims, even if the expenses go beyond the proposed amount of MoRTH.

Components of victim-related cost (X): The components of the variable X have been listed in the above flowchart below along with elucidation of whether they are tangible or not. These are the factors that should be given the maximum weightage as they play the most crucial role in deciding the "Fair compensation".

Property damage cost (Y): This variable portion of the equation deals with the costs related to public or private property damages in case of loss of public property, that amount shall be deducted from the compensation amount.

Administrative cost (Z): This variable cost deals with the costs incurred during the administrative formalities, that includes amount spend on police investigation, visits to courts, emergency vehicle, etc.

7 Conclusion

Road accidents are something that has become completely unavoidable. They come along with a lot of social costs attached to them, there are various methods that are used across the globe to measure the economic losses due to road accidents. It has been pointed out that the Human capital costs method and the Willingness to pay method are the most relevant methods when road accident costing is in question. In India, the majority of the studies have used the HC method while limited studies are found using the WTP method. However, after going through all the literature, a fairly new method called the "Hybrid method" seems to be best for a country like India, where we have to look into pecuniary and nonpecuniary aspects. Extending it further this paper proposes Ramadilian equation model for calculating the compensation amount. Compensation needs to be calculated precisely in a scientific manner. Thus, the majority of road accidents should be investigated and compensated accordingly. Compensation can be paid directly to the victim in case of a major and serious accident or it can be provided in the form of credits and those credits can be used for education and medical bills of the victim and their family members. In case of a fatal accident, the immediate family members should get compensation. The maximum time duration for the compensation amount should be in two parts, one as immediate relief which should not take more than 48 h. And the second part should be final compensation that should be handed over in maximum of 2 months. As the compensation has lots of variables the amount of compensation needs to be calculated accordingly. The Ramadilian equation tries to keep all these aspects into the picture and hand out "Fair compensation" in a time- bound manner.

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Investigating Pedestrian Crash Risk at Unsignalized Midblock Crosswalks on Arterial Road

Shubham Thapliyal, Heikham Pritam Singh, and RB. Sharmila

Abstract The increase in pedestrian deaths in road accidents across the developing countries like India is a growing issue of concern. Statistical data indicate that most pedestrian fatalities occurred at midblock crosswalk locations. Pedestrian crossing behaviour data were collected using videography at unsignalized midblock crossing on arterial road. Effect of waiting time, accepted gap, vehicle speed and vehicle type will be considered. Age group, gender, luggage in hand, glancing before walking, all these variables will also be considered in modelling. Reduction in speed of vehicle will also be analysed. The present study will analyse safety margin of pedestrian-vehicle conflict under mixed traffic condition at uncontrolled midblock crossing on arterial road. The study will also use both multiple linear regression and the Machine Learning algorithms to model safety margin and assess significant factors affecting it. Rolling behaviour's effect on pedestrian safety margin will also be analysed.

Keywords Midblock crosswalks · Spatial gap acceptance · Safety margin · Vehicle type · Machine learning algorithms · Multiple linear regression

1 Introduction

Pedestrian deaths in India have gone up from 13,894 in 2015 to 23,483 in 2020, as per the Union Ministry of Road Transport and Highways. The increase in pedestrian deaths in road accidents across the developing countries like India is a growing

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issue of concern. It indicates the worrying trend of planning and development of road infrastructure tilting in favour of cars and heavy vehicles. There is a need of giving more attention towards vulnerable road users like pedestrians. Road crossing is considered as the most dangerous location for pedestrians. Main locations for pedestrian crossing are: the midblock crosswalks and intersection (signalized or unsignalized). Statistical data indicate that most pedestrian fatalities occurred at midblock crosswalk locations. Traffic control devices (pavement marking, traffic signals and pedestrian sign board) are used to increase safety of pedestrian at midblock crosswalks. Pedestrian refuge island is made to reduce crossing distance and divide crossing in two or more phases. Frequency and severity of pedestrian accidents are affected by location of crosswalks. Pedestrian demographic factors have an impact on crash outcomes. distraction due to mobile use, luggage in hand, crossing in pair (holding others hand), listening music also affect pedestrian behaviour while crossing. Pedestrian has to do following tasks while crossing road: walking, glancing towards approaching road, maintaining appropriate speed, judging available gap (accept or reject), and negotiating all other distractions.

Previous research has shown that some pedestrians who cross road while multitasking fail to display cautionary behaviours (e.g., looking for oncoming traffic, walking inside crosswalk boundary). Vehicle speed is considered as an important risk factor in road accidents. Traffic conditions such as traffic density, speed limits, cross-street activity, and the presence of speedometer influence the speed compliance behaviour of drivers. An arterial road is a high-capacity urban road that sits below freeways/motorways on the road hierarchy in terms of traffic flow and speed. Safety Margin (SM) is defined as the difference between the time a pedestrian crossed the conflict point and the time the next vehicle arrived at the same conflict point [1]. The objective of the present work is to analyse safety margin of pedestrian midblock crossing on arterial road.

2 Literature Review

This study investigates pedestrian crash risk with vehicles at unsignalized midblock crosswalks, where vehicles and pedestrians share section of roadway with or without conflicts in a traffic stream. The vehicle can slow down and let pedestrian cross or can accelerate in case of aggressive driving. Pedestrian gap acceptance and road crossing behaviour has been widely studied in the case of signalized intersections [2–4], unsignalized intersections [5, 6], uncontrolled midblock crosswalks [7, 8], and other areas. In the present study, our focus will be on uncontrolled midblock crosswalks on arterial road.

2.1 Pedestrians' Gap Acceptance Behaviour

Pedestrians' gap acceptance behaviour at uncontrolled mid-block crosswalks is a crucial aspect of pedestrian safety. The critical gap, which represents the threshold value of the gap acceptance/rejection, plays a significant role in determining whether a pedestrian will safely cross the road. While the arrival time of vehicles has been considered in previous studies, recent research indicates that gap acceptance depends on the distance between the pedestrian and the approaching vehicle [9].

Studies have identified several factors that influence pedestrians' gap acceptance behaviour, including gap size, crossing distance, pedestrian speed, and vehicle speed [10–13]. Vehicle type has also been found to impact pedestrian's gap acceptance behaviour [11]. Researchers have observed that the probability of pedestrian platooning increases with the size of the platoon and the traffic volume [8]. Additionally, the frequency of conflicts is likely to increase with an increase in vehicle speed [14].

Pedestrian waiting group composition based on gender, age, disability status, and luggage-carrying capacity has been found to influence gap acceptance behaviour [15]. Safety features such as pedestrian refuge islands, safety guardrails, and road markings have been found to improve the level of service for pedestrian facilities. On urban multilane roads, pedestrians may choose the rolling gap to cross the road, which can be convenient but risky, especially if traffic speed is high [16].

To investigate the factors that contribute to pedestrian-vehicle conflicts, researchers have proposed ordered probit models that consider factors such as traffic volume, vehicle speed, pedestrian crossing behaviour, and pedestrian refuge [14]. Pedestrian safety margin, defined as the time difference between the time gap of an approaching vehicle and the pedestrian's crossing time, has been used to analyse the severity of pedestrian-vehicle conflicts at unsignalized midblock crosswalks under mixed traffic conditions [17].

2.2 Research Objective

Following are the objectives of the present study:

- To analyse safety margin of pedestrian-vehicle interaction under mixed traffic condition at uncontrolled midblock crossing on arterial road.
- To use multiple linear regression and the Machine Learning algorithms like Decision Tree and Random Forest to classify risk of vehicle-pedestrian interaction based on safety margin values and to check the significant factors affecting risk of vehicle-pedestrian interaction. Categorizes them into three categories, such as low, medium, and high risky interaction.

3 Data Collection

Video camera was installed to collect pedestrian crossing behaviour data at unsignalized midblock crossing on arterial road (G.S. road) in the city of Guwahati, located in the northeastern part of India. The video data were collected in morning peak hours from 8:00 a.m. to 10:00 a.m. and evening peak hours from 3:00 p.m. to 5:00 p.m. on weekdays. For data collection, one high-definition recording camera was used with recording speed of 25 frames per second. Road geometry was four-lane divided with a 1.95 m wide median, with 7 m width of road on one side of median and 6.7 m width of road on another side of median. The posted speed limit was 30 km/h on selected road stretch. Pedestrian warning signs were present on site. Raised table top crossing was not found on site. Figure 1 shows Google Earth image of the site selected and data collection setup. Camera setup was such that it could cover pedestrian crossing midblock properly and at least 100 m of approaching traffic from Guwahati to Shilong side.

In Fig. 2, road geometry is shown. Light poles are taken as stationary points A, B, and C. Fourth stationary point is taken at the stop line before midblock crosswalk. Distance was measured between these stationary points on ground with the help of distance measuring wheel device. Distance between A & B was 33 m, between B & C was 30 m and between C & D was 20 m. Median width was 1.95 at the midblock crosswalk, which is slightly less than recommended width of median (2 m).

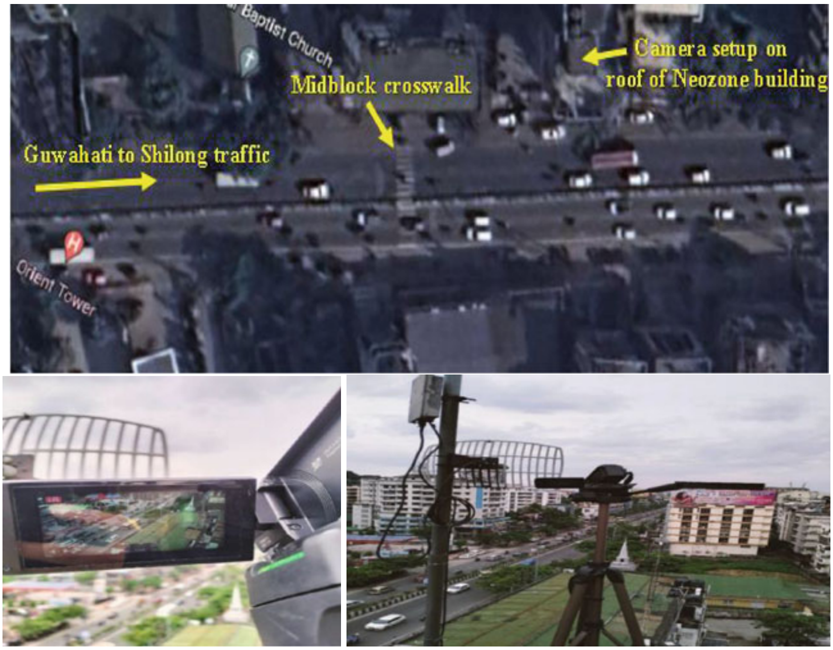

Fig. 1 Data collection setup and google earth image of site

Fig. 2 Road geometry with stationary points and trap lines on pavement

4 Data Extraction

Pedestrian and traffic data were extracted from video recordings manually. Time was noted down when pedestrian:

1. Enters at the start point of midblock crosswalk.
2. Starts phase 1 journey.
3. Reaches median.
4. Starts phase 2 journey.
5. Reaches other side of midblock crosswalk.

The following details were noted down for every pedestrian by visual inspection:

1. Age category.
2. Gender category.
3. Is he/she glancing towards approaching traffic before crossing?
4. Is he/she crossing street in group or alone?
5. Luggage in hand (hands not free)?
6. Jaywalking (crossing street carelessly or in illegal manner)?
7. Type of accepted vehicle gap (near lane or far lane).
8. Is pedestrian using mobile phone while crossing?

5 Data Preprocessing

In raw extracted data, there were both categorical and numerical variables. Table 1 shows extracted categorical variables for the modelling while Table 2 shows extracted numerical variables for the study.

In Table 2, Speed AD is average speed of vehicle between points A and D. In the same manner, Speed AB will be average speed of vehicle between A and B, Speed CD will be average speed of vehicle between C and D. Difference of Speed AB and Speed CD is also taken as a parameter Speed AB–CD, which shows reduction in speed of vehicles while approaching pedestrian crosswalk. Crossing speed at phase 1 is also taken for modelling.

Table 1 Categorical variables used in modelling

Serial number	Variables	Categories
1	Age group	1. Child 2. Young 3. Old
2	Gender	1. Male 2. Female
3	Grouping	1. Yes 2. No
4	Glancing before walking	1. Yes 2. No
5	Luggage in hand	1. Yes 2. No
6	Vehicle type	1. TW (two-wheeler) 2. A (Auto) 3. C (Car) 4. LCV (Light commercial vehicle) 5. HV (Heavy vehicle)
7	Accepted gap type	1. Near 2. Far
8	Rolling behaviour	1. Yes 2. No

Table 2 Numerical variables used in modelling

Serial number	Variables	Unit of parameter
1	Wait at start	s
2	Accepted spatial gap	m
3	Speed AD	m/s
4	Speed AB–CD	m/s
5	Crossing speed phase 1	m/s
6	Safety margin	s

Table 3 Accident risk based on safety margin value

Safety margin range (sec)	Accident risk
0–2.5	High risk
2.5–6	Medium risk
≥6	Low risk

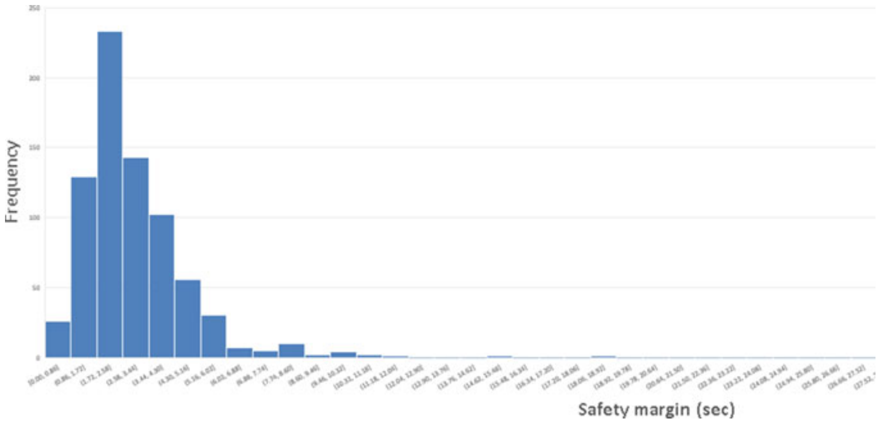

Fig. 3 Histogram of pedestrian safety margin values

Figure 3 shows histogram of pedestrian safety margin values.

Following Table 3 shows the corresponding accident risk for pedestrian based on safety margin values.

Risk of accident increases with decrease in safety margin value, so higher the safety margin value safer the pedestrian. In this study, a multiple linear regression and different machine learning algorithms are used to classify accident risk of pedestrian and their performance is compared.

6 Preliminary Data Analysis and Results

There were 754 pedestrian data points in the study, out of which 33% were female and 67% were male pedestrians. Considering the age categories, 11.3% pedestrian were children, while only 3.8% were old pedestrians and others were young pedestrians. Out of all pedestrians, 13.3% were carrying luggage.

Table 4 shows mean, standard deviation, minimum and maximum value for safety margin, crossing speed, vehicle speed, change in vehicle speed, accepted gap and waiting time. In data analyses, average safety margin was found to be 3.03 s. Average crossing speed of pedestrian was 1.163 m/s. For the leading vehicle, average reduction in speed while approaching to midblock crosswalk was 0.279 m/s (1 km/h). So, it was concluded that vehicles slow down very slightly while approaching to midblock crosswalk. Average waiting time was 9.3 s for the pedestrian during morning and evening peak hours in our study.

When leading vehicle was in far lane, average accepted gap for the pedestrian was 38.25 m and when it was in near lane, average accepted gap was 27.10 m. It concludes that pedestrian has tendency to accept bigger gap when leading vehicle is in far lane. We can also say that pedestrian gap acceptance behaviour changes as per the position of leading vehicle in different lanes.

Average safety margin value when the approaching vehicle speed was <28 km/h was 2.65 s, but when vehicle speed was ≥28 km/h, average safety margin was 3.26 s. So, we can conclude from the study that safety margin value increases as the approaching vehicle speed increases. Posted speed limit for the study area was 30 km/h. Crossing speed for the male pedestrian was slightly more than the female pedestrian. For male pedestrian, it was 1.19 m/s and for female pedestrian it was 1.11 m/s.

Average accepted gap for the pedestrian who had luggage in hand was 32.6 m while for pedestrian without luggage it was 31.6 m. So, we can conclude that when pedestrian hands are not free they tend to accept larger gap. Average crossing speed for children was found 1.06 m/s, for young pedestrians 1.18 m/s and for old pedestrian 1.12 m/s.

In this analysis of pedestrian safety during road crossing in midblock crosswalks, we evaluated the performance of five different classification models: Decision Tree, Random Forest, and Gradient Boosting. The models were evaluated based on their accuracy, precision, recall, and F1 score. Our results indicate that Gradient Boosting had the highest accuracy (80.98%), followed by Random Forest (77.04%), Decision Tree (67.54). The precision, recall, and F1 score for all the models were relatively similar. These findings suggest that Gradient Boosting and Random Forest may be the most suitable models for predicting pedestrian safety during road crossing in midblock crosswalks. However, additional studies are needed to optimize model

Table 4 Descriptive data analysis

	Safety margin (s)	Crossing speed (m/s)	Speed of vehicle (m/s)	Change in speed (m/s)	Accepted gap (m)	Waiting time (s)
Mean	3.03	1.163	7.4	−0.279	31.69	9.3
Std	2.23	1.084	2.8	1.37	21.6	12.25
Max	39.64	29.17	33.7	7.6	100	88
Min	0	0.192	0.02	−14.5	5	0

Table 5 Comparison of classification model performance using common evaluation metrics

Model name	Accuracy	Precision	Recall	F1 score
Decision tree	0.6754	0.6752	0.6754	0.6750
Random forest	0.7704	0.7702	0.7705	0.7700
Gradient boosting	0.8098	0.8096	0.8098	0.8091

performance and explore other factors that may influence pedestrian safety. Overall, our analysis highlights the potential of machine learning models for improving pedestrian safety and guiding the design of effective safety measures in midblock crosswalks.

7 Future Scope

In this study, posted speed limit is 30 km/h on the arterial road, in future studies, midblock crossing on higher speed limit roads can be analysed. Age group and gender are visually observed in this study. Age is categorized only in three categories: young, child, old and gender are categorized: male and female. In future studies, detailed demographics of pedestrians can be included in the study.

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Economic Benefit Assessment of Black Spot Improvements

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Abstract Road transport remains the most favoured mode of transport for both freight and passenger movement in India. The fast-growing population, exceptional rate of motorization coupled with the ever-growing urbanization have made people vulnerable to frequent road accidents resulting in fatalities, injuries/disabilities. The number of fatalities related to road accidents in India has been on an upward trend for the last two decades. Black spots account for a major portion of the total road accident fatalities for the past 3 years. Hence, it is important to undertake appropriate countermeasures to improve the geometric aspects of such black spot locations in a systematic and cost-effective manner. In this paper, the city of Nagpur has been identified as the test bed to understand the efficacy of the economic benefit assessment of black spot improvement by estimating the impact of the proposed countermeasures for four identified black spots in the Nagpur rad network. In this regard, road crash data obtained from Nagpur Police in the form of First Information Reports (FIRs) were analysed to identify the black spots conforming to the protocol of the Ministry of Road Transport and Highways. The geometric countermeasures deduced for the four most severe black spots are discussed followed by the economic benefit assessment with the primary objective to demonstrate the estimated reduction in fatalities as well as the total number of crashes which can happen due to the proposed intervention at the identified black spots. The methods adopted in this study are benefit-cost ratio method, net present value method and economic internal rate of return method as prescribed by the Indian Roads Congress. As a result of the proposed countermeasures, it is estimated that there is a significant decrease in both fatal and serious injury crashes. It is hoped that the above study will serve as an eye-opener for the relevant stakeholders in terms of undertaking steps towards the implementation of the

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suggested black spot remedial measures. Apart from the above engineering measures, proper enforcement and road safety campaigns that educate the road users about the potential dangers need to be implemented.

Keywords Black spot · Countermeasure · Benefit-cost analysis · Countermeasures · Economic internal rate of return

1 Introduction

According to figures from the World Health Organization, 1.3 million individuals worldwide lose their lives due to road crashes every year, raising concerns about global road safety. Around 93% of the lower and middle-income nations account for the majority of global road crashes, despite the fact that these nations account only for 60% of the world's automobiles. In the year 2020, road crashes in India claimed the lives of 1,31,714 as well as causing 3,48,279 injuries and thus accounting for 12.8 and 22.4% less as compared to the year 2019 wherein the reported road deaths were 1,51,113 and road crashes were 4,49,002. This can be attributed to the fact that the imposition of lockdown across the country due to COVID-19 pandemic having a negative impact on mobility resulting in reduced number of trips, which in turn could have contributed for the reduced number of fatalities and road crashes. Hence, the above-reported figures in the year 2020 should be considered as an outlier for the time being unless otherwise the above-reported trend continues in 2021. Further, it was observed in 2020 that 69% of the fatal road crash victims belong to the working age population on Indian roads. Apart from the immediate impact a road crash can inflict to the concerned individual and their families, the indirect ripple effect that results are that it always has a negative impact on the India's GDP in the long term as the impact on GDP is estimated to be ranging between 3 and 5% due to the road crashes in the country. Nagpur is one of the fastest-growing metropolitan cities in India. The city has the uniqueness of being the 'geographical centre of India' by its very location on the Indian map and due to the above, two major national highways (NHs), namely, NH-7 and NH-6 pass through the city which act as major connectors for the traffic plying between north–south and east–west parts of the country. Looking at the historical crash data in the city of Nagpur, it was observed that the total crashes from 2008 to 2016 were around 1500 per year. Since then, there has been a decrease in the total number of accidents of around 20% in the years 2018 and 2019. But this decrease in total accidents did not affect the fatalities, which hovered around 200–250 each year with the crash severity index, i.e., *persons killed per 100 road crashes* remained 24.8 and 27.1 in 2019 and 2020, respectively. Thus, the number of road crash and fatality rates are on the higher side for a city like Nagpur which necessitates the need for devising appropriate engineering interventions and estimating its economic benefits. In the year 2020, out of the 774 crashes in rural Nagpur there were 382 fatal crashes whereas in urban Nagpur the fatalities reduced to 210 [1]. This can be primarily attributed to large-scale speed violation and lack of golden hour facilities in the vicinity. Pedestrians, bicyclists and two-wheelers account for 85% and 59% of all road crashes in urban and rural areas of the city respectively; which implies the fact that there is a lack of adequate infrastructure to ensure the safe commute for the above category of vulnerable road users [2] in the city. Improvements in road geometry like provision of a median, number of lanes and lane width mainly influence the behaviour of drivers [3]. Hence, the black spots must be treated to reduce the number of fatal crashes and grievous injuries by devising the countermeasures which would be forgiving in nature to address the human errors and their unpredictable behaviour into account [3]. To accomplish the same, the succeeding section provides an overview of some of the success stories in terms of reduction in road fatalities as well as number of road crashes which were achieved elsewhere through the implementation of set of appropriate countermeasures have been reviewed. The reviewed literature would be considered as the base to devise the countermeasures for the selected black spots in Nagpur followed by the assessment of economic benefit assessment in the event of its implementation.

2 Literature Review

Common types of crashes can be successfully reduced by specific engineering treatments known as countermeasures. In certain black spots, where there is not a prevalent crash type, multiple interventions related to different types of scenarios have to be proposed [3]. The effectiveness of countermeasures needed to carry out the benefit–cost analysis was taken based on the values obtained from international organizations such as iRAP, Road Safety Measures Handbook, GRSF and various research papers as there is no indigenous safety manual prepared for India from which one could utilize the effectiveness values. Evidence from crash frequency models in Latin American cities suggests that the well-designed median openings can reduce crashes, including severe crashes, by 30–40% [4]. Further, the provision of well-designed median has been demonstrated to decrease the percentage of pedestrian casualties and grievous injuries by 70% on average in the US [5]. Also, a strong dependence on impact speed and pedestrian crashes is found with the fatality risk at 50 km/h being more than twice as high as the risk at 40 km/h and more than five times higher than the risk at 30 km/h [6]. Studies done in Norway demonstrated that the provision of speed calming measures in the form speed humps passing through urban areas reduce the number of fatal and severe injury crashes, for a given amount of traffic by around 50% [7]. In Seoul, Korea, crashes decreased by 39% in school zones after the implementation of design suggested geometric design improvements and traffic calming measures [8]. Chevron markings in horizontal curves have been found to reduce road crashes by 30% [7]. Traffic signal control reduces the number of road crashes by around 15% at T-junctions and around 30% at crossroads [7]. The combined effect of Edge line, Centreline markings, delineator posts and chevron signs at curved portions has resulted in 45% decrease in severe injury-related road crashes [7].

3 Methodology

Taking cue out of the above-reviewed literature, the study methodology has been devised, which is presented in Fig. 1 wherein appropriate countermeasures are intended to be formulated followed by the assessment of the economic benefits due to the same.

First Information Reports (FIRs) obtained from the Nagpur Police served as the baseline in identifying the black spots. The FIRs encompassing the road crash and fatality data from 1.1.2019 to 30.11.21 provided by the Nagpur Traffic Police (NTP) were analysed in the study. The data consisted of the date and time of the crash, the latitude and longitude of the crash location, number and type of vehicles involved, type of crash, age of victim(s), number of fatalities, whether the victims have sustained grievous or minor injuries. To assess the existing traffic flow parameters at the identified black spots, the following traffic studies were carried out at all the selected locations: classified traffic volume counts, spot speed studies, pedestrian volume counts and speed and delay studies were obtained [9]. 12 h Classified Traffic Volume Count (CTVC) studies were carried to capture the number of vehicles using the road network and to estimate the intensity of traffic on the intersecting roads

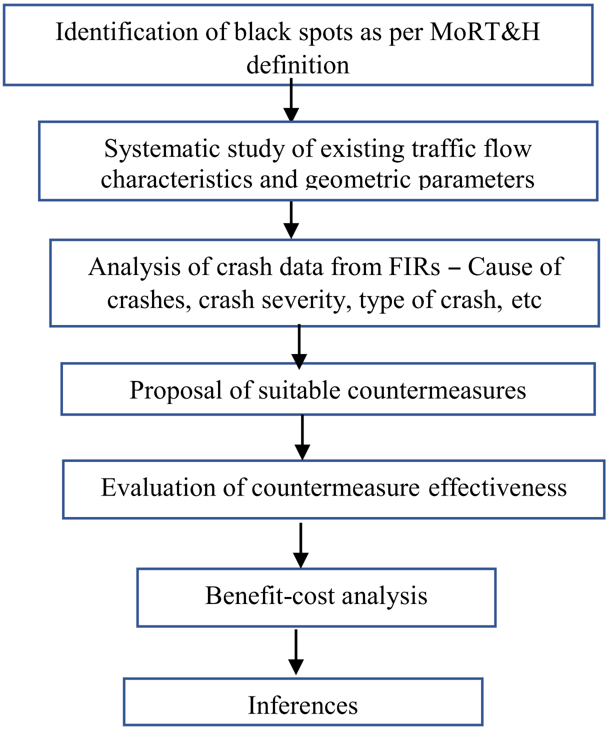

Fig. 1 Study methodology

as well as on the midblock sections of the same. This survey was collected using videography on a typical working day from 8:00 am to 8:00 pm to cover the morning and evening peak flow along with off-peak flow. To understand the variation of speed profile, spot speed studies conducted using Laser Speed Guns at vulnerable locations of the road network covering both directions of travel. Direction-wise sample data of spot speeds of different vehicles were analysed to get various speed characteristics, namely, minimum speed, maximum speed, average speed and different percentile speeds such as 15th, 50th, 85th and 95th in order to evolve appropriate speed control and safety measures [9]. Topographic surveys by using total station system were conducted at each of the black spots to assess the existing geometric features, which would help create the base plans. With the obtained base plan, detailed geometric design plan was formulated by CSIR-CRRI in the form of detailed project report (DPR). Summary of the CVC data collected at the above-identified black spots is presented in Table 1.

To evaluate the effectiveness of the proposed countermeasures, the construction and maintenance costs were calculated for a period of 5 years. The unit rates for each countermeasure were taken from Development of model road stretches by NHAI [10] and appropriate inflation rates obtained from World Bank data were applied for a period of 5 years [11]. The cumulative effectiveness values for each type of crash were calculated as outlined in Sect. 6 and the reduction in crashes was computed for each black spot is as follows

$$\textbf{Crash Reduction} = (\text{cumulative effectiveness} * \text{number of crashes})$$

$$\textbf{Total Discounted Cost} = \text{Total discounted construction cost} + \text{Total discounted benefits}$$

$$\textbf{Benefit/Savings} = (\text{Reduced number of crashes/Total number of crashes}) \times \text{Average annual crash cost}$$

Table 1 Observed traffic flows at the four black spots over 12 h period on a typical working day

Name of the black spot	Classified traffic volume count (CVC) for 12 h period				
	Two-wheelers	Small and big cars	Auto rickshaw	Others	Total
Prakash high school	26,337	8,564	1,907	8,940	45,748
Chikli square	37,435	5,832	3,606	14,457	61,330
Telephone exchange to C.A.	34,884	4,013	1,534	10,126	50,557
Jhansi Rani Square-1	46,420	10,728	12,092	5,631	74,871

The cost of construction, maintenance and savings for the future year by considering the analysis period of 5 years and its discounted value have been determined by using the following equation

$$\textbf{Future Value} = \text{Previous year value} + (\text{Previous year value} * \text{GDP growth \% in the year considered}).$$

4 Crash Data Analysis of the FIR Records of Nagpur

The most common types of reasons found in the FIR reports of Nagpur were found to be Hit in side, hit in rear, hit pedestrian/cyclist and hit and run types of crashes. 54% of the crashes happened during the day and 46% took place during the night, so there is more or less an even distribution of road crashes throughout the day despite the fact that only about 15% of the road traffic ply during the night time. Further, 70% of the total 119 road crashes involve at least one motorcycle, which can be directly corroborated with their proportion on the road network (ranging between 50 and 75% of the total flow) coupled with their erratic driving behaviour and absence of dedicated lanes for their movement due to insufficient Right of Way (RoW). The number of road crashes involving other type of Vulnerable Road Users (VRUs), namely, pedestrians (13%) and bicycle crashes (8%) underline the fact that no measures are in place at the identified black spots to address their safety concerns [2]. Since the overall share of fatal, grievous injury and minor injury/property damage crashes are 17%, 63% and 20%, respectively, it is felt that the implementation of the proposed countermeasures will have a huge bearing in terms of reduction of fatalities as well as serious type of injuries on the road network of Nagpur. It may be noted for about 32% of the reported road crashes, no proper reasoning is recorded in the FIRs and hence only 68% of the road crashes in FIRs could be considered while designing the countermeasures. This is because the effectiveness can be deduced based on the reported causes for the road crashes, which is missing for 32% of the FIR records. Table 2 presents the number of crashes and fatalities reported between 2018 and 2021 at the above black spots.

5 Black Spots in Nagpur

A total of 38 locations/road stretch/intersections were identified as black spots conforming to the Ministry of Road Transport and Highways (MoRT & H), which states that location wherein either 5 road crashes or 10 fatalities occurred within distance of 500 m during the last 3 calendar years should be categorized as a black spot in the city of Nagpur. Using the above protocol, engineering interventions carried

Table 2 Profile of road crashes reported between 2019 and 2021

Name of the black spot	FIRs at each black spot	Number of deaths	Number of grievously injured victims	Number of minor injury victims
Prakash high school	20	4	13	5
Chikli square	30	6	16	9
Telephone exchange to C.A.	37	4	30	6
Jhansi Rani square-1	32	6	19	12
Total	119	20	78	32

out for four black spots have been considered in this paper to showcase the effectiveness of this approach, which encompasses two intersections and two mid-block locations. The two intersections selected include Chikli Square and Jhansi Rani Square, which are 4-arm and 5-arm intersections, respectively, whereas the midblock locations chosen are Prakash High School and Telephone Exchange Road. Common road safety issues observed at the black spots include:

- Open areas lacking proper geometric design.
- Absence of road markings, delineation, and road signs, etc.
- Absence of channelizing islands in the intersections leads to higher chances of conflicts between turning and through moving traffic.
- Improper pedestrian facilities such as lack of designated areas for crossing, absence of refuge areas, depressed median for at grade crossing.
- Obstructions in footpath by parked vehicles and vendors, absence of contiguous footpaths.

Obviously, many of the countermeasures suggested were common/generic in nature amongst the four black spots, which are as follows:

- Provision/improvement of physical channelizing islands at intersections/diverging/merging locations, median refuge areas with concrete bollards to ensure safe crossing of pedestrians. Placement of speed table at the designated pedestrian crossing locations including the free left turning approaches.
- Provision of speed breaker/Table/rumble strips/transverse bar markings (TBM) at the minor intersecting approaches and other applicable crash-prone locations.
- Provision of lane markings and edge marking using zigzag marking, median markers, solar road studs *(for about* 100 m*)* followed by conventional retro-reflective road studs at 9 m centre to centre on the outer edge of the pavement/median/traffic lanes conforming to IRC:35 [12].
- Provision of Single and two-way Object Hazard Markers (OHM) ahead at all the diverging and merging sections conforming to IRC:35 [12].

- Provision of various signages such as prohibitory/Regulatory Signs, Cautionary/Warning signs *(especially Accident-Prone Sign)*, Informatory and Directional information/Facility Information signs wherever applicable, conforming to IRC:67 [13].

The site-specific solutions developed for the four black spot locations spanning a length of 500 m on either side of the road in the case of the midblock locations and 250 m from the centre of the intersection in the case of the intersections are briefly discussed in the subsequent sections.

5.1 Prakash High School

This black spot is encompassed by Asian Highway 46 in the east–west direction. The road geometry present near Prakash High School is a mid-block section between two intersections. The total traffic volume handled at this midblock during the 12 h survey period was observed to be 49283 PCUs (45748 Vehicles) as shown in Table 1. The peak hour traffic flow was measured to be 4418 PCUs/h in the morning and 4478 PCUs/h (passenger car unit) in the evening [2]. This peak hour flow and the available right of way were utilized to propose the necessary countermeasures. This stretch of the road consists of two minor roads merging with the national highway. The specific issues observed at this black spot were improper merging of minor road traffic coupled with traffic calming measures on minor intersecting roads, absence of chevron signs and markings ahead of a sharp curve and absence of treatment at locations wherein minor construction works are in progress. Further, there are no forms of traffic calming measures deployed near the school zone to slow down the traffic speeds. Considering the above, the major site-specific countermeasures devised for 1 km stretch along the Prakash High School are presented:

- Improvement of geometrics/traffic calming measures on all the minor intersecting roads with the project corridor.
- Provision of speed table and speed limit signs of 30 Kmph ahead of pedestrian crossing area before the school zone.
- Provision of Chevron signs on both sides of the carriageway to improve visibility in the curved section during night time.

The construction cost to carry out the proposed countermeasures was computed to be 28.25 lakh rupees. If the interventions are carried out, the number of anticipated traffic accidents is expected to be 8, which would represent a 60% decrease from the 20 recorded accidents (see Table 2) that occurred along the road corridor near Prakash High School during the research period. Figures 2 and 3 provide a glimpse of the typical road conditions 'before' and 'after' geometric design plan conceived *(typically depicted here only for about* 100 m *length only)* near the Prakash High School.

Fig. 2 Road section 'before' implementation of the countermeasures near Prakash High School

Fig. 3 Typical illustration of geometric design for the case of 'after' countermeasure implementation scenario

5.2 Chikli Square

This black spot is formed at the intersection of National Highway NH 44 and the major state highway of Maharashtra SH-09. The total traffic volume handled at this intersection during the 12 h survey period was observed to be 50291 PCUs (61330 Vehicles) as shown in Table 1. It can be noted that the two-wheelers (61.04%) dominate the traffic composition followed by three-wheeler and four-wheeler goods vehicles (11.10%), cars including (9.51%), and slow-moving vehicles including bicycle and cycle rickshaw (7.86%). The proportion of Bus and Mini-Bus observed to be only around 0.27%. The main issues at this black spot were found to be lack of road marking and signs, absence of channelizing islands as well as large open area of the intersection encouraging the road users, mainly motorcycles (61% of the total vehicle composition) to drive in the wrong direction at high speeds [2]. To mitigate these issues, two alternative geometric designs were proposed: first one is a standard signalization of the four-arm intersection; second alternative is to provide a guided U-turn towards the major approach. This option reduces the number of conflict points from 24 to 8. Since this black spot is situated in an open area, there is ready availability of ROW which facilitates widening of the carriageway. There is a possibility of further reduction in the road crashes and fatalities as large-scale disobedience of traffic signals (rampant violation of Signal was observed during the surveys) is addressed in the second alternative through the provision of back-to-back U-turn. Considering the above issues, the major site-specific countermeasures devised for the two options conceived for the Chikli Square [2] are presented:

Fig. 4 Typical illustration of large size intersection with open area without road marking and signs and absence of speed tables

- Provision of channelizing islands and depressed medians with cement concrete bollards to facilitate safe pedestrian crossing.
- Pedestrian signal to be installed at a distance of 45 m from the zebra crossing; the signal will be open for pedestrians for 25 s during the 60 s signal time in the case of Option 2 implementation.
- Provision of speed table at free left turns on every approach.
- Provision of back-to-back U-turns by flaring the road within the available RoW.

The construction cost in order to carry out the proposed countermeasures was computed to be 39.83 lakh rupees and 81.23 lakh rupees for Options 1 and 2, respectively. The increase in total cost in option 2 is attributed to the development of the road structure, lane widening within the available RoW and footpath construction costs to be incurred for the case of back-to-back U-Turn. The estimated road crashes if the countermeasures are implemented is estimated to be 13 and 11 for options 1 and 2 and thereby accounting for more than 60% and 72% reduction, respectively, from the 30 reported crashes (given in Table 2) at the Chikli Square during the study period. Some of the existing safety-related issues observed at the Chikli Square and its vicinity are presented through a few snapshots in Figs. 4 and 5 whereas Fig. 6 provides a glimpse of the typical road conditions for the scenario of 'after' geometric design plan implementation, which is typically depicted here only for the Option 2 covering only the intersection portion of the Chikli Square.

5.3 *Telephone Exchange to C.A. Road*

This black spot is encompassed by Asian Highway 46 running in the east to west direction of the city. The road geometry present at this location is a typical mid-block section serving two major intersections covering in excess of 1 km. The average speeds of the traffic flow were found to be 39.5 km/h and 34.75 km/h in the morning and evening peak hours respectively. 12 h traffic count in both directions from Juni Pardi Naka to Mayo square was found to be 40,835 PCUs (50,557 vehicles) as shown

Fig. 5 Poor upkeep of the Channelizers and absence of depressed median to facilitate pedestrian crossing

Fig. 6 Typical illustration of Chikli square option 2 having junction closure on the major approach

in Table 1. Out of which two-wheelers dominated as the most preferred mode of transport accounting for 69% of the total traffic volume. Passenger cars and auto rickshaws comprised of 15 and 6% of the total traffic volume. Considering the above issues, the major site-specific countermeasures devised for the midblock stretch between Telephone Exchange to C.A. Road [2] is presented:

- Provision of bus bay on both sides o after the intersection approach in Chappru Chowk.
- Provision of channelizing island with depressed medians to allow safe pedestrian crossing on all the major/minor intersecting roads at Chappru Chowk and Dr. Rajendra Prasad Chowk.

- Provision of traffic calming measures ahead of the Telephone exchange metro station to reduce the travelling speeds where pedestrian volume is on the higher side.

The construction cost in order to carry out the proposed countermeasures was computed to be Rs. 49.55 lakh. The estimated road crashes if the interventions are implemented is estimated to be 15 and thereby accounting for 59% reduction from the 37 reported crashes (given in Table 2) on the road stretch between Telephone Exchange to CA Road during the study period.

5.4 *Jhansi Rani Square*

Jhansi Rani Square is a 5-arm intersection wherein AH-46, SH 255 and SH264 intersects along with the road leading to Nagpur Railway station also meet at this location. This black spot also encompasses of two 4 arm intersections in the south direction, namely, Dattawadi intersection and Jhansi Rani Square 2. The average journey speeds of the traffic flow were found to be 31.9 km/h on the Chhatarpati to Jhansi Rani Square direction whereas 30.3 Kmph on the opposite direction during the morning and evening peak hours, respectively. 12 h traffic count in both directions was found to be 55,774 PCUs (74,871 vehicles) as already shown in Table 1. Out of which two-wheelers dominate the flow accounting for 62% of the total traffic volume. Passenger cars and auto rickshaws comprised 14% and 17% of the total traffic volume, respectively [2]. Considering the above issues, the major countermeasures devised for this location will be similar to the Chikli Square—Option 1, which is discussed in Sect. 5.2.

The construction cost in order to carry out the proposed countermeasures was computed to be 81.21 lakh rupees. The estimated road crashes if the countermeasures are implemented is estimated to be 11 and thereby accounting for more than 66% from the 36 reported crashes (given in Table 2) at the Rani Jhansi Square 1 during the study period.

Further, it is noted that that at all four black spot locations, the proportion of public transport traffic is less than 1%, which indicates there is a reluctance from road users end to use public transport. This indicates that the level of service provided by the public transport needs improvement, which would result in lesser traffic volume thereby improving road safety and traffic congestion.

6 Effectiveness of Countermeasures

The cumulative effectiveness was calculated since there are multiple interventions used in each scenario.

$$\text{Cumulativeeffectiveness} = [1 - \{(1 - E1) * (1 - E2) * \ldots (1 - En)\}];$$

where $E1$, $E2$ and En are the effectiveness of 1st, 2nd and nth countermeasure, respectively [14].

Since there are no indigenous manuals/research studies that provide standard effectiveness of interventions in India, the effectiveness scale data were taken from various sources such as World Bank, The Handbook of Road Safety Measures, WRI and ITE, iMAAP software, etc. Addressing the various forms of appropriate road geometric improvement measures like the provision of Channelizing Island, zebra crossing and median refuge islands were found to be 70% effective in terms of preventing pedestrian and bicycle fatalities/serious injury type of road crashes [7]. Similarly, the effectiveness of the various forms of traffic calming measures like the speed breakers/rumble strips/TBM are expected to reduce 50% of the fatalities/ serious injury type road crashes at intersections [5]. Considering the above-stated effectiveness scale, one can arrive at an estimate of reduction in fatalities/serious injury type road crashes for the above types of collisions, which is presented in Table 2. The costs of the countermeasures provided were taken from development of model road stretch by NHAI reports, which was proposed in 2020 [10] (Table 3).

Table 3 Effectiveness of countermeasures

Serial Number	Countermeasure and effectiveness	Cost of countermeasure provided (INR)	Source
1	Speed humps, TBMs and rumble strips—35% reduction in injury crashes	TBM, rumble strips—580 per sq. m	Elvik et al. [7]
2	Edge line, centreline markings, delineator posts and chevron signs—45% reduction in crashes	Edge line and CL—580 per m^2 Chevron signs, OHMs—5155 per unit	Elvik et al. [7]
3	Provision of medians—30–40% reduction in severe crashes	Earthwork—666 per cum Construction of footpath—769 per m^2	Duduta et al. [4]
4	Channelizing island, median refuge islands—70% decrease in fatal crashes		FHWA Safety [5]
5	Design improvements in school zones—39%	N/A	Sul [8]
6	Provision of median U-turns—30% reduction in intersection injury-related crashes	N/A	FHWA Safety [5]
7	Speed limit signs, warning signs—10% reduction in overall crashes	Speed limit signs, warning signs—5155 per unit	World Resources Institute [15]

7 Usage of Data on Road Crashes and Deaths

Road crashes pose a social and economic cost due to sudden deaths, injuries and loss of potential income. Depending on the severity of the injuries, road crashes have been further classified as fatal, grievous and minor injury road crashes. MoRT&H uses Human Capital Approach while calculating the total road crash costs. The basis of Human Capital Approach is the concept that the cumulative output through the life of an individual can be quantified and the cost of future lost output can be calculated based on duration of productive life [16]. In this method, the cost of a road crash consists of victim-related costs, property damage costs and administrative costs. The study has estimated that out of the total costs, 97.7% are victim-related, and the rest 2.3% are for the other two components. The overall total crash cost for the Indian roads for the year of 2018 was estimated to be INR 5,96,829 crores, which is equivalent to 3.14% of the nation's GDP. The baseline used for calculating crash cost based on the type severity of injury/fatality is as follows [16].

It is clear that the cost of a fatality creates a huge loss in terms of a valuable life and also to the economy of a country. Hence, there is a huge responsibility on the road authorities to reduce the fatalities as much as possible. A study undertaken by the World Bank revealed that by reducing the road mortality by 50% and sustaining it for a period of 24 years could generate additional flow of income, which is equivalent to 14% of India's GDP [17].

8 Benefit–Cost Analysis

To device the appropriate cost-effective countermeasures, adequate analysis has to be conducted prior to implementation by understanding the ground conditions and referring to the associated literature [3]. After deducing the same, Benefit–Cost Analysis can be conducted. This is a method that determines the future risk reduction benefits of a hazard mitigation project and compares those benefits to its costs. In India, economic evaluations of road safety measures are rarely published in the scholarly literature. There is considerable uncertainty of forecasts beyond a certain reasonable period. Human behaviour may change, travel pattern may undergo a shift and technology may experience transformation. Thus, it is worthwhile to limit the analysis period to 5 years since the amount invested amount is relatively on the lower side in the case of black spot treatment. The discount rate is taken as 12%, which is used by the planning commission [14] and the reference year is taken as 2022. The maintenance cost of all countermeasures is taken as 5% of the construction cost. A project is considered as cost-effective if the benefit-cost (B–C) ratio is >1 [14]. Other than B–C ratio, other methods like net present value (NPV) method and economic internal rate of return (EIRR) are also deployed, and all the three methods are based on the discounted cash flow (DCF) technique of discounting all future costs and benefits to a common year. The three methods used for economic evaluation are discussed below in brief [14].

8.1 Benefit–Cost Ratio Method

There are a number of variations of this method, but a simple procedure is to discount all costs and benefits to their present worth and calculate the ratio of the benefits to costs. Negative flows are considered as costs whereas positive flows as benefits [14].

$$\text{Benefit} - \text{cost ratio} = \frac{\text{Total benefits over the analysis years discounted to the reference year}}{\text{Total cost over the analysis years discounted to the reference year}}$$

8.2 Net Present Value Method

In this method, the stream of costs/benefits associated with the project over an extended period of time is calculated and is discounted at a selected discounted rate to give the present value. Benefits are treated as positive and costs as negative and the summation give the Net Present Value (NPV) [14].

$$\text{NPV0} = (B0 - C0) + \frac{B1 - C1}{(i+1)^1} + \cdots + \frac{Bn - Cn}{(i+n)^n}$$

8.3 Economic Internal Rate of Return

Economic internal rate of return (EIRR) is the discount rate that makes the discounted future benefits equal to the initial outlay. In other words, it is the discount rate that makes the stream of cash flows to zero. The solution to the equation given below can be done by trial and error. However, the task of computing EIRR is rendered very simple nowadays due to the availability of this function as an inbuilt one in many software. If the EIRR calculated from the above formula is greater than the rate of interest obtainable by investing the capital in the open market, the scheme is considered acceptable [14]. A summary of the deduced results for the four black spots is presented in Table 4.

Table 4 Average unit costs of crashes for each crash severity

Type of severity	Average unit crash cost per victim (INR)
Fatality cost	91,16,363
Grievous injury cost	3,64,398
Minor injury and property damage cost	83,201

9 Conclusions

This study focused on analysing four severe black spots in the city of Nagpur and provided an understanding of the economic benefit assessment of the proposed countermeasures. Some of the salient findings derived are as follows:

- The proposed countermeasures were found to be cost-effective for the four black spots conforming to IRC:131 [3]. It is estimated that about 60–66% reduction in the overall road crashes coupled with 40% reduction in fatalities if the countermeasures are applied assuming a similar rate of road crashes in the next 5-year period on the road network of Nagpur as black spot improvements falls under cost-effective improvements.
- The total cost savings for each black spot are summarized in Table 5. On average, there is a cost-saving of about Rs. 89.15 lakhs in the midblock locations and Rs. 1.26 crores in the intersections after the implementation of 1 year of proposed countermeasures.
- The Economic Internal Rate of Return (EIRR) was found to be ranging between 54% and 63% through the analysis period of 5 years, which can be considered to be a significant return on investment (ROI). Even the First Year Rate of Return

Table 5 Summary of benefit–cost evaluation

Name of the black spot	Benefit–cost ratio	First-year cost savings (INR)	First-year rate of return (%)	Net present value	Economic internal rate of return
Chikli square—option 1	9.82	1,18,02,695	2.71	4,21,19,210	63.53
Chikli square—option 2	5.16	1,24,36,765	1.42	4,04,63,423	54.72
Prakash High school	10.01	85,32,006	2.76	3,05,13,013	63.71
Telephone exchange to C.A.	6.22	92,96,482	1.72	3,09,96,116	57.89
Jhansi Rani square 1	5.46	1,33,88,615	1.51	4,34,54,691	55.76

(FYRR) was estimated to be ranging between 1.4 and 2.76%, which shows that there is bound to be an immediate ROI in all four black spot locations.
- At the same time, though the implementation of black spot interventions is an effective tool in treating the affected road stretches/locations, it is to be borne in mind that it is one of the forms of interventions to address road safety in the road network of any city.

10 Future Scope

The limitations and the future scope of this study are as follows:

- More robust form of data collection is necessary to account for unreported and minor injury crashes.
- The effectiveness of the countermeasures needs to be studied extensively in the Indian road conditions.
- Countermeasures need to be studied for its medium to long-term effectiveness by using reliable projections of the traffic and crash data through studying the trends in road crashes.

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Emerging Technology, Logistics and Sustainability

Impact of Autonomous Vehicles on Capacity of a Two-Lane Highway

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Abstract An autonomous vehicle (AV) or driverless car ensures safety using advanced sensor and positioning technologies with little or no human input. With numerous emerging technologies, it is important to know the potential capacity effects of AVs to aid our decision-making process with future investments. As AVs are not widely used especially in developing countries like India, it is difficult to study the behaviour of AVs and their interactions with other vehicles in the field. Hence, the mid-block section of a two-lane highway incorporating AVs is simulated using VISSM in this study to analyse how AVs impact the capacity of the highway. Passenger cars are replaced by autonomous cars in the model and the model is calibrated using travel time as measure, and the change in the capacity for various penetration rates of AV is estimated. The study gave promising outputs in terms of capacity enhancement and found that the introduction of the AVs can enable better utilization of the available road space.

Keywords Autonomous vehicle · Capacity · Two lane highway · VISSIM

1 Introduction

An autonomous vehicle (AV), also known as a self-driving car or driverless car, is a vehicle that can detect its environment and move safely with little or no human input. AVs use various sensors such as radar, lidar, sonar, GPS, odometry and inertial

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measurement units to perceive their surroundings. In 2014, automotive standardization body SAE International published a classification system of AVs with six levels ranging from fully manual (Level 0) to fully automated systems (Level 5) [1]. At present, Level 3 AVs are available for consumers to purchase.

Today AVs are being introduced in the market and have the potential to mitigate some of the negative impacts of transportation [2]. They can improve highway safety and efficiency by decreasing driver inputs, reducing driver workloads and human error, and providing environmental benefits by reducing emissions and fuel consumption. Many of the benefits of AVs will not be obtained until after full market penetration, which is not expected to happen immediately. It is estimated that autonomous cars will reach 50% global penetration only by the year 2024. With numerous emerging technologies, it is important to understand the potential capacity effects of AVs to aid our decision-making process with future investments.

Adebisi et al. [3] conducted simulations on basic freeway, freeway merge, and freeway weaving segments using two major AV applications: cooperative adaptive cruise control and advanced merging. The study discovered that as the market penetration rate of Connected AVs (CAVs) increases, it can significantly improve roadway capacity by up to 35–40% under certain circumstances. This improvement was observed not only on basic freeways but also on merge and weaving segments. Park et al. [4] conducted a study on urban roads to investigate how gradual increments of AV penetration and traffic volume impacted traffic flow and road capacity. The study discovered that as AV penetration increased, traffic flow improved with a reduction of up to 31% in average delay time. Links with three or four lanes had a more significant impact on delay reduction than those with fewer lanes. When AV penetration reached 100%, the road network could accommodate 40% more traffic. Olia et al. [5] discovered that if all vehicles are driven in a cooperative automated manner, it is possible to achieve a maximum lane capacity of 6,450 vph per lane (300% improvement). The study also found that incorporating AVs into the traffic stream did not significantly affect achievable capacity.

Zhao et al. [6] built VISSIM simulation models for different sections of freeway under different traffic status scenarios and various levels of autonomous vehicle penetration to quantify the traffic impact under multiple penetrations of autonomous vehicles. The analysis showed that in free flow scenarios, penetration of autonomous vehicles had a slight positive impact on traffic operations for all segments. Ye et al. [7] studied how CAVs affect traffic safety under various penetration rates and discovered that as CAV penetration rate increased, smooth driving increased while velocity difference between vehicles decreased which greatly smoothed out traffic flow. Stop-and-go traffic was also greatly eased.

Majority of the studies were done on freeways using simulation software and not on Indian traffic condition. In most literature, they arrived at the conclusion that AVs to have a positive impact on traffic and safety. Traffic in India is very different compared to the lane following traffic in developed countries. Different types of vehicles like auto and truck share the same lane as two-wheelers and cars. This brings the need to study the effect of AVs on our existing traffic condition when they are moving along with other vehicles. With numerous emerging technologies, it is important to know the potential capacity effects of AVs to aid our decision-making process with future investments. In this study, the effect of AVs on the capacity of a two-lane highway is studied. The scope of the study covers developing simulation model of a mid-block section of a four lane divided National Highway and the calibration and validation of the simulation model incorporating AVs using Travel time as the characteristic.

2 Methodology

2.1 Data Collection

A two-lane unidirectional highway of the Bypass near Thiruvallam, Thiruvananthapuram is selected for the study. An aerial view of the site is given in Fig. 1. A site visit is done and geometric details of the section are collected. In order to obtain data like volume, traffic composition and so on video data were collected from a foot overbridge near the section.

Geometric Details. The study location is a two-lane unidirectional highway. Each lane is 3.5 m. The left shoulder is of 1.5 m width, and the right shoulder is 30 cm wide.

Video Data. Video data were collected for 2 h from 3:30 pm to 5:30 pm. Camera and tripod were set on a foot overbridge near the location. It was possible to collect data through a distance of 160 m.

Fig. 1 Aerial view of the NH bypass near Thiruvallam, Thiruvananthapuram

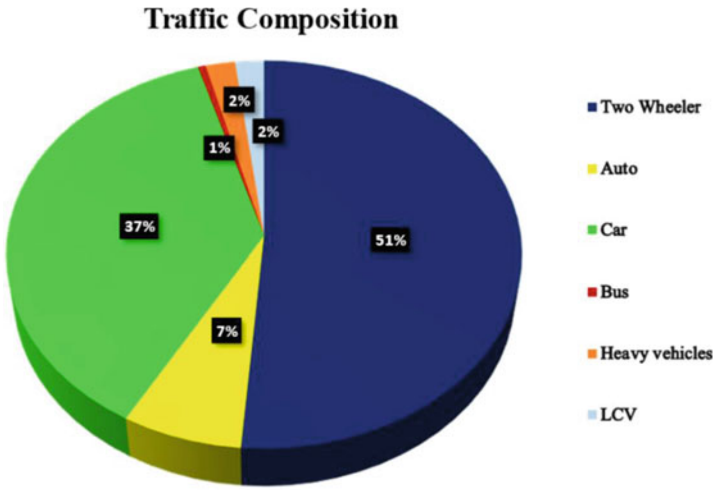

Fig. 2 Traffic composition

Traffic volume was counted manually and was found to be 1060 veh/h. Traffic composition was also found by manual method. The video was played on a computer and count was taken. The traffic composition is given in Fig. 2. Two-wheelers dominated with 51% of the traffic. It is followed by cars with 37% and three-wheelers with 7%. Mainly three types of cars were seen on the field, hatchback, sedan and jeep or five-seater. Also, the buses were divided into mini bus and normal bus. Heavy vehicles were divided into three categories based on their dimensions.

To find average travel time of vehicles, KinoveaTM software was used. Using the KinoveaTM software, grid lines were drawn on the video data from start to finish of the 160 m stretch. Then travel time of each type of vehicle was found using manual method. The average travel time of entire traffic was 9.8 s. The average speed was also found.

2.2 *Simulation Model Building*

Simulation model was built using PTV VISSIM 2021 micro-simulation software and the snapshot of the model is given in Fig. 3. PTV VISSIM is a microscopic multi-modal traffic flow simulation software package developed by PTV Planning Transport Verkehr AG in Karlsruhe, Germany [8]. The geometric details, traffic volume and composition collected from field were input into the Wiedmann 74 model of VISSIM. Autonomous vehicles (Wiedemann 99 model is used) replace passenger cars only, with two types of autonomous cars, namely cautious and aggressive types. AVs were not allowed to overtake on the same lane, and pedestrian movement and interactions are not considered in the model.

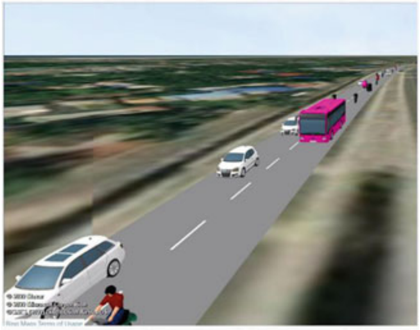

Fig. 3 Snapshot from VISSIM simulation

Calibration of Simulation Model. Simulation model built in VISSIM by inputting geometric details, traffic volume and composition will not be an accurate representation of the field condition. In order to achieve that, various lateral and driving behaviour-related parameters need to be adjusted to match the field condition. PTV VISSIM user manual [9] consists of all the different parameters, their typical range and based on that calibration is done. The values of default and calibrated parameters of the model are given in Table 1.

To check whether the simulation is an accurate representation of field condition, travel time from field data and simulation is compared. The travel time for different vehicle types for a certain distance in the field is obtained from video data. Travel time for the same length of road is also found from simulation and compared with

Table 1 Default and calibrated values of parameters in the model

Parameter	Default value	Calibrated value
Driver behaviour parameters		
Look ahead distance (minimum) (m)	0	10
Look back distance		
Minimum	0	10
Maximum	150	50
Weidmann 74 car following characteristics		
Average standstill distance (m)	2	2.5
Overtake on same lane	No	Yes
Minimum lateral distance at 0 and 50 km/h (m)		
Two-wheelers	1, 1	0.50, 0.75
Three-wheelers	1, 1	0.50, 0.75
Car	1, 1	0.50, 1.00
Heavy vehicle	1, 1	0.75, 1.00
LCV	1, 1	0.75, 1.00
Bus	1, 1	0.75, 1.00

field data and MAPE value is calculated. Before calibration, the MAPE value was 33%, which reduced to 3.18% after calibration.

Simulation of AVs. To determine the impact of AVs, simulation is run with increasing penetration of AVs. That is, at first 10% of the cars in the simulation are converted to autonomous cars, then 20% and so on until the entire cars are converted to autonomous cars. Only cars are given autonomous features in simulation since it is autonomous cars that are mainly sold to consumers.

Autonomous cars are not yet introduced in India, so it is not possible to model them based on observation from field. So, AVs are simulated in VISSIM based on previous studies. CoExist simulation study [10] provides insight into simulation of AVs in mixed traffic using VISSIM. Different studies have made different adjustments to the default values for modelling AVs. These studies have made some common adjustments also, like AVs keeping a shorter distance with the front vehicle, having faster and smoother reactions and observing more around vehicles.

Two types of autonomous cars, cautious and aggressive type were used in the simulation. Cautious AVs are the ones that are initially introduced in the market. They keep more headway and less acceleration values than that are capable by the vehicle so that general public starts trusting AVs. As the AVs become more and more accepted aggressive AVs will be released. The parameters and the values used for aggressive and cautious vehicles are given in Table 2.

Table 2 Simulation parameters used for simulation of AVs [11, 12]

Parameter	Aggressive	Cautious
CC0 (standstill distance) (m)	1	1.5
CC1 (headway time) (s)	0.6	1.5
CC2 (‘following’ variation) (m)	0	0
CC3 (threshold for entering ‘following’)	−6	−10
CC8 (standstill acceleration) (m/s^2)	4	3
CC9 (acceleration at 80 km/h) (m/s^2)	2	1.2
Minimum look ahead distance (m)	0	0
Maximum look ahead distance (m)	300	250
Minimum look back distance (m)	0	0
Maximum look back distance (m)	150	150

3 Results

3.1 General

The simulation was run with increasing penetration of AVs. The simulation was run for a duration of three hours. The first and last 30 min of the simulation were taken as warm up period and not considered for data collection. The total number of different vehicles in the simulation is obtained as output. From this, volume in veh/h is found and then converted to PCU/h.

3.2 Capacity Determination

Simulation runs were used to determine capacity by increasing inflow from near-zero volume to higher volumes during successive simulation runs and calculating throughput (outflow) from simulation output. At lower inflows, outflow volume increases as vehicle input (inflow) increases but when maximum throughput is attained, higher vehicle input will not result in same increment in outflow volume and decrement in outflow despite the increase in vehicle input in consecutive simulations shows that freeway segment reaches its capacity [13]. This method is used to determine the capacity of each simulation run.

3.3 Simulation of Cautious AVs

Simulation was run with increasing penetration of cautious AVs and capacity of each simulation run was determined. The capacity at different penetration of AVs is given in Fig. 4. Not much change in capacity was observed at 0–100% penetration of AVs. Only 1% improvement in capacity was observed when penetration was 40%. Maximum improvement was 2% observed at 70% penetration of AVs. Capacity actually reduced by 0.5% at 100% penetration of AVs. This might be due to cautious AVs keeping more headway and driving with less acceleration compared to HDV. These vehicles also follow lane and won't overtake on the same lane. This might be the reason why improvement in capacity was low even at 100% penetration of cautious AVs.

3.4 Simulation of Aggressive AVs

Simulation was run with increasing penetration of aggressive AVs and capacity of each simulation run was determined. The capacity at different penetrations of AVs is

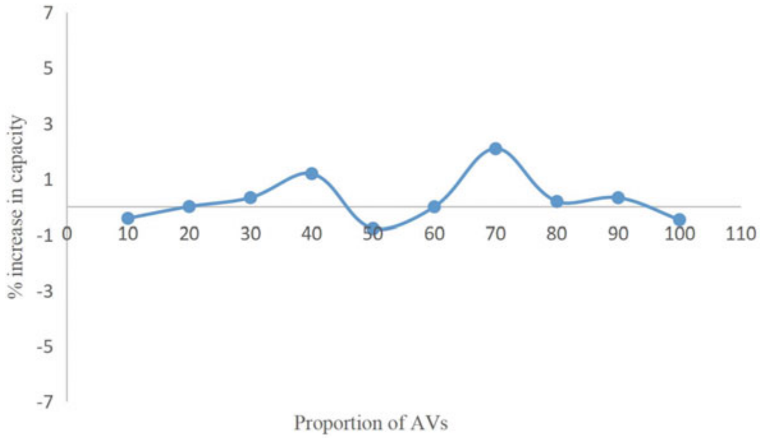

Fig. 4 Percentage increase in capacity at increasing penetration of AVs for cautious AVs

given in Fig. 5. It was observed that the capacity increases with increasing penetration of AVs. An increase of 10% was observed at 50% penetration of AVs and an increase of 19% was observed at 100% penetration of AVs. Even though AVs were not allowed to overtake on the same lane their presence improved capacity by eliminating human error and keeping lower headway than possible by HDV.

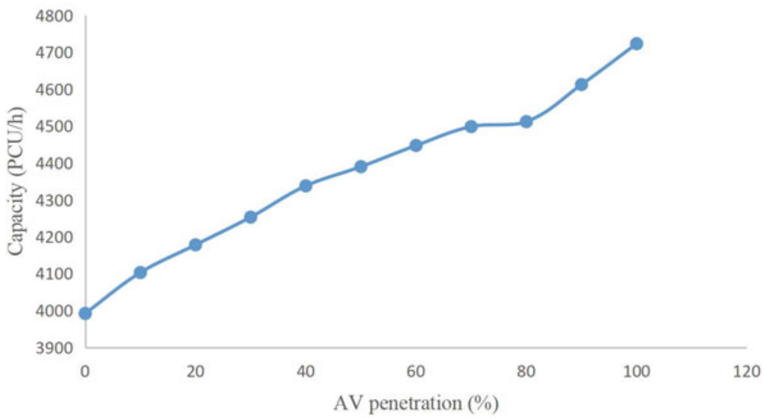

Fig. 5 Capacity at increasing penetration of AVs for aggressive AVs

4 Conclusion

In this study, VISSIM is used to determine the impact of AVs on capacity of a two-lane highway. The impact of both cautious and aggressive AVs was determined separately. Cautious AVs did not have much impact on capacity (maximum improvement was 2% at 70% penetration rate); the capacity values were observed to be fluctuating instead. This might be due to the AVs keeping more headway and having comparatively less acceleration, thereby forcing other vehicles to overtake and proceed. On the other hand, aggressive AVs improved capacity by 19% at 100% penetration of AVs. For them, the capacity increased with increasing penetration of AVs.

Hence, based on the study, it may be concluded that the presence of AVs would bring a positive impact on our roads. Impact of autonomous vehicles on the behaviour (safety) of other vehicles and pedestrians in the stream is not addressed in the study. Wiedemann 99 model for all vehicles may help in better modelling the microscopic levels of interaction between vehicles and AVs in a heterogeneous traffic flow. Calibration of the model is done at a macro level; comparison of parameters (speed, acceleration, lane changing) at micro-level is essential to accurately model heterogeneous traffic flow incorporating AVs. Further studies are necessary incorporating a blend of cautious and aggressive as well as hybrid AVs in the stream, and analysing the impact of these changes on the capacity, level of service and safety on the highways.

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Quality Assessment of App-Based Bike Taxi Services by Benchmarking and Numerical Rating Approach: Guwahati

Lalit Swami, Mokaddes Ali Ahmed, and Suprava Jena

Abstract Shared mobility is an umbrella term that includes various forms of bike-sharing, carsharing, carpooling, vanpooling, and on-demand ride services. It also includes feeder transport services like shuttles and paratransit. With the advancements in technology, options for shared mobility are evolving. In Indian cities, app-based mobility services have become an emerging business. App-based bike taxi service is also an innovative shared mobility model that has emerged recently. This study evaluates the quality of app-based bike taxi services in Guwahati by benchmarking and numerical rating approach (NRA). Key Performance Indicators (KPIs) were developed and derived from various studies based on existing scenarios. A questionnaire survey was conducted, and then NRA was used to evaluate the quality. Additionally, this study explores the aspects that make these services more reliable and accessible, causing commuters to switch. The overall level of service (LOS) of app-based bike taxi services is found two, indicating room for improvement in some areas. The study finds that app-based bike taxi services perform better in real-time tracking, affordability, service coverage, comfort, and travel speed. However, the extent of supply of service needs to be improved. By NRA, deficiency from an acceptable level is spotted in the luggage carrying capacity of the service. In addition, app-based bike taxi services provide online ride-booking, door-to-door pickup and drop service, live location tracking, and multiple fare payment options, making it more attractive than other modes.

Keywords Shared mobility · Bike taxis · Bike-sharing · App-based bike taxis

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1 Introduction

Shared mobility is a transportation model that enables users to gain short-term access to transportation modes on an "as-needed" basis. It is an umbrella term that includes various forms of carsharing, bike-sharing, vanpooling, carpooling, and on-demand ride services. Shared mobility can also include transit services like shuttles, para-transit, and private transit services. On-demand ride services based on apps have recently shifted the paradigm of urban transport [1]. These services are provided by transportation network companies like Ola and Uber and are also termed rides-ourcing services. Using a smartphone to request trips from possible suppliers in real time, Ridesourcing continuously balances demand and supply. Ridesourcing services are distinguished from traditional taxi services by the use of smartphones and ride-matching techniques [1]. As the options for urban mobility are evolving, App-based bike taxi service is also an innovative model that has emerged recently. App-based bike taxi service is a demand-driven system where the booking is made via an Android app, and one bike rider carries one pillion rider. Many ridesourcing platforms like Ola, Uber, and Rapido have started providing rides by motorcycles, scooters, or e-bikes. App-based bike taxi services have emerged as a disruptive mode in Indian cities for transporting and delivering goods. However, they have also generated concerns among many regulators.

As city authorities explore policies on these new innovative modes like app-based bike taxi services, independent data on use, service quality, and impacts are highly required, this study aims to fill this research gap and provide a quality evaluation of app-based bike taxi services by benchmarking. The NRA is also used for considering passenger satisfaction and their perception so that this process can be used in any Indian city.

1.1 Background

App-based bike taxi services permit users to request a ride through an Android application; this notifies nearby drivers of the passenger's location. After the ride is accepted, the users can see the driver's current location and expected arrival time. The smartphone application offers location tracking, making it easier for inexperienced drivers to find their destinations and lowering the likelihood that they would take a wrong turn [2]. Prices respond dynamically to demand, and passengers can pay the fare by UPI, credit card, or cash. At the end of the trip, drivers and passengers rate each other, creating a system of rewards that encourages polite behaviour. Unlike traditional taxis or rickshaws, Ola, Uber, and Rapido drivers lack a commercial vehicle permit, drive their personalized scooter, motorcycle or e-bike, and work part-time. The nomenclature used to describe these services has been highly debated. Other names for app-based taxi services include: "real-time ridesharing," "ride-hailing," "bike taxis," "on-demand rides," "bike-sharing," and "ridesorcing" [1]. The name

"App-based bike taxi services" is used in this study because it precisely conveys the fundamental technology- an application where a motorcycle or scooter can be hired for a ride. However, definitions are confusing, mainly because these services continue to develop.

1.2 Related Literature

Research on new shared mobility models (ridesharing/ridesourcing) is used to shed light on the expected usage, features, and performance of app-based bike taxi services because independent research on App-based bike taxi services is quite limited. City authorities have supported ridesharing for decades because it reduces the vehicle miles travelled. Individually, ridesharing users benefit from reduced commute stress, travel costs, and journey time from high occupancy vehicle lanes [3]. Despite the many advantages, ridesharing has encountered several obstacles, such as a willingness to give up the convenience, comfort, and flexibility of the personal automobile, a need for privacy and personal time, and concerns about personal safety when riding with unknown people [4]. Taxi services cater to a number of markets, including senior citizens, wealthy individuals, and low-income households without cars. Despite having a low modal share, taxis serve an essential need by offering transportation when cars or other public transportation forms are unavailable [5]. Shared taxis may have advantages like higher efficiency, lower passenger costs, and reduced traffic and overall vehicle usage [6]. However, sharing a cab with strangers is often not allowed in some countries like America. In the street hail, customers cannot compare information on fare or quality before selecting a ride, a lack of information is really a problem. A lack of information is an issue since customers cannot compare information on fare or quality before selecting a vehicle. This leads to poor service quality. Due to the low entry barrier in the taxi industry, these markets have over competition. This leads to reckless and aggressive driving, poorly managed vehicles, and congestion [7]. Regulatory responses to these concerns include control over market entry or supply, tariff regulations, and vehicle and driver safety standards. Furthermore, technological advancements raise concerns about how the necessity for regulation may have evolved. Rating systems may be successful in overcoming the lack of information issue. People are now not required to wait on the street or make a phone call to hail a taxi or auto rickshaw. Passenger can now track their own and the driver's whereabouts in real-time. App-based bike taxi services have traits similar to traditional taxis but also have the ability to use the advantage of technology. App-based bike taxi services possess a challenge for regulators. Addressing these challenges clearly requires more information regarding their quality and use in Indian cities.

2 Study Area

Guwahati is the largest city in N-E region. It is also the largest industrial, commercial, and educational hub in the region. Furthermore, significant number of students and workers from other northeastern states are used to migrate to the city. Traffic jams have become a regular phenomenon in the city, due to its increasing population growth and limited infrastructure and road development. The public transit system comprises organized and unorganized public transport. The organized system contains city bus services operated and managed by Assam State Transport Corporation. The unorganized public transportation system consists of e-rickshaws, App-based taxis, traditional taxis, and other paratransit services. RAPIDO aggregator launched the app-based bike taxi services in Guwahati in 2018, putting bike-sharing on the online portal to provide users with a smooth and trouble-free ride experience. Ola, Uber, and Rapido are currently major aggregators in Guwahati providing app-based bike taxi services in Guwahati by booking through their online apps.

3 Methodology

The strategy adopted for the research to achieve the objectives is displayed in Fig. 1. Seven key performance indicators are considered to evaluate overall performance and identify performance areas that require improvement. Likewise, for NRA, 11 service attributes indicating the service quality of app-based bike taxi services are recognized. Together, the two evaluation techniques show where the app-based bike taxi excels and where it needs to improve.

3.1 KPI for Benchmarking

The performance areas have to be recognized to evaluate the performance of app-based bike taxi services in Guwahati by benchmarking. Comfort, mobility, convenience, affordability, and ITS facilities are the performance areas that are essential for the trip makers and urban development authorities, as stated in Table 1.

3.2 Service Attributes for Numerical Rating Approach

Eleven service attributes are considered essential to evaluate the service quality from the passenger's point of view. Service quality is the difference between passenger expectations and perception of app-based bike taxi service. Self-rating questionnaires were used as a data collection method in this study. The attributes considered are given

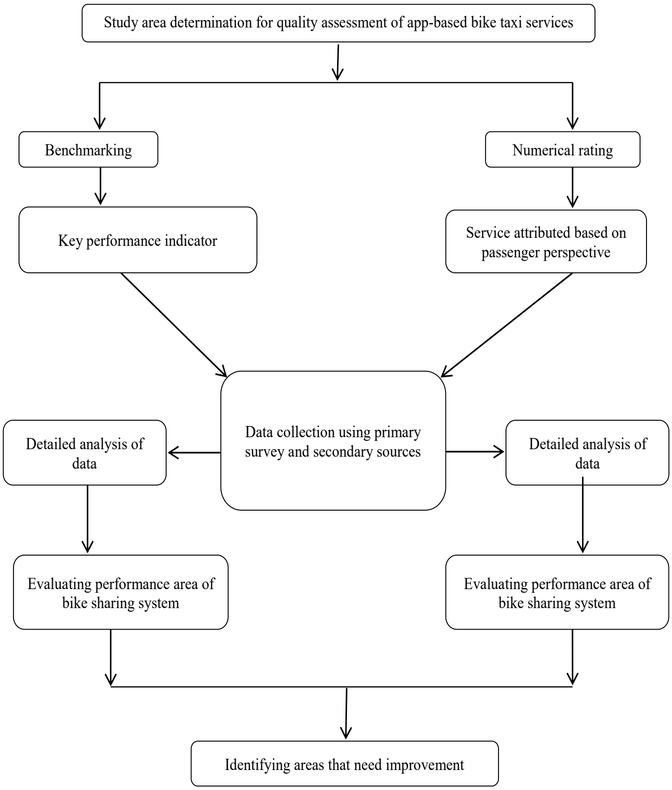

Fig. 1 Flowchart showing the methodology

in Table 5. The rating of the service attributes is based on individual experience and varies between individuals. The relative weights are given among attributes based on preference by passengers. Travel costs reflect the requirements for affordability, and depend on the income of the trip maker. Driver behaviour demonstrates the driver's attitude, complying with traffic rules, drinking, and driving.

4 Data Collection and Extraction

A trip maker survey was performed at major locations in Guwahati by random sampling. About 200 random passengers were asked to grade using a Likert scale ranging from very good (5) to poor (1). To assess the relative weight of various service attributes, passenger attitude survey was performed on a Likert scale. Speed and delay studies were conducted along major routes to find the average travel speed of app-based bike taxi services.

Table 1 Key performance indicators for benchmarking

Performance measure	Performance indicators	Method of KPI determination	Standard's derived from
Availability	Extent of supply	Supply of vehicle/1000 population	[8]
	Service coverage	Distance in route on which service services to the entire urban	[9]
Comfort	Driver behaviour, fare collection system, simplicity to use service, luggage carrying capacity	Index of acceptability	Author
Convenience	Waiting time	Average waiting time from trip maker survey.	Author
Mobility	Travel speed	Travel speed of service along major routes in the city	[8]
Affordability	Affordability	The ratio of income spent on bike taxi trips to their monthly income	[8]
ITS facility	GPS for vehicle	The ratio of vehicles with GPS functionality to total vehicle	[8]

5 Result and Discussion

The average trip length for app-based bike taxi service in Guwahati is found to be 4.8 km. It is found that just 20% of trips are longer than 6 km and 50% are shorter than 4 km. It also observed that most of the trip makers in less than 5000 thousand income group are in 20–25 age group who use service primarily for college or any other work-related trips. The survey shows that the online fare system, door-to-door service, and online booking are major attractions for choosing app-based bike taxi services. The performance by both benchmarking and NRA is explained below.

5.1 Performance of Bike Taxi by Benchmarking Technique

Each indicator considerably influences the app-based bike taxi service's overall LOS. The calculated Level of Service (CLOS) for chosen indicators section is discussed below:

Extent of Supply: Lack of bike taxis will lead to users switching, while an uncontrolled supply of bike taxis may cause traffic issues and safety concerns. The presence of bike taxis/1000 population obtained 10.44, hence based on Table 2, CLOS is determined to be 4.

Service Coverage: It measures how easily a service can be offered at different locations, more service coverage will result in better mode choice. The total length of the corridors on which app-based bike taxis ply in the city is 171.6 km. The area of the urban limits of the city is 216 km^2. The ratio of both gives the service coverage of app-based bike taxi service as 0.79. The respective CLOS is 2 based on Table 2.

Comfort: The CLOS measure for comfort was taken to be the geometric mean of the relative values for the four constituent elements of comfort. The relative values of the driver behaviour, simplicity to use service, luggage carrying capacity, and fare collection system are 0.668, 0.656, 0.528, and 0.674, respectively. The geometric mean of the four attributes of comfort is 0.6284, which shows the overall acceptance is 62.84%. Thus, based on the LOS criteria of comfort in Table 2, CLOS is found to be 2.

Average waiting time: The degree of waiting time is directly concerned with the reliability of the bike taxi service. The average waiting time calculated from a survey at different locations was found to be 6.16 min. Therefore, the CLOS for the average waiting time of app-based bike taxis is found to be 3 based on the LOS criteria.

LOS criteria for the above indicators are given in Table 2.

Travel speed

Traffic congestion is reflected by travel speed. The average speed of app-based bike taxi service along major routes in the city is obtained to be 30.40 kmph, which indicates CLOS of app-based bike taxis for the city is 1.

Affordability

According to the trip maker survey, the average trip maker affordability across all income levels is 13.12%, and the CLOS is 2.

Table 2 LOS criteria for extent of supply, comfort, service coverage, and average waiting time

LOS criteria	Extent of supply	Comfort (geometric mean of relative values)	Service coverage	Average waiting time (minutes)
1	<4	>0.85	≥1	<3
2	5–6	0.85–0.5	0.7–1	3–6
3	7–8	0.5–0.25	0.3–0.7	6–10
4	>8	<0.25	<0.3	>10

Table 3 LOS criteria for GPS for bike taxi, affordability, and travel speed

LOS criteria	GPS for bike taxi (%)	Affordability (%)	Travel speed (kmph)
1	≥75	<10	>20
2	50–75	11–14	18–20
3	25–50	15–19	16–18
4	<25	>20	<16

Table 4 Performance report for app-based bike taxi services in Guwahati by benchmarking

Indicator	CLOS	Suggestions to improve app-based bike taxi service
Extent of supply	4	More bike taxis should be registered and should operate zone wise
Service coverage	2	App-based bike taxi services can be made more available on the routes where the city bus services are less operational
Affordability	2	Dynamic prices should be standardized
Comfort	2	The traffic police should monitor adherence to traffic regulations by drivers
Average waiting time	3	During peak hours and odd hours, the frequency of bikes may be increased
Travel speed	1	Travel speed matches the criteria, since motorized two-wheelers are more prone to accidents, the speed should be regulated for safety
GPS for mode	1	LOS of GPS is more than targeted, but sometimes passengers have trouble tracking the driver's location. The app should be able to tell the precise location of the driver and passenger to each other

GPS for bike taxi mode

For the app-based bike taxi service bike rider and the passenger, both have the application of a respective aggregator (Ola, Uber, Rapido), which provides a continuous live location to both passenger and driver. So, the ratio for vehicles with GPS and without GPS is 1 giving the CLOS as 1. The level of service criteria for the above indicators is given in Table 3.

The mean of CLOS of all the indicators shows the overall LOS of the app-based bike taxi services. The overall LOS of the app-based bike taxi service is the mean CLOS of all the KPIs. The overall LOS is found to be 2, This reflects the performance of app-based bike taxi service by benchmarking. Suggestions to improve services are given in Table 4.

5.2 *Performance of Bike Taxi by Numerical Rating Approach*

Quality of service (QoS) is computed for every quality attribute. The methodology for numerical rating is based on the rating of service attributes and relative weightage among them. From survey ratings, the relative weightage and service quality with

Table 5 Deficiencies in service levels using NRA

Service measure	Relative weight, W_i (scale value) (1)	Service quality R_i (w.r.t unity) (2)	QoS$_i$ (3) = (1) * (2)	Acceptance level, 60% of scale value (4)	Deficiency from acceptance level (5) = (3)–(4)
Prior information about fare	0.0712	0.8987	0.0640	0.0427	0.0213
Prior information about journey time	0.0696	0.8733	0.0608	0.0418	0.0190
Availability of service (in terms of day and night)	0.0653	0.7547	0.0493	0.0392	0.0101
Luggage carrying capacity of service	0.0613	0.5053	0.0310	0.0368	-0.0058
Driver behaviour during the journey	0.0614	0.6173	0.0379	0.0369	0.0011
Fare collection system (in terms of by cash or online)	0.0691	0.7053	0.0487	0.0415	0.0073
How easy it is to book a ride in android app	0.0730	0.7147	0.0521	0.0438	0.0084
Ride acceptance by service provider	0.0792	0.6493	0.0514	0.0475	0.0039
Reliability (in terms of arrival time at pick up point and reaching time at drop point)	0.0805	0.7293	0.0587	0.0483	0.0104
Accessibility (in terms of pick up and drop point)	0.0765	0.8653	0.0662	0.0459	0.0203

(continued)

Table 5 (continued)

Service measure	Relative weight, W_i (scale value) (1)	Service quality R_i (w.r.t unity) (2)	QoS_i (3) = (1) * (2)	Acceptance level, 60% of scale value (4)	Deficiency from acceptance level (5) = (3)–(4)
Travel cost saving as compared to other modes	0.0654	0.7427	0.0486	0.0392	0.0093
Overall quality of service			0.7341		

respect to unity were calculated. The overall QoS is the summation of $QoS_{i,}$ as indicated in Table 5. It can be concluded from Table 5 that passengers give more weight on reliability, accessibility, prior information about the fare, ride acceptance, and ease in booking rides. The passengers' perception of service quality is more on the attribute's prior information about the fare and journey time, accessibility. It is least for luggage carrying capacity and driver behaviour. Table 5 shows that the QoS_i for the attribute's prior information about the fare and journey time, reliability, and accessibility are more. It is also discovered that the QoS_i ride acceptance and driver behaviour are lower. A deficiency from the acceptance level is observed for the attribute luggage carrying capacity, which is abundantly clear for bike taxis.

6 Conclusion

The performance of app-based bike taxi service has been assessed using benchmarking and NRA. Benchmarking shows that the app-based bike taxi services in Guwahati are performing better in real-time location tracking, travel speed, service coverage, affordability, and comfort. The better performance in these areas is predominantly due to services like online ride-booking, door-to-door pickup and drop, live location tracking, and multiple fare payment options. The app-based bike taxi service performance is found to be poor in extent of supply, and average waiting time. The poor performance in the extent of supply is primarily due to less supply of rides in outer areas of the city by app-based bike taxi services. Furthermore, the poor CLOS in average waiting time is due to traffic congestion in the city, and the denial of service by some bike taxi drivers to a specific location increases the average waiting time. The overall LOS of app-based bike taxi service is obtained to be 2, indicating room for improvement in some areas. NRA shows the passenger put more weight on the attributes prior to information about the fare, journey time, and accessibility in terms of pick up and drop while travelling. 73.40% of users are estimated to be satisfied with the app-based bike taxi service, according to the QoS measurement of 0.7341. Thus, the benchmarking and the NRA assess the performance gaps of the app-based bike taxi services from all dimensions. For greater accessibility in particular regions, the app-based bike taxi services may be controlled and should operate on a perimeter basis. Motorized two-wheelers are more vulnerable to accidents, so safety measures must be followed more strictly. App-based bike taxis provide ample scope to fill the gap in transit services during odd hours. However, regular monitoring of app-based bike taxi services is required to improve the mobility and overall quality of the service.

7 Future Scope and Limitations

Some additional performance areas and indicators, such as travel time ratio occupancy, ridership, and accident rate, can also be considered to assess the overall performance of the app-based bike taxi services. A comparative study of app-based bike taxi services with app-based cabs and traditional taxi services can be made considering different cities with different populations.

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Car Parking in Indian Cities: A Review of the Impediments to Sustainable Mobility

Paulose N. Kuriakose and Binayak Choudhury

Abstract Irrespective of how little pollution is emitted by cars or their fuel efficiency, it is required to park cars somewhere. Poorly designed parking policies can induce vehicle ownership, urban sprawl, and less patronage for public transport. The goals it should fulfil hence come from the greater agenda of sustainable urban development that usually includes a strong and vibrant economy supported by a proficient transport system, clean urban environment, better accessibility, a safe environment, and a more equitable society. In India, a country with rising urbanization level and the concomitant induced derived demand for mobility, scanty research has been undertaken on the implications of parking policies. This study attempts to understand the current enabling policy and legal environment, institutional mechanism, existing parking strategies, and pricing and its impact on average generalized cost of trip in Indian cities and it also probes the possible policy implications for the future. This study relies on exploratory research methods based on secondary data available with various institutions and organizations and focused group discussions with different stakeholders. The study finds that India is depending on Generic Minimum-based parking approach that doesn't consider transit proximity, popularity of a particular establishment, walkability, income and parking management practices like availability of public parking lots, parking pricing, and overall peak demand. There is an urgent need to provide efficient legal support for the creation of institutional mechanism, unbundling of parking pricing, adoption of smart growth parking policies.

Keywords Parking policy · Parking pricing · Travel demand management

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1 Introduction

Irrespective of how little pollution is emitted by cars or their fuel efficiency, it is required to park cars somewhere. Since a car uses several parking spaces each week [8] and spends about 95% of its life parked on an average [14], it has a conspicuous impact on both land use and the transport network [9, 14]. Lack of curb parking became a big problem with increased vehicle ownership during the 1920s in the United States of America. Therefore, in order to address the growing demand for on street parking urban planners came up with the idea of 'Minimum Parking Requirements' for various types of land uses [11]. The system of Minimum Parking Requirements as per the land use impacted the imaginations of urban planners in many other countries. But this policy induced urban sprawl reducing the density of residential and commercial development and over supply of parking spaces thereby encouraging further car dependence [12, 13]. Various researches have established that sound and pragmatic parking strategies would ensure effective use of public transport, smart land use development, reduced GHG emissions, and equity in built environment development [1].

Parking policy should not be developed in a standalone manner; it should instead be a part of all hierarchies of initiatives from local area plan to regional built environment plans [9]. The goals it should fulfil hence come from the greater agenda of sustainable urban development that usually includes a strong and vibrant economy supported by a proficient transport system, clean urban environment, better accessibility, a safe environment and a more equitable society [8]. To deal with the issues of generic minimum-based parking and to manage the overall demand for mobility, urban planners and transport experts have developed new approaches to parking supply management. These mainly include area-specific parking and flexible parking. Under Area Specific parking standard, the entire city would be classified into various zones and each zone would have its context-specific parking requirement. Since this approach can segregate areas with lower parking demand and ensure a better range of transportation alternatives, it can make it easier to begin phasing in more progressive measures, such as parking caps or parking maximums [6]. Parking supply reduction and usage were guided by various cross elasticities involved in parking market. This is done by restricting parking through suitable charges in the urban core where the density is high [18]. The American Planning Association in 1983 published Flexible Parking Requirements, detailing out various flexible approaches, includes fees in lieu of parking, off-site parking, shared parking, and parking reductions to induce ride-sharing programs [15]. Cities have linked the minimum or maximum parking norms to site-sensitive variables such as availability of nearby offsite parking (e.g., public parking), transit accessibility, and the availability of trip aggregators [7] This approach allows having different parking supply norms for different urban development precincts according to the context-sensitivity [6].

In India, a country with rising urbanization level and the concomitant induced derived demand for mobility, scanty research has been undertaken on the implications of parking policies. This paper makes an attempt to find answers to a few pertinent questions, namely, (i) what are the parking regulations across the states and cities in India? (ii) what have been the difficulties in the enforcement of parking regulations? (Sect. 3.2), (iii) are there any differences in the parking regulations across various statutes and codes? (iv) how parking norms in residential area and commercial area differ across states and cities? (v) what have been the emerging issues in on-street parking?

2 Methods

The present study aims to assess the current status of parking policies, regulations and management and their impact on urban mobility in India. This study relies on exploratory research methods based on secondary data available with various institutions and organizations and focused group discussions (FGD) with different stakeholders. FGDs have been conducted with concerned stakeholders like Municipal Corporations (MC), Development Authorities (DA), and commuters in the urban areas in order to get a deeper insight about the existing situation and about the issues and imperatives towards resolving the problems in parking management. Stakeholders' survey consisted of a set of open-ended questionnaire structured around the following eight aspects: (i) existing parking standards and increasing private vehicle ownership, (ii) parking pricing and its impact on the average generalized cost of a trip, (iii) current parking policy and legal enabling environment, (iv) current institutional arrangements, (v) awareness about modern parking management measures, (vi) benefits of parking maximums, (vii) cost of parking provision, and (viii) other challenges and opportunities in parking management. Accomplished through literature survey, the first stage includes the appraisal of various approaches to parking supply and the evolution of parking management policies. In the second stage, a detailed assessment of parking management policies in India is made. In this stage, the impact of minimum parking standard on built-up areas in residential and commercial zones, parking pricing, revenue of MCs, protest against the parking pricing in shopping malls, cost of parking space provision etc. are analysed in detail. In the conclusion, the possible interventions and recommendations for making parking policy complementary to travel demand strategy have been suggested.

3 Parking Management in India

As on 2011, the urban population was about 31% in India [2]. One estimate predicts that the total urban population of the country would touch 40% by 2021 [5]. The pace of urbanization in India is quite significant (as compared to the global average and average of Asia, Europe, Latin America and North America) and this has been inducing the derived demand for mobility. While the number of buses and other public passenger vehicles registered in India are on the decline, that of private cars and motorcycles are increasing at an alarming rate despite the economy being pressurized with a higher fossil fuel import bill. The disproportionate dominance of private motor vehicles not only inflicts various negative externalities on the environment but also creates an inequitable appropriation of urban spaces.

3.1 National Urban Transport Policy (NUTP) and Parking

In 2006, the government of India adopted NUTP in order to better coordinate urban transport development [10]. The NUTP acknowledges the fact that an increase in private vehicle ownership leads to a greater damage to sustainable living in urban areas. It also highlights the issues of low parking charges, congestion, decreased use of NMT, absence of land use transport integration, and accidents and sets forth a multi-pronged agenda for improving the urban transport scenario. The objective of this policy has been to ensure integrated land use and transport planning in all cities to (i) minimize travel distances, (ii) encourage more use of public transport, (iii) ensure equity is road space allocation, (iv) introduce multimodal public transport systems so that a well-integrated, seamless travel across modes are put in place. NUTP underscores the importance of land as a valuable resource of urban areas, a large portion of which gets occupied by parking spaces. This particular fact must be acknowledged and parking fees, commensurating the value of land occupied by the parking lot, should be adopted to make the use of public transport more attractive. The policy further envisages that encouragement to park and ride facilities for bicycle users, with a suitable inter-change, would be another advantageous measure to encourage people to use public transport for their different trip purposes. But the policy loses its track when it aims to modify building by-laws in all million-plus cities to 'make sure adequate parking space' is present for all residents. Thus eventually, the rising private vehicle ownership would be adding more congestion on city roads and demanding more parking spaces across all other land uses [10].

3.2 Parking Regulation and Enforcement in India

The supply of parking places is regulated in India by minimum parking requirements. Parking standards are decided by the Indian Road Congress (IRC), National Building Codes, The Motor Vehicles Act 1988 and Development Control Regulations under Urban Development Acts. The 1997 IRC Standards recommend the provision related to parking while laying roads. While the 1988 IRC Guidelines stipulate the requirements related to off-street/building parking, the 1997 Guidelines specify the road marking requirements for parking on-street. The Motor Vehicle Act, 1988 lays down the traffic and parking area regulations as well as the enforcement architecture. It forbids parking near or at road crossings, on a footpath, hindering another vehicle. Section 117 of the Motor Vehicle Act, 1988 permits the State Government or any other specific authority on its behalf to determine places at which motor vehicles are allowed to park in consultation with the local authority having jurisdiction in the specific area. MC and City Traffic Police are responsible for the enforcement of parking rules. Available public off-street parking spaces and major street stretches are leased out to private contractors for parking management. Although the Town Planning wing of respective MC is responsible for ensuring the provision of parking supply as per the building control regulations, indiscriminate and blatant violations of building control regulations by delinquent builders have been a routine affair. There is no proper database available with the MCs or Development DAs across the country on the parking supply actually made available against the approved building plans.

3.3 Residential Area Parking in India

All the residential parking standards presume that vehicle ownership increases with the increasing population and prosperity of the city and pay less attention to the fact that the share of public transport and proximity to MRTS routes need to be considered to fix caps on parking supply. None of the factors like transit accessibility, mixed land use, high residential or employment density is considered in deciding the parking standards. The floor space index (FSI) or total built-up areas are normally related to residential parking standards. With the spatially diverse increase in household vehicle ownership and income over time, few cities have been raising their minimum parking requirements. According to the Building Bye-Laws 1983 of Delhi Development Authority, one Equivalent Car Space (ECS) was to be provided for every 90 m^2 built-up area in group housing. But in 2013, it was amended and now it is required to provide 2 ECS for every 100 m^2. Ahmadabad DCR of 1991 made it mandatory to provide 15% of the utilized FSI for parking. However, the latest DCR of Ahmedabad Master Plan 2021 mandates that 20% of the FSI should be kept for parking for group housing and detached houses should have one ECS for more than 80–300 m^2 floor area with additional one ECS for every 100 m^2. floor area. Table 1 depicts a scenario of residential built-up area of 5000 m^2 and required parking spaces in different states and cities in India. As is evident, every authority follows a different methodology in deriving the standards. As it can be observed, some of them have segregated requirements based on the residential typology, while others have not. In the whole of Kerala State and Chennai city, the parking requirements are not based on the classification of the residential typology. Across the country, two important aspects have not been taken into consideration: (i) the cost of providing a parking space and (ii) variation in real estate market and the land availability with the location.

Parking standards are not uniform throughout the city of Mumbai regulated under the Development Control Regulations for Greater Mumbai. Though the residential typology segregation is not followed in Mumbai, it has an area or location-specific parking regulations. Plot—Area-based parking requirements are stipulated in the Chandigarh Master Plan, which is not followed in any other cities or States surveyed

Table 1 Car parking requirements for 5000 m^2 of residential area

Authorities/building rules and regulation	Total ECS	Parking area
Ahmadabad	30	750
Kolkata	28	350
Bangalore	50	687.5
Chennai	67	833.33
Greater Mumbai	84	1155
Pune	50	625
Bhopal	21	260.42
Surat	40	1000
Chandigarh	60	1500
Andhra Pradesh	60	1500
Delhi	90	1237.5
Kerala	7	175

Source Respective Development control regulations of the cities and states

under the present study. It is only under Unified Building Bye-Laws and Development Control Regulation of Master Plan for Delhi that studio apartments are mentioned as a residential typology. Although categorization based on the residential typology is followed by most of the cities and States in India, this specification is not followed in the State of Kerala, and across the cities of Mumbai, Pune, and Chennai. In these cities and State, the requirement is the same for all the types of residential buildings. Such a formulation of standards, however, is not a rational approach to follow.

In the cities of Ahmedabad, Chandigarh and in the State of Andhra Pradesh, the parking area is calculated by considering a fixed percentage of the total built-up area. Such straightforward approach is very irrational as it allows a certain percentage of land to be used by vehicles, which could otherwise have been used for housing people. This can be illustrated by the General Development Control Regulation (GDCR) of Ahmadabad, where 750 m^2 area is assigned for parking for the total built-up area of 5000 m^2. Given the standard size of Economically Weaker Section (EWS) housing being 30 m^2, it can thus be found that at least 25 EWS housing units could have been built up within the parking area as provided under the GDCR of Ahmedabad. But on the contrary, in Kolkata, if it exceeds 200 m^2, then for each additional 200 m^2, one car parking space should be provided.

3.4 Commercial Area Parking in India

The principle of 'more the car ownership more the parking space' is applied in commercial areas also. Such parking requirements make the city friendly to cars but not to people, make the city drivable but not walkable [14]. Table 2 represents the car parking requirements for commercial buildings in some cities and States in India according to their building rules and regulations. The total built-up area of the commercial building considered here is 5000 m^2, and the comparison table is prepared accordingly. There are different types of commercial buildings, each having different parking requirements. But concerned authorities and respective building regulations of the city of Ahmedabad and the State of Kerala have not considered this fact and instead imposed a common parking requirement for all types of commercial buildings. On the contrary, several Indian cities follow specific parking requirements based on the typology and the location of the commercial building. The space required for car parking in commercial buildings of the cities and States covered under the present study varies from a minimum of 312.5 m^2. for the city of Bhopal to a maximum of 3000 m^2. for the State of Andhra Pradesh. The requirement for Bhopal is almost 10 times lower than that of Andhra Pradesh. One of the main reasons for this is that while the parking requirements and the categories of the commercial buildings are very specific in Bhopal, the parking requirements are based on a certain percentage of the built-up area irrespective of the size or location of the commercial buildings in the case of Andhra Pradesh. In Delhi, it is not mandatory to provide parking space for informal bazaar (market). Such erroneous regulations (or absence of regulation) encourage on-street parking and cruising for parking spaces and eventually cause congestion. Although the motorized two-wheelers contribute a significant share in the personalized mode of transport in India, none of the regulations referred to under the present study mentions the parking requirement for motorcycles except Chennai. As mentioned above, the parking requirements are indicated as a certain percentage of the built-up area in the States of Kerala, Andhra Pradesh and cities of Ahmedabad, and Surat. This is certainly not the appropriate method to calculate the parking requirement, which should have been based on the type, location and size of each unit of the commercial building. Segregation and specification based on these parameters should be considered while formulating the parking norms.

Only in the case of Chennai and Kolkata, no parking requirement is provided up to a certain built-up area. Under the regulations in Chennai, no parking is required up to 50 m^2 of built-up area for each commercial unit. In the case of Kolkata, this exemption is extended up to 25 m^2 of carpet area of each of the commercial unit. Area or location-specific parking norms are stated in detail only under Delhi regulations. In Bhopal Development Plan Building Regulations, only the area is specified between the upper-income group areas and other-income group areas without any reference to the locations these income groups belong to. In the case of Mumbai, location-based segregation is done only in the parking requirements for residential buildings with total disregard to the parking requirement for commercial buildings.

3.5 *On-Street Parking*

Most of the Indian cities have an old city core that follows a different urban development paradigm with narrow streets, highly mixed land use, heritage structures and

Table 2 Car parking requirements for commercial buildings

Authorities	Parking norms	Number of ECS	Area @25 m^2
Ahmadabad	30% of max. permissible F.S.I	112	1500
Andhra Pradesh	Multiplexes, shopping malls—60% of FSI	218	3000
Kerala	1ECS/100 m^2	50	1250
Chennai	1ECS/and 1 two-wheeler space for every 50 m^2	99 + 15	2580
Kolkata	1 ECS/35 m^2	143	3575
Bangalore	Retail business 1ECS/50 m^2	100	2500
	Multiplex integrated with shopping—1ECS/40 m^2	125	3125
Pune	1ECS/100 m^2	50	1250
Mumbai	1ECS/80 m^2	65.2	1630
Delhi	Local shopping centre 2 ECS/100 m^2	100	2500
	District centre/subcentral business district 3ECS/100 m^2	150	3750
Chandigarh	Multiplex/malls—4ECS/100 m^2	200	5000
Bhopal	1ECS/45 m^2 of floor space	111	2777

commercial activities. Many of these urban fabrics have come up before the arrival of automobiles and minimum-based car parking standards. However, sadly enough, high traffic density and illegal on-street parking in these core city areas have led to a completely chaotic situation over the years. The carrying capacity of the streets in these core areas in particular has been shrinking over the years due to the indiscriminate on-street parking posing a serious barrier for the smooth movement of public transport and non-motorized transport. In the early 1980s, the Government of India has brought out parking guidelines following which many commercial buildings have started providing parking spaces. Although the commercial establishments had provided the parking spaces as per the guidelines in those days, most of the people had converted the parking areas into other commercial uses and depots since the demand for parking was meagre owing to low ownership of private vehicle. However, the economic reforms undertaken in 1991 and the consequent liberalization, privatization and globalization of Indian economy reversed the scenario dramatically. The exponential growth in the ownership of private vehicles far exceeded the availability of off-street parking within the building thereby making the car owner end up in parking the car on the street.

3.6 Parking Pricing

Distortion in cost of trip is created by low parking charges and/or the availability of free parking space. Major share of the commuters who use personalized vehicles do not pay for the total supply cost of parking, which leads to large inadequacies in transport pricing [3]. Parking charges in India is one of the lowest in the world. Even where the parking charges are levied, it is not harmonized between off-street, on-street and multi-level car parking (MLCP). The Capital cities in each Federal States in India experience a high percentage of government employees that use either a car or two-wheeler for daily commuting. Government offices across these Capital cities provide parking space free of cost. Free parking provision at the workplace is an important factor in indirectly inducing the use of personalized modes of transport. An assessment of various trip purposes shows that close to 75% of the peak hour trips are made for work and education. As large as 90% of on-street parking in arterial and sub arterial roads are available free and since on-street parking is available free, most of the MLCPs remain largely underutilized.

There is a strong protest going on in almost all metro cities against the parking fees charged by shopping malls. Commuters are claiming that it should be provided for free. Many court cases are lying before the Consumer Courts and High Courts across India against the parking fee at shopping malls. Following the 74th Constitutional Amendment Act 1992, MCs are the statutory agencies that should ensure the earmarking, regulation and supervision of parking places. There is a huge cost associated with the provision of various types of parking, namely,—off-street, surface, stilt and multi-level parking. On average, one ECS of off-street parking would cost close to Rs 0.5 million and Rs 0.2 million in the urban and peri-urban areas respectively. Stilt or basement parking would cost about Rs 0.6 million and Rs. 0.25 million in the core urban and outskirts of the urban area, respectively. Cellar or MLCPs are the costliest parking; it costs close to Rs 1 million and Rs 0.5 Million for one ECS at the CBD areas and the suburban areas, respectively. These costs majorly include two components: the cost of land and cost of construction. Providing such a costly infrastructure free or at sub-optimal price would always bring loss to the exchequer. Mall authorities should have the natural rights to decide whether the cost of providing parking space is recovered from the retailers (storekeepers) housed in the mall or the shoppers visiting the mall or from both besides deciding the duration-wise parking levy. In view of the huge cost in the construction of parking space in malls, and in the event of the government preventing the malls from collecting parking fees, mall authorities will be forced to recover the cost by raising the rent of the leasable space. It would create a situation in which a public transport user shall indirectly pay for the parking space that they do not use. Parking management practices that indulge in sub-optimal pricing and appeasement populism may cause indirect inducement to private vehicles and the slow death of public transport.

3.7 Who Pays for Parking?

Parking price levied by the municipalities are only applicable for dedicated off-street and on-street parking. It distorts the average generalized cost of the trip for various types of commuters. It has been observed across Indian cities that while the visitors to commercial areas largely pay for parking, employees get assured parking spaces in their workplaces free of cost. If a trip by private car and metro respectively is compared having the same origin and destination, it would reveal that trip by private car is cheaper due to the mere absence of parking price at the destination. The metro users, on the other hand, have to pay the parking charges at the Metro parking plaza and take the ride.

There is a regulatory deficit with the parking regulations both in terms of space and location parameters as well as in terms of enforcement. The parking regulation that ought to have regulated and promoted the parking market has been largely dysfunctional due to the bundling of parking price in the end product or services. There are three major scenarios of bundling: (i) firstly the bundling of parking-space price with the house price; (ii) secondly, provision of free parking for the employees by the respective employers; (iii) thirdly, the bundling of parking price with the product or service price by commercial establishments. For example, if the parking space price is bundled with the price of the house by default, buyers or renters need to pay for the parking space facilities even without owning a vehicle. If the home buyers are given an option to avail a lower price of the house by charging for the parking facilities separately, they are likely to reduce their vehicle ownership or even not owning a vehicle at all (assuming that the neighbourhood enjoys a very good and affordable public transport accessibility). Similarly, if employers (office/workplace) start charging the employees for parking facilities, there is every likelihood that employees would shift to other modes of transport and save the parking charges to add on to their disposable income. If parking comes as a bonus to the employee, they would tend to use their own vehicles for office commuting. A comparison of average generalized cost (AGC) of trips by a car and car and metro reveals that a person using metro for the line haul trip and car for access trip (to metro station) will end up paying more than double the AGC of the direct trip made by car only. This substantiates the need for a comprehensive parking-pricing strategy in keeping with the bigger principles of sustainable mobility.

There is an absence of off-street parking market across different land uses in Indian cities. The high social cost associated with the exponentially growing use of personalized vehicles gets obviously ignored by an unregulated market. The Capital cities in each Indian State experience a large percentage of people working in government, commercial, institutional sectors using four or two wheelers for their day-to-day trips. Government offices, many commercial, industrial and institutional establishments across these Capital cities provide parking space free of cost to their employees as well as visitors. Provision of parking spaces for free at the workplace is an important factor in indirectly inducing the use of personalized modes of transport. Although cities like Delhi have constructed Metro with huge capital expenditure, it failed to bring a modal shift from personalized modes to MRTS in any perceptible extent. Municipal revenue collected from parking has been abysmally low at less than 1% of the total municipal revenue across Indian cities.

4 A Way Forward: Policy Implications for the Future

From the analysis made above, the following factors seem to have afflicted the parking management in different ways in Indian cities: (i) adoption of conventional generic minimum parking approach, (ii) absence of scientific rationale in parking pricing, (iii) bundling of parking pricing as implicit subsidy, (iv) absence of legal support and (v) poor transport governance.

International experiences of maximizing social welfare and promoting sustainable urban transportation underscore the urgent need to restructure the management of parking across the cities in India. In order to achieve the goals of curtailing traffic congestion, improving air quality, reducing greenhouse gas emissions, making streets more liveable, freeing up road space for public space, and providing bicycle lanes, parking policy has been implemented in a large number of European and North American cities. Increasing attention is being paid to regulate the parking provisions to such levels that the roads can support besides ensuring the normative air quality standards. Efficient parking management is acknowledged as integral to competitive and liveable cities. Indian cities, in their endeavour to become competitive, efficient and liveable, can gain so much from the best practices of European cities. There is an urgent need to make an integrated parking and auto ownership management plan for Indian cities. The explosive increase in the ownership of private vehicles and the ensuing decrease in the level of service on the road network and other negative externalities originating from the fleet of private vehicles can be better tackled through integrated parking management strategies, particularly that of travel demand management tool.

4.1 Modifications in Parking Supply Approach

There is a need for the modifications of parking standards by adopting parking maximums, parking supply caps, and flexible supply of parking spaces. The presence of mixed land use, walkable streets, bicycle-sharing facilities, public transport accessibility, employment density, residential density etc. should be given due importance in fixing the parking rules [17]. The application of uniform standard at pan city level should be abolished and parking districts should be delineated. House registration procedures should be modified to unbundle the parking and separate registry needs to be maintained for parking spaces. For the recent largely growing private parking market, regulating the number of parking spaces can turn out to be largely helpful in parking management. As the parking standards are reduced in a building, this space can be utilized for increasing the built-up area of that building or for a new building for low-income group housing, provided it complies with the building regulations. Hence, this can act as a significant tool in increasing the housing supply. Rather than focusing more on minimum-based parking approach, parking authorities and planners should begin to consider maximum parking [14]. Maximum parking spaces should be estimated by taking into account all the costs and benefits associated with it. The components of costs and benefits associated with parking maximum may be contextualized to a few questions: (i) whether the traffic on the adjacent road is affected due to these extra parking requirements (decline in Level of Service)?; (ii) what is the cost of polluting the environment by the vehicular flow?; (iii) how and by what extent does the parking maximum affect the real estate prices?; (iv) who will pay for this additional cost?; (v) should a home buyer pay for parking space if s/he does not require it?; (vi) what is the opportunity cost of a parking space? Through this approach, we can try to control the demand for parking by reducing its supply.

4.2 Adoption of Rational Parking Pricing

A rational parking pricing should essentially deter the use of personal vehicles. Parking price is a widely accepted and an efficient measure to manage the modal shift [13] besides reducing vehicle congestion on urban highways [9], and coping up with parking demand in high-trip attraction urban zones. Parking price makes personal vehicle users pay directly for the use of parking facilities [14]. Besides regulating parking, parking price acts as a mobility management strategy. It collects revenues to recoup the capital and operation and maintenance expenditure of the parking facility besides supplementing the municipal coffer. At present, most of the parking plazas are inefficiently priced and/or subsidized and/or just provided for free and/or bundled with home price/rentals (notwithstanding the need for parking by the home buyer/renter). Vehicle owners do not face any disincentive for the use of their vehicles as they are just required to pay mostly meagre flat monthly or annual parking fees and as a result, the likelihood of their migration to public mode of transportation is very thin [8]. Therefore, performance-based pricing should be adopted to increase the efficiency of the use of parking space which implies that almost 15% of the total parking spaces must be made available and vacant at any point of time [14]. Intelligent Transport Systems and Innovative Parking Pricing mechanisms should be followed for equitable and efficient utilization of urban land and to augment the municipal income.

4.3 Legal Provisions for Parking Management: Lessons from Best Practices

The institutions responsible for managing parking in Indian cities should emulate the best practices in parking management achieved in foreign countries besides undertaking the pertinent fiscal and administrative reforms.

The successful implementation of parking strategy requires strong legal support. Under the Japan Parking Places Law—1958, any car buyer is required to produce the certificate of availability of parking space (proof-of-parking) to register the vehicles [4]. This proof of parking regulation helps manage the street parking in residential areas. The UK Parking Places (Surcharge) Act of 1975 charges a monthly fee on non-residential parking. The UK Transport Act (2000) helped implement the workplace parking levy in Nottingham. Adopting Clean Air Laws, Santa Monica, California mandated that parking cash-out measures should be implemented in firms with 50 or more employees [14]. It also aims at eliminating haphazard night-time parking in streets and alleys. The Perth Parking Management Act, 1999 helped create an area called the Perth Parking Management Area (PPMA) within which there is a requirement to license all parking except private residential parking spaces. The Act gives powers to levy tax on all types of parking spaces except the residential parking spaces and collects revenue that can only be used in the PPMA [16]. Parking fees and fines can be a major source of revenue for MC and proper ring fencing of this revenue can help the urban local bodies achieve the goal of sustainable mobility. Suitable statutes need to be enacted and enforced to implement the workplace levy so that it can impact the average generalized cost of not only the visitors to various establishments but also that of the employees. Transport Department should ensure that vehicle registration is made mandatorily dependent on the production of certificate of availability/ownership of parking space. Registration of parking spaces can help reduce private vehicle ownership due to the spatially and dynamically efficient taxes on parking space. Japan has implemented this kind of parking strategies and achieved reduction in parking supply as well as land use—transport integration [9].

4.4 Strengthening Transport Governance

Deficit in transport governance has been one of the stumbling blocks in managing parking in India. Parking enforcement is an important aspect in parking management. In India, the responsibility to enforce the parking rules lies both with the traffic police and MC of respective cities. It, therefore, reiterates the need for United Metropolitan Transport Authorities (UMTA) for metropolitan areas and similar such institutional architecture for other cities and towns. There should be a dedicated wing in each UMTA for parking management within the overarching principle of public transport promotion and travel demand management. UMTAs should maintain the database of the entire on-street, off-street and public and private parking supply for effective policy implementation and monitoring. Cities should be classified into different parking districts or precincts based on land use and activity characteristics to prepare separate parking management plan for each parking districts.

5 Conclusion

On society, the effects of parking go beyond the vehicle owners' costs. Most of the urban local bodies in India are not administratively and financially equipped for the efficient management of parking. Indian cities are replete with scores of examples where off-street parking is hardly strictly followed; leave alone the appropriate pricing for off-street parking. Matters worse when it comes to the enforcement of on-street parking regulations under the ambit of a wider travel demand management principle. Parking policies should be prepared in tandem with the policy objectives of transit-oriented development and metro rail. At a time when Indian cities are on the track of improving public transport, it is very much necessary to implement some other soft policy measures that induce more ridership by Mass Transit Systems. The central principle for parking policy should be to progressively reduce the demand for parking and use it as a strategy to induce a modal shift to public transport and other active modes. But at the same time, the policy should enable structured parking for all types of vehicles. Smart—urbanism-related parking methods are the new paradigm, which is a close associate of the larger sustainable development goals and equity. Selecting the suitable parking approach may depend on a city's development objectives and trajectories besides the market forces that drive parking supply. In some areas, a hybrid approach that combines several approaches might be useful. With the changes in urban growth pattern, mobility and vehicle ownership, Indian parking management plans should take into cognizance the multidimensional factors explained as above.

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Limits to Commute: The Case of Indian Women

Nachiket Gosavi and Naga Siva Gayatri Dittakavi

Abstract Research has shown that there is an inherent physiological and psychological difference between men and women resulting in differing commute patterns. Using the household level data from the 76th NSSO round on 'Drinking Water, Sanitation, hygiene and housing conditions;' The present research sheds light on whether the hypothesized commute differences are apparent. Additionally, the research investigates whether the commuting pattern of women is different in urban and rural settings. Further, the research tries to understand the determinants of observed commute patterns in women. This analysis is carried out by subjecting the data to stratified regression. The results are important for developing women centric policies.

Keywords Commute · Rural · Urban · Income · Similarity · Stratified regression

1 Introduction

Women are an important part of the workforce. Transport and gendered division roles are not only correlated, but women's opportunities and accessibility to the available prospects are adversely affected. Gender specific transport attributes such as the resource and time poverty faced by women, their work participation rates in urban and rural areas, high concentration of women in the informal and home-based production sectors etc. are more often than accepted as anecdotal evidence. This intersectional field of gender and mobility suffers from the constraint of availability of data. This constraint is deeply reflected in the research, policy and subsequent implementation of policy. While past investigations confirm that a mix of socio-demographic, economic, transport infrastructure and area characteristics may be helpful in capturing and explaining the commuting pattern, however this has not

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reduced the gender bias in transport policies [1]. Further, this bias is percolating in fast urbanizing developing nations like India. Certain important questions like How far do women commute?, What influences these commuting decisions? etc. remains unanswered.

After setting the need and significance for the research, in the next section, the literature on commute patterns is reviewed. In the third section, the methodology and the data sources are detailed. The fourth section examines whether men and women have identical commute patterns. In the fifth section, we investigate the variables that influence commute patterns. In the sixth section, we summarize our results. Additionally, we conclude by listing a few shortcomings of our article and the path ahead.

2 Literature Review

While past investigations confirm that a mix of socio-demographic, economic, transport infrastructure and area characteristics is helpful for capturing and explaining the commuting patterns in Indian rural–urban megaregion [2], opportunities and accessibility to the available opportunities are adversely affected. It is important to highlight this aspect in order to introduce a transport policy that integrates women's transport needs and which is sensitive to the effects of poor accessibility on the lives of the poor, it is only possible to carry out an investigation into the existing trends, as it is difficult to analyse these trends and suggest a plausible pattern based on the limited data available. The said study tries to investigate transport infrastructure to arrive at a conclusion on commuting patterns in an urban–rural megaregion only to ignore several other underlying parameters that determine the said patterns. For example, the study identifies the relationship between high income women and private vehicle usage among women to be positive, but it is merely correlational and does not define a causal relationship. In the same manner, the positive correlation in respect of high skilled workers and intermediate trips, income and increased private vehicle usage of women has been mistaken for a positive impact, i.e., a determinant. The word 'impact' implies a causality and not a correlation.

While other studies recognize the constraints faced by women with respect to commute, the underlying argument that "commute is determined by so and so" is still conjectural due to anecdotal evidence. Gender is one of the most important variables for mode choice [3]. The differences in travel behaviour by gender are mostly due to the complexity of activities more often experienced by women than men. To what extent does the complexity of activities affect commuting trips is not statistically defined to arrive at conclusions [4].

While it is difficult to factor every constraint faced by women, it becomes important to identify those variables that are statistically significant for commuting trips. The study by Jain [2] uses rail and road density in particular to understand commuting patterns. It proposes an increase in rail infrastructure while there is no objective evidence that it is necessary, when in fact the current rail infrastructure is underutilized.

Anand and Tiwari [4] explore the above constraints objectively, with the same setting of the urban-rural megaregion of Delhi, but due to a lack of information, no conclusion can be deemed fit enough. But one key derivation of the study shows that the lack of mobility of poor women (which is the premise) has a trichotomic effect: it is *caused by their poverty*, *is an indicator of their poverty*, and in *turn causes their poverty.*

Gender inequalities result in the social devaluation of women who are assigned a lower position in society. Gender interacts with transport systems to reinforce inequalities between women and men. Transport infrastructure and services play a facilitative role in women's and men's mobility and access; however, they can also impose physical restrictions on their ability to move [5].

In a country like India, social dynamics to a large extent have suffered due to the amalgamation of caste, creed and religion and its effect on the economic structure. Uteng and Cresswell [5] explain that the removal of women from spaces, due to their social exclusion by virtue of their belonging to a particular caste or creed, causes further social exclusion by distancing them from the physical society. This removal of the physical body from the socio-spatial realm results in an exodus of the physical removal from the inner city. Proper women citizens come into being through their belonging to spaces of normal femininity and respectability and are out of place in inner-city spaces where some other women belong [5]. The notions of race, religion and caste reflect socially in terms of respectability and femininity. These subjective factors cause, indicate and result in constrained mobility in India. However, in the developing stage that the country is in, where there is the constraint of income, and lack of security of employment, the above mentioned factors(caste, religion and creed and hence respectability) are not statistically significant in the context of mobility.

It is also worthy to notice that income and mobility for women is a bidirectional concept. Income restricts or promotes mobility, and mobility restricts or creates income. For women, income and ownership of vehicles are not as statistically significant as they are for men. Factors like safety play a very important role, and such factors differ in degree from one region to another, and so do the expectations and behaviour patterns. So it becomes difficult to pinpoint on a certain set of determinants that apply for all regions, and it becomes crucial to know how much of one determinant restricts the movement of women in their region.

On the periphery, it becomes necessary to look at housing arrangements of both the genders in urban and rural areas, distance from work, family setup, whether the family setup is car or income deficient and several other parameters while drafting a transport policy that is not only women-friendly but also sustainable for the environment [6]. Manoj et al. [7] states that housing tenures of working women in cities like Bangalore show precedence of owned establishments in commuting longer distances. Majority of transport policy is ex-post planning. As people's income rises, they prefer greater floorspace. There is always a growing market for high-end buildings on the periphery of large cities. As is the norm, wherever there is a unit of well-off people residing, a cluster of low income people move in search of low income employment. But the mode of transport and the constraints faced will not be the same for the two. Hence we have different travel behaviours exhibited by people living in the same region/locality.

There seems to be a dearth of structured investigations in understanding the statistical significance of various factors that affect commute, especially in the conjunction of women and commuting patterns. Some studies use the size of a particular area(urban or rural) as a gradient on the scale of small, mid and large to define how the commuting patterns remain the same in comparison to other developing countries. Advani and Mahadevia, [8] state that the proportion of 'no choice trips' or 'forced trips' is much higher among women due to the double disadvantage of constraints they face, and poorly planned transportation schedule/

transport infrastructure. However, it should be kept in mind that with geographic and demographic diversity in India, other mid-size towns in the same country may show patterns distinguishable from each other. Rogers and Srinivasan [9] show that there is a significant difference in travel behaviour because two localities in the same city show marked differences in accessibility.

Therefore, different determinants of travel behaviour for women exhibit different statistical significance in different cities, and sometimes even in different localities in the same city. Because the distance travelled is a function of factors, both measurable(and measurable objectively) and immeasurable(subjective). Women on average also travel less often and for shorter distances than men [10] and are more willing to reduce vehicle use than men [11]. The differences in travel behaviour by gender are mostly due to the complexity of activities more often experienced by women than men. However, the said studies on understanding the constraints experienced by women use anecdotal evidence, which reduces the conclusion to being merely conjectural.

Although there are discernable patterns in the determinants of transport choice for commuting trips across different continents among the two genders still varies due to the constraints that the commuters face, which are experienced differently in different countries. It is now widely acknowledged that transport policies are gender biased in as far as they fail to accommodate the specific gender-based needs of the transport users [1].

3 Objectives

From the literature, certain important questions like whether men and women have similar commute patterns, whether their patterns vary across rural and urban landscapes, or are their other determinants which influence the commute pattern of women remained unanswered. The present research is a modest attempt to understand this. For addressing this overarching question, we maintain the following objectives:

> Ascertaining whether commuting patterns are identical for men and women
> Investigating the influence of income on commute distance
> Examining the influence of socio-economic characteristics on commute distance
> Identifying the influence of family size on commute distance
> Scrutinizing the impact residential characteristics have on commute distance.

4 Methodology and Data Sources

For addressing these objectives we follow a purely quantitative approach. We use the third level of the 76th round of NSSO on “Drinking Water, Sanitation and Housing conditions.” [The National Sample Survey Organization (NSSO) is a part of the Ministry of Statistics and Programme Implementation (Mospi), Government of India. NSSO has conducted nationwide sample surveys on various socio-economic aspects since 1950. These surveys are conducted in the form of rounds extending normally over a period of one year. The sample is selected through a multi-level random sampling and is considered representative of India’s population.

Gender is one of the key socio-demographic variables that can influence travel behaviour, but it is often the least understood. As the focus of the present analysis is to understand the determinants of commuting in women, hence, the households where the women are not participating in commuting are removed from the data. This leaves us with 40745 observations or 38.17% of the original sample.

We begin our analysis by descriptive statistics. This provides a preliminary overview of general behaviour trends on mean travel distance. Subsequent to this analysis, we compare and contrast the commute pattern amongst males and females. This analysis is carried out at two levels namely at the aggregated level and at the disaggregated level, i.e., rural and urban. In addition, we compare the commute distance characteristics of urban and rural females. As the variable is not jointly distributed, we use the cosine similarity for this comparison.

The data set is collated at the household level, i.e., individual characteristics like age, level of education etc., are not collected. Therefore, for the analysis, we use household level socio-economic characteristics. This is in addition to the nature of the residential area and the tenure of the dwelling (Refer Table 1) for characteristics of the variables).

For setting-up the regression equation, we examine the cross-tabulated results between the socio-economic variables and commute trip distance. This analysis will help in understanding whether the socio-economic variables have an impact on commute distance.

Subsequent to this analysis we setup the regression equation. Though the data is ordinal in nature, the categories are exclusive, and hence, we use the stratified regression. Our regression equation takes the form

$$y_i = d_i + b_i x_i + b_i w_i + b_i z_i \tag{1}$$

where y_i is the commute distance category, d_i is the dummy which represents the area type namely urban or rural, x_i represents all the socio-economic variables including the characteristics of the dwelling and b_i is the estimator.

Table 1 Description of variables

Variable		Nature	Level
Dependent		Categorical	8
Predictor	Rural/Urban	Dummy	2
Predictor	Religion	Categorical (with Buddhism as the calibrating level)	8
	Social Group	Categorical (With Other Backward class as the calibrating level)	4
	Tenurial status	Categorical (employee quarter as the base level)	7
	Area of residence	Categorical (with denoted slum as the base level)	4
	Family size	Integer (with minimum value of 1)	
	Usual monthly consumer expenditure	Integer (proxy for income)	

4.1 Is There a Stark Difference Between Male and Female Commute Patterns?

As a primer to our analysis, we begin by examining the commute of men and women across the different distance buckets at an aggregate and disaggregate level, i.e., rural and urban (Refer Table 1). Additionally, we use descriptive statistics for understanding how far men and women commute.

A cursory examination of the commute pattern reveals that more than half the commute trips are for a distance of less than five kilometres and that at an aggregate level '1–5 kms' is the modal and median class for both men and women. At the disaggregated level we observe that for men this characteristic is preserved. Whereas for rural women, the modal class is '1–5 km', while the median class is '<1 km.' In contrast to this for urban women the modal class is 'No distance,' while the median interval is '<1 km.' This means that men and women have different commute patterns. For validating our claims we compute the cosine similarity between these. We propose this measure as the commute trips undertaken by men and women are not jointly distributed (Refer Table 2 for the cosine similarity).

In stark contrast to our earlier conjecture, we observe that the value of cosine similarity carries a positive sign and is greater than 0.7. Therefore, we deduce that there is not much difference between the commute patterns of men and women. The only plausible reason that can be forwarded for these patterns is the social roles that women undertake. Or alternatively, we can deduce that the family setting may be playing a role in determining the commute pattern. On contrasting the commute pattern of rural women against urban women (cosine similarity of 0.9229), we can hypothesize that urban and rural women have similar commute patterns.

4.2 Determinants of Commuting Patterns in Women

In the previous section, it was observed that men and women had statistically similar limits to commute patterns. Socio-economic and demographic characteristics of the households can be the only explanatory variables that can result in such an outcome. Therefore, before setting-up the regression equation, we look at the association of the different categorical variables namely rural/urban, religion, social group, area of residence and tenurial type with the maximum distance the female commutes (Refer Table 3, 4, 5, 6 and 7).

Based on the above table, we inferred that the participation rate of women is higher in rural areas (69.85%). Due to these higher participation rates, rural females are more mobile, i.e., more than six-tenths of the rural women have different work and residence. We observe an inflection point at '10–15 km' beyond which urban women travel more than rural women. Based on the Cramer v we conclude that there is a very weak association between the maximum distance commuted and the binary

Table 2 Distribution of commute trips over the different distance buckets (author's computation)

	No distance	<1 km	1–5 km	5–10 km	10–15 km	15–30 km	>30 km	Modal class	Median class (km)
Aggregate men	8.85	18.62	39.97	14.35	7.79	5.63	4.78	1–5 km	1–5
Aggregate women	34.42	21.26	34.53	5.19	2.26	1.47	0.88	1–5 km	1–5
Rural men	8.94	21.4	43.22	11.8	6.05	4.71	3.87	1–5 km	1–5
Rural women	31.76	23.68	37.55	3.95	1.44	1	0.64	1–5 km	< 1
Urban men	8.67	12.96	33.32	19.56	11.32	7.52	6.64	1–5 km	1–5
Urban women	43.15	13.36	24.63	9.26	4.94	3	1.66	No distance	<1

Table 3 Cosine similarity

	Aggregate men-aggregate women	Rural men-rural women	Urban men-urban women	Rural women-urban women
Cosine similarity	0.8434	0.8836	0.7025	0.9229

Table 4 Commuting patterns in rural and urban women

	No distance	<1 km	1–5 km	5–10 km	10–15 km	15–30 km	>30 km
Rural	23.29	16.88	24.87	2.85	0.97	0.58	0.41
Urban	14.41	4.37	6.75	2.29	1.15	0.69	0.49

Table 5 Maximum distance commuted by women across various religions

	No distance	<1 km	1–5 km	5–10 km	10–15 km	15–30 km	>30 km
Hinduism	27.44	17.07	25.45	3.88	1.63	0.99	0.73
Islam	4.94	1.35	1.59	0.35	0.1	0.08	0.08
Christianity	3.06	1.83	3.27	0.7	0.26	0.12	0.05
Sikhism	0.62	0.07	0.18	0.04	0.04	0.02	0.02
Jainism	0.13	0.03	0.04	0.02	0.01	0.01	0
Buddhism	0.69	0.35	0.48	0.08	0.02	0.01	0.01
Zoroastrianism	0	0	0.01	0	0	0	0
Others	0.82	0.55	0.6	0.08	0.05	0.04	0.01

Table 6 Maximum distance commuted by women across various castes

	Schedule tribe	Schedule caste	Other backward classes	Others
No distance	6.24	5.82	14.58	11.06
<1 km	5.38	3.82	8.35	3.7
1–5 km	7.89	6.18	12.56	4.99
5–10 km	0.98	0.91	2.02	1.23
10–15 km	0.34	0.41	0.8	0.57
15–30 km	0.12	0.28	0.46	0.42
>30 km	0.1	0.16	0.32	0.33

(rural/urban) of 0.2225. We observe that more than a third of the working women reside at the place of their work. Likewise, another thirty percent of the sampled women commute for up to five kilometres.

Table 7 Maximum distance commuted by women across various tenurial status of residence

	No distance	<1 km	1–5 km	5-10 km	10–15 km	15–30 km	>30 km
Owned/ freehold	33.21	19.07	28.52	4.23	1.73	1.06	0.72
lease hold	0.29	0.13	0.32	0.11	0.04	0.02	0.01
hired; employer quarter	0.76	0.33	0.21	0.06	0.04	0.01	0.02
Hired unit with written contract	0.56	0.18	0.43	0.16	0.1	0.06	0.04
Hired unit with no written contract	2.48	1.24	1.85	0.53	0.19	0.12	0.1
No dwelling	0	0.02	0.01	0	0	0	0
Others	0.4	0.29	0.27	0.05	0.03	0	0.01

In terms of religious constitution, Hindu women constitute more than three-fourths (77.19%) of the female workforce. This is followed by women believing in the faith of 'Christianity' (9.29%) and 'Islam' (8.49%). The other religious sects account for less than six percent of the female workforce. More than seven-tents (72.79%) of the women who reside at the place of their work follow the Hindu faith. Likewise, we find that the representative population that follows Zoroastrianism don't commute for more than five kilometres. This result may show a sampling bias. A very weak association between distance commuted and religion of 0.06 was observed.

From the preliminary scrutiny of Table 5, we can deduce that women belonging to the 'Other Backward Class' are ready to commute or conversely, we can presume that the female participation of households belonging to the 'Other Backward Class' is higher. In more than a third of the households, females work at the place of their residence. For understanding whether the maximum distance travelled by the female is impacted by the household's social group, we compute Cramer's V. A weak association (0.0995) is observed.

Though Naess in his seminal work shows that residential characteristics rarely affect commute patterns, we check whether the stated fact is true for Indian women. We compute the association between the maximum distance that the female travelled at the residential tenurial type and the area of the residence (Refer the association between residential tenurial type and maximum distance travelled by the female of the household and area of residence and the maximum distance that the female travels).

We find that more than four-fifths of the women travelling the highest distances for accessing employment live in owned/free-hold households and a third of the total sampled households have females who work and reside in the same place. This shows that fixed tenurial status is highly associated with wage employment. At an aggregate level, we find a very weak association (0.0443) between tenurial status and commute distances. This bolsters the claims of Naess [12]. To validate these claims we also compute the association between area type and limits to commute distance.

From the cursory scrutiny of Table 6, we may infer that the more formal the arrangement, i.e., legal status of an area, the propensity to commute increases. Even then, we observe that more than in third (37.72) of the sampled households the maximum distance that the female commuted was 'No distance' or these women resided at the place of their work. Negligible association of 0.02241 was observed between the area type and the limits to female commute. Based on the above associations, we may construe that other factors like family size, education status, income, etc. may be influencing the limits to commute necessitating a detailed analysis.

The commute distance is an effect of residential self-selection (RSS) [12]. Therefore, we consider each commute distance category as a discrete result arising from the interaction of socio-economic and demographic variables (Refer Table 8 for regression results). For the regression, we use rural, Buddhism, other backward classes, Employer rented and denoted slum as the zero value of the dummy represented by sector, religion, social group, tenure of residence and area type respectively. As income data was not available, we used the monthly expenditure as the proxy for monthly income

In the previous analysis, it was observed that rural and urban commute patterns are similar. Therefore, it could be inferred that the urban/rural dummy would become statistically insignificant. In contrast to this, we observe that the urban/rural dummy is statistically significant across all the seven levels. This difference may be attributed to the weight effect, i.e., the regression results are for the sample.

From the regression results, we infer that 'Religion' shows varying influence. Religion is statistically significant till the '1–5 km' beyond which the variable is insignificant. Further, this significance pertains to certain religions. This meant that the social construct is having an influence on the commute distance. Additionally, these results may suffer from sampling bias, e.g., according to the data 'Parsis' do not travel for more than five kilometres accessing jobs (Refer the cross tabulation of religion-distance (Table 4). Similarly, we observe that both tenure of residence and area of residence have a varying influence on the maximum distance travelled by the female for accessing employment. This result appears to be at variance to the results of the study conducted for Bangalore. Monthly expenditure or income appears to have a strong influence on the maximum distance commuted by the female. In contrast to this family size has a fluctuating influence on the maximum distance that the female commutes, even the signs fluctuate. This meant that education and the nature of employment may be playing an important role in the limits to female commute.

Table 8 .

	No distance	<1 km	1–5 km	5–10 km	10–15 km	15–30 km	>30 km
Notified slum	0.42	0.18	0.37	0.07	0.04	0.01	0.01
Non-notified Slum	0.26	0.16	0.29	0.06	0.03	0.04	0.01
Squatter settlement	0.13	0.1	0.07	0.02	0.01	0	0
Others	36.91	20.8	30.89	4.99	2.03	1.22	0.87

5 Conclusion

Both at an aggregate level and at a disaggregate level, i.e., urban and rural, men and women have statistically similar commute patterns. Additionally, urban and rural women too show very high similarity in their commute patterns. The only possible reasons that we can provide are the social functions that women undertake. This result is important for transport planners, as this shows that the special requirements of women like safety, security etc., can be incorporated in the comprehensive transport policy and a separate transport policy as entailed by feminist scholars is not necessary.

From these results, we may infer that the socio-economic and demographic variables may be playing an important role in determining the limits to commute. In contrast to these assumptions, we find that the household characteristics were weakly associated with the commute distance. These results corroborate the findings of Naess [12].

The regression results provide us an insight into this. The type of region, i.e., urban or rural has a positive impact on the limits to travel. Likewise, income too has an influence on limits to female commute. In contrast to this religion, social group, tenurial status of residence, area of residence and family size have a varying influence. This means that education, skill set that the female possesses and the nature of employment are likely to have an influence on the limits to female commute.

The present data set was collected at the household level therefore, details about the mode of commute, level of education that the female has attained etc. were not available. This may have had an influence on the model. As an extension to this study, we will investigate whether the limits to commute and income follow an inverted Kuznets curve. Additionally, we intend to separate wage employment from other kinds of employment and check the constancy of the relation.

Appendix

See Table 9.

Table 9 Results of regression

Dependent variable	Intercept	Sector_Urban	Religion_christian	Religion_hindu	Religion_islam	Religion_jain	Religion_other
No travel	−4.163e-01 (1.577e-01)**	7.258e-01 (2.749e-02)***	−4.222e-01 (8.802e-02)***	−5.536e-01 (8.180e-02)***	1.913e-01 (8.958e-02)*	−1.600e-01 (2.244e-01)	−3.520e-01 (1.072e-01)**
<1 km	−7.818e-01 (1.951e-01)***	−5.743e-01 (3.610e-02)***	−8.580e-02 (1.051e-01)	2.078e-01 (9.783e-02)*	−8.453e-02 (1.093e-01)	−1.935e-01 (3.392e-01)	2.911e-01 (1.240e-01)*
1–5 km	−8.698e-01 (1.825e-01)***	−5.908e-01 (3.104e-02)***	3.304e-01 (9.367e-02)***	3.233e-01 (8.805e-02)***	−2.859e-01 (9.909e-02)**	−8.655e-02 (2.861e-01)	5.141e-02 (1.156e-01)
5–10 km	−9.253e-06 (2.220e-06)***	4.139e-01 (5.529e-02)***	4.800e-01 (1.974e-01)*	−5.158e-02 (1.893e-01)	−3.087e-01 (2.089e-01)	−1.111e-01 (4.426e-01)	−2.722e-01 (2.595e-01)
10–15 km	−4.590e+00 (5.169e-01)***	7.076e-01 (8.255e-02)***	8.627e-01 (3.738e-01)*	4.461e-01 (3.625e-01)	−1.739e-01 (3.971e-01)	4.309e-01 (6.371e-01)	7.298e-01 (4.220e-01)
15–30 km	−6.205e+00 (7.745e-01)***	5.924e-01 (1.055e-01)***	7.590e-01 (4.807e-01)	2.358e-01 (4.574e-01)	−1.429e-01 (4.932e-01)	8.755e-01 (6.637e-01)	9.301e-01 (5.249e-01)
>30 km	−5.874e+00 (8.066e-01)***	7.346e-01 (1.246e-01)***	2.054e-01 (6.228e-01)	4.847e-01 (5.876e-01)	3.147e-01 (6.158e-01)	4.802e-01 (9.330e-01)	−3.745e-01 (8.211e-01)
Dependent variable	Religion_parsi	Religion_sikh	Group_others	Group_sc	Group_st	Tenurial_lease	Tenurial_ non-written
No travel	−1.012e+00 (8.537e-01)	5.754e-01 (1.322e-01)***	4.113e-01 (2.789e-02)***	−1.311e-01 (3.101e-02)	−2.864e-01 (3.394e-02)***	−6.372e-01 (1.429e-01)***	−6.705e-01 (9.478e-02)***
<1 km	−3.053e-01 (1.095e+00)	−9.924e-01 (2.195e-01)***	−1.639e-01 (3.525e-02)***	−1.209e-02 (3.535e-02)	2.498e-01 (3.614e-02)***	−8.746e-01 (1.818e-01)***	−1.687e-01 (1.123e-01)
1–5 km	8.327e-01 (7.811e-01)	−3.336e-01 (1.563e-01)*	−3.499e-01 (3.139e-02)***	6.899e-02 (3.079e-02)*	1.586e-01 (3.268e-02)***	8.607e-01 (1.626e-01)***	8.793e-01 (1.268e-01)***

(continued)

Table 9 (continued)

Dependent variable	Religion_parsi	Religion_sikh	Group_others	Group_sc	Group_st	Tenurial_lease	Tenurial_ non-written
5–10 km	1.086e+00 (1.108e+00)	−5.092e-01 (3.205e-01)	−1.172e-01 (6.074e-02)	1.784e-02 (6.571e-02)	−3.277e-01 (7.730e-02)***	1.230e+00 (2.587e-01)***	6.065e-01 (2.142e-01)**
10–15 km	−9.190e+00 (1.979e +02)	8.926e-01 (4.393e-01)*	−1.152e-01 (9.222e-02)	1.451e-01 (9.856e-02)	−3.793e-01 (1.245e-01)**	8.379e-01 (3.651e-01)*	1.359e-01 (2.884e-01)
10–15 km	−8.747e+00 (1.948e +02)	3.973e-01 (5.665e-01)	1.100e-01 (1.135e-01)	3.246e-01 (1.230e-01)**	−1.022e+00 (1.932e-01)***	1.171e+00 (5.363e-01)*	6.247e-01 (4.384e-01)
>30 km	−1.149e+01 (8.922e+02)	7.794e-01 (7.031e-01)	3.605e-01 (1.291e-01)**	1.435e-01 (1.568e-01)	−3.636e-01 (2.107e-01)	−3.770e-01 (6.731e-01)	4.858e-02 (3.732e-01)

Dependent variable	Tenurial_others	Tenurial_owned	Tenurial_written	Area_ not-denoted	Area_others	Area_squatter	Size	Monthly
No travel	−4.985e-01 (1.337e-01)***	−3.148e-01 (8.684e-02)***	−7.914e-01 (1.205e-01)***	−3.006e-01 (1.560e-01)	5.286e-01 (1.016e-01)***	8.867e-02 (2.069e-01)	3.033e-02 (5.848e-03)***	−1.838e-05 (1.937e-06)***
<1 km	1.244e-01 (1.491e-01)	−4.112e-01 (1.025e-01)***	−5.991e-01 (1.610e-01)***	6.231e-02 (1.904e-01)	−1.412e-01 (1.320e-01)	7.989e-01 (2.280e-01)***	2.307e-02 (6.937e-03)***	−1.318e-05 (2.652e-06)***
1–5 km	5.002e-01 (1.637e-01)**	6.824e-01 (1.201e-01)***	1.055e +00 (1.496e-01)***	−2.035e-03 (1.532e-01)	−5.981e-01 (1.040e-01)***	−6.122e-01 (2.349e-01)**	1.192e-03 (6.162e-03)	−9.253e-06 (2.220e-06)***
5–10 km	2.333e-01 (3.033e-01)	3.449e-01 (2.046e-01)	7.221e-01 (2.416e-01)**	1.568e-01 (2.888e-01)	1.385e-01 (1.944e-01)	−1.361e-01 (4.342e-01)	−1.859e-02 (1.196e-02)	2.608e-05 (2.832e-06)***
10–15 km	1.988e-01 (4.069e-01)	2.289e-01 (2.684e-01)	6.718e-01 (3.121e-01)*	5.199e-02 (3.736e-01)	−1.678e-01 (2.488e-01)	−2.409e-01 (5.655e-01)	−9.764e-02 (1.929e-02)***	3.732e-05 (3.472e-06)***
10–15 km	−6.409e-01 (8.247e-01)	6.708e-01 (4.174e-01)	9.954e-01 (4.646e-01)*	1.643e+00 (5.240e-01)**	6.118e-01 (4.572e-01)	−2.945e-01 (1.103e+00)	−9.131e-02 (2.389e-02)***	4.206e-05 (3.865e-06)***
>30 km	−2.299e-01 (6.100e-01)	−2.172e-01 (3.462e-01)	2.562e-01 (4.238e-01)	3.177e-02 (6.514e-01)	1.511e-01 (4.205e-01)	−1.220e+01 (2.031e+02)	2.751e-02 (2.462e-02)	2.253e-05 (4.952e-06)***

***99.99% Confidence interval
**99% Confidence interval
*95% Confidence interval

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Regularization of Micro-mobility Modes in an Emerging Economy: A Case of India

Shalini Kumari

Abstract The rise in growing traffic congestion, long urban commute durations, and lack of last and first-mile connectivity to public transport has led to citizens scouting for alternative modes to reach their desired destinations in the most economic, convenient, and time-saving manner. The concept of micro-mobility fits in these conditions to provide hassle-free, affordable, and easy-to-use solutions for the citizens across all demographics. Micro-mobility has emerged as a popular discourse in the world to address transportation and climate-change-related urban issues and challenges. While in other parts of the globe, micro-mobility is booming, in India, it is still in the early years of introducing micro-mobility. The outbreak of the COVID-19 pandemic has made the users explore these options of micro-mobility especially the young as a group as the risk of contracting the coronavirus is lower through micro-mobility. This paper is an attempt to critically review the existing operational characteristics of micro-mobility modes and also analyze the user's opinion towards this new concept of micro-mobility. This paper also tested various regularizing strategies to control the measures ranging from limiting the circulation speed, limiting the scale of operations, limiting the operation areas, and restricting it completely from a few public spaces to provide better safe conditions to the users as well as promote to have a better regulation process of micro-mobility in Indian cities.

Keywords Micromobility · Electric mobility · Regularization

1 Introduction

Micro-mobility can be defined by multiple criteria, comprising the speed of the vehicle, the weight of the vehicle, powertrain (electric or human-powered), etc. However, the most accurate definition of micro-mobility could be defined as any form of transportation which can occupy the urban spaces along with bicycles [1]. The basic concept is that the vehicles are smart, used for short distances, lightweight,

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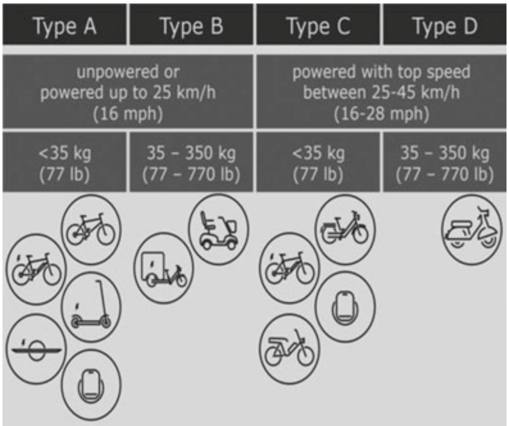

Fig. 1 Proposed micro mobility classification

low-speed, it doesn't have an Internal Combustion Engine, may not be electric, and are usually, single-person occupancy vehicles infused with intelligent and advanced technologies. Vehicles falling under the category of micro-mobility are typically allowed to use the bicycle infrastructure [2]. The powered micro-mobility vehicles can be defined through the three common characteristics: (a) Motorized-it can either be fully motorized or motor-assisted. Usually involving a battery-powered electric motor; (b) Small Size—a standard width is less than 3-feet, fitting within the standard sidewalk width or bicycle lane. (c) Low Speed: Most micro-mobility vehicles are designed to travel at the speed not more than ~30 kmph/20 miles/h, and are compatible with the sidewalk use (Fig. 1).

Due to enhancements in technology, these micro-mobility vehicles in only the last few years have been emerging as potential solutions for urban mobility. Globally, the dockless era technically began later in the 2000's which allowed users to unlock the bike with SMS through mobiles. Copenhagen in 2014 gave the world's first large-scale urban bike-sharing program, followed by Ofo and Mobike in China which deployed 20 times more bikes in 2016. In 2017, Santa Monica introduced Bird, a free-floating micro-mobility service of eco-conscious scooters [3]. A venture named Yulu has brought this trend of micro-mobility in India in 2017.

2 India Towards Electric Transformation

Electric mobility in India is seen as a transformational strategy towards sustainable urban mobility and thus, requires high investments and would require long-term collaborative efforts. 18% of the total energy consumption in India comes from the transport sector. Based on the current trends of energy consumption the demand for this sector by the year 2030, would require an estimated 200 million tonnes of oil equivalent (MTOE) of energy supply annually. Moreover, the sector also contributes

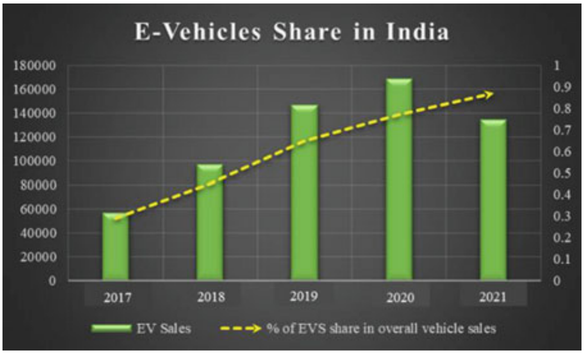

Fig. 2 E-vehicles share in overall vehicles sales in India

an estimated 142 million tonnes of CO_2 emissions annually, out of which 123 million tonnes is contributed by the road transport segment alone [4].

To keep the climate goals, if coupled with the decarburization of the power sector, electric vehicles would also provide major contributions. Keeping in view the climate change commitments made by the Government of India during the COP21 Summit held at Paris to reduce the emission intensity by 33–35% from 2005 levels, it is pertinent to introduce alternative means of transport which can be coupled with India's rapid economic growth, rising urbanization, travel demand and country's energy security [4]. Electric mobility when packaged with innovative pricing solutions, support infrastructure, and appropriate technology presents as a viable alternative in addressing these challenges.

Currently, India may not seem like a logical market for e-scooters and micro-mobility, but it is worth considering. A small percentage of the country's population uses micro-mobility transport, the country will nonetheless be one of the largest micro-mobility markets in the world. Looking at the statistics of Electric Vehicles in the country, a total of ~6 Lakh EVs were sold in India over the past 5 years.

The current penetration level of Electric Vehicles in the Indian market is very low viz. <1% when compared to non-EVs but comparatively, the share has increased over the time period of the past 5 years [5]. A clear increase can be observed in the percentage share of electric vehicles to the overall vehicles registered in India as shown in Fig. 2. The light vehicles such as two-wheelers segment in EV will grow in India as it is the most affordable vehicle in developing nations. Since 2017, the share increased from 0.29 to 0.87% in 2021 [6]. Similar to e-mobility, the demand for shared mobility is also expected to increase in the near future.

3 States' Government Promoting EV Policies

Governments play an essential role in supporting the mainstreaming of EV technology in the motor vehicle sector. To reduce the import of oils and, most importantly, to reduce pollution levels in the country, the Government of India is taking several

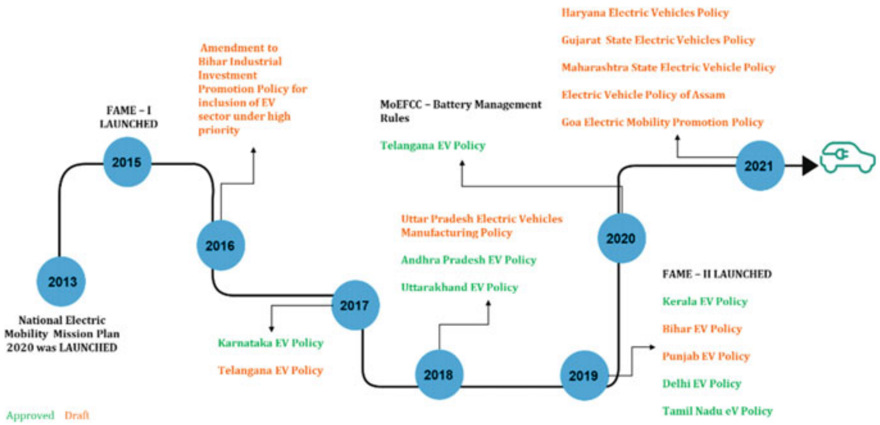

Fig. 3 Electric vehicle policy timeline of Indian States

initiatives in the form of policies to promote and adopt the use of electric vehicles. A timeline is shown below of the development of EV policies in the states of India, highlighting their activity on electric mobility in the past few years [7] (Fig. 3).

Thenceforth, 15 States and Union territories in India have published draft/notified electric vehicle policies as of December 2021. These policies vary widely in their scope and scale together with the fiscal, non-fiscal, and other incentives.

In implementing the EV policies, these states and UTs share a few similar visions and aim to become e-mobility hubs within the country. One set of objectives focuses on making states preferred destinations for EV manufacturing. The second set of objectives aims to increase the uptake of EVs within the states, for benefits such as reducing pollution and a transition to sustainable mobility.

4 The Micro-mobility Revolution in India

The disruption created by COVID-19, however, has significantly changed people's perception of walking and biking, leading many decision-makers to rethink the role of active transport. In fact, several major cities have already seized that momentum to advance their urban sustainability agenda. India too has put in place a nationwide programme known as the Cycles4Change Challenge programme which has gained immense popularity both by the users as well as the government.

According to the AICMA, 41 Lakhs bicycles were sold in the country in the five months from May to September 2020 indicating an increase of 100% [8]. The coronavirus pandemic crisis has made people aware of their health and immunity and at the same time, people have become conscious about social distancing too resulting in higher purchase and use of active modes such as bicycles and other micro-mobility options.

During the pandemic period, people have become more aware of micro-mobility services available to them in their communities. Despite some infrastructure-related challenges, users are now acknowledging the Indian micro-mobility start-ups like Bounce, Zypp, Chartered bike, and Yulu. These are now somewhere shaping the urban mobility in the country and also progress towards broader environmental and socio-economic goals (Table 1).

Presently, these services of micro-mobility are only offered in limited cities (15 cities). This popular technology-driven platform for MMVs enables Integrated Urban Mobility across public and private modes of transport in a seamless, shared, and sustainable manner. This first and last-mile connectivity service requiring no vehicle registration or driving license uses two-wheelers (motorized/non-motorized) to reduce the congestion of transport and air pollution in urban India, redefining urban mobility across the country.

Table 1 Operational characteristics of micro-mobility services in India

	Bounce	Yulu	Mobycy/Zypp	Chartered bike
Launch year	2014	2017	2017	2017
				
Fleet size	~6,000	~18,000	~1,000	~4500
Maximum speed (kmph)	45–50	25	25	25
Ride mileage (kms)	60	50–55	20–35	30
Operating cities	Bangalore, Vijayawada, Hyderabad, Jaipur	Delhi, Ahmedabad, Mumbai, Pune, Bhuvaneshwar,	Gurgaon, Delhi-NCR, Hyderabad	Bhopal, Ranchi, Surat, Kolkata and Prayagraj
Charges	Rs. 6/km	Rs. 10/30 min	Rs. 2/min	Rs. 5–6/30 min
Purpose	Commute; recreational	Commute; recreational	Commute; recreational; any shipment delivery	Commute; recreational

5 Need of Regularization for Micro-mobility in India

In order to reduce the effects of climate change and pollution on the Indian road network, the Indian Government is looking at various solutions, and micro-mobility would be an essential part of these initiatives. A regulated nation can help small distance mobility services prosper.

The Cycles4Change Challenge Programme launched in 2020 by the Ministry of Housing and Urban Affairs, GoI has inspired over 100 cities already in just a year. To make the cities cycle friendly, these 100 cities have identified over 3500 km and 400 km of neighbourhood streets and main roads respectively. As per the survey conducted by ITDP India Programme in 2020, 95% of people across India said they would shift to cycling if it becomes safe and convenient. The top 11 cities among these 100 cities have been awarded India's Cycling Pioneers and have entered the second stage of the programme to implement and scale up cycling or other public bike-sharing options. 5 of the 11 cities in this list already have micro-mobility services, including Bangalore, Surat, New Town Kolkata, Vadodara, and Bhuvaneshwar.

India's current legislative issue is not directly associated with the regulation of e-scooter use since it is relatively unregulated currently. Both the Motor Vehicle Act, 2019 and the National Motor Transportation Act, 2016 do not define vehicles that weigh less than 350 kg or have a design speed of 25–50 kmph. 33.2% of all road fatalities in India occur due to two-wheeled vehicles and, evidently, this is not promising for e-scooters/e-bikes or any other 2 W micro-mobility vehicle. Micro-mobility adoption in India is arguably hindered more by this issue than by direct legislation (Table 2).

The table above highlights the category-wise number of road accidents in India for the years 2017–2018–2019. Based on the data, it can be observed that though the

Table 2 Road accidents in India

Road user category wise road accidents in India			
Years	FY2017	FY2018	FY2019
Pedestrian	13.8	15	14.4
Bicycles	2.4	2.4	3.1
2-Wheeler*	33	36.5	33.2
Other motor vehicles (including e-rickshaw)	7.7	7.3	1.4
Slow moving vehicles (cycle rickshaws, hand carts, animals drawn vehicles etc.)	2.4	2.4	8.1
Others (auto, cars/taxis/vans/LMVs/trucks/lorries)	40.7	36.4	39.8
Total (A + B + C + D + E)	59.3	63.6	60.2
High speed vehicles (F)	40.7	36.4	39.8
Grand total	100	100	100

* 2-wheeler include motorcycles, scooters, mopeds

overall number of accidents has drastically decreased in India, the share of modes that are bicycle-friendly still accounts for more than half of the accidents.

Table 3 outlines the global regulatory situation for micro-mobility at a global level [9]. The Indian cities are far behind other urban areas throughout the world that have regulated variants of micro-mobility parameters.

In order to understand the public's opinion towards this new concept of micro-mobility, an online survey was offered via Google Forms during the last few months of 2021. The survey was conducted in different cities of India, including Delhi, Gurgaon, Noida, Hyderabad, Bangalore, and New Town Kolkata in which a total of 150 sample surveys have been collected. More than half of the responses received from these cities are from youth aged between 17 and 25. While none of the respondents own an E-Vehicle, 34.1% have taken advantage of micro-mobility services in their cities (Figs. 4, 5 and 6).

Available data suggest that the shared E-scooters and bikes are explicitly suitable for short trips in urban areas that may range from 0 to 5 km and thereby help to reduce traffic congestion and decrease vehicle emissions. The typical MMV user or bike-share pass-holder rides for 30 min to 1 h and 2–5 km (51% of the users) on an average trip in major cities across the Indian States. The trips are typically for leisure activities. E-bikes are often used for relatively long journeys or for product shipment purposes where walking is not a viable option. According to the CEEW [10] study, urban dwellers in Indian cities make 39% of their trips within 0–5 km and 31% within 5–10 km. In spite of the fact that MMVs are considered to be a convenient mode of transport, the lack of infrastructure and safety is yet a concern. Micro-mobility has been found to substitute for walking, cycling, and public transport trips. With no emissions, zero fuel consumption, and smaller road space that motorized vehicles occupy, these are the greener trips. A regularized micro-mobility segment can be crucial to overcoming such challenges as lack of infrastructure and safety concerns, as well as allowing MMVs to serve as a mode of transport that is utilitarian rather than recreational.

6 Results and Discussion

This study made an attempt to understand the user preferences using Analytical Hierarchy Process (AHP) to estimate the user weightages towards identified regulatory measures. From the data analysis, it has been observed that the user's preference to choose micro-mobility modes relies on the key indicators such as safety, infrastructure availability, driving behaviour, and dedicated lanes. To assess the user preference on the above stated four indicators, an AHP analysis has been carried out, using seven scale [11]. For the AHP, 150 samples were collected. From the AHP analysis, it has been observed that the users have almost an equal importance towards safety and dedicated lanes with 30% weightage and 27% for infrastructure availability. The table below shows the results of AHP along with a statistical check of the model using consistency ratio which is under acceptable range.

Table 3 Regulation of MMVs around the globe

Place	Year of Approval	Regulation Parameters									
		Speed Limit	Age Limit	No. of Operators	Scale of Operation	Types of Micro Modes	Safety	Fleet size	Parking	License /Permits	Hours of Operation
Germany	2019	20kmph	>14								
Sweden	2019	20kmph									
Denmark	2019	20kmph									
France	2019	25kmph	>12								
Italy	2020	20kmph	>14								
Belgium	2020	25kmph									
Spain	2020	25kmph									
UK	2021	25kmph									
Brussels	2021	20kmph									
Austin	2019	32kmph									
Chicago	2019	25kmph									
Denver	2019										
Los Angeles	2019	25kmph									
Oakland	2019	25kmph									
Santa Monica	2019	25-32kmph									
Washington DC	2019	16-32kmph									
India	-										

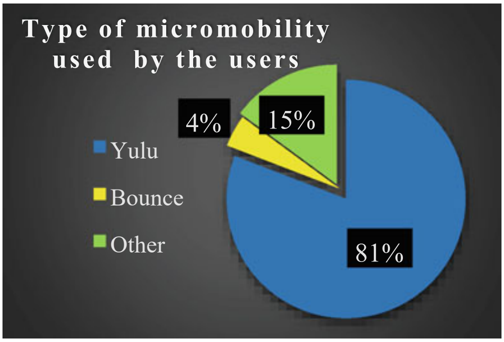

Fig. 4 Type of micro-mobility service used by the users

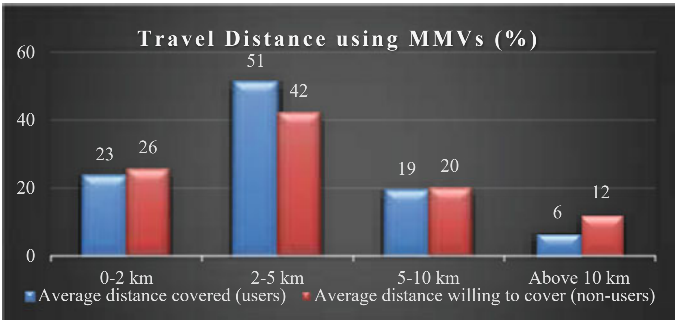

Fig. 5 MMVs usage rating by the users

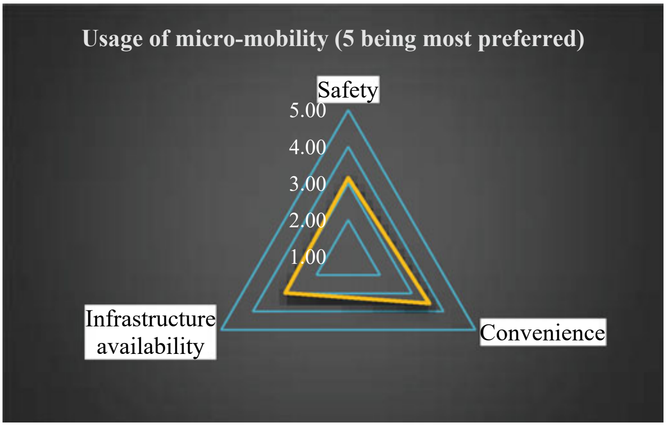

Fig. 6 Average distance travel by micro-mobility vehicles in India

Indicators	Criteria weights
Safety	0.30
Infrastructure availability	0.27
License	0.14
Dedicated lanes	0.30

7 Strategies for Regularizing Micro-mobility in India

The regularization of micro-mobility in India will prepare the ground for the provision of urban spaces for PBS and micro-mobility vehicles, for instance, parking at metro stations, separate pavement, charging infrastructure, grants through Viable Gap Funding to attract private sector players to invest and support, prevention from irresponsible usage and exploitation of the MMVs. In addition, regulating micro-mobility will also contribute to the mindset shift of people who can then view MMVs as a primary source of transport for their day-to-day mobility. Below are some strategies which may be applied by the GoI in order to regularize the micro-mobility:

1. *Strategy 1*—Create Zones: Micro-mobility shall be available within the mega residential and integrated townships. Similarly, for neighbourhood communities, commercial zones, institutional zones, etc.
2. *Strategy 2*—Create and maintain infrastructure: Define micro-mobility zones to eliminate or minimize the barriers to MMV Users. Define the operational areas for MMVs after consulting with the operators.
3. *Strategy 3*—Data sharing: For the purpose of ensuring compliance with licensing requirements.
4. *Strategy 4*—Fleet size: There should be an assessment of the minimum and maximum fleet size needed for a service to be viable and profitable.
5. *Strategy 5*—Manage vehicle speeds to support and encourage the use of MMVs
6. *Strategy 6*—Allocate and design right-of-way that also supports the Street Design Principles: Develop and update standards, guidelines, and policies.
7. *Strategy 7*—Fund new programs to promote micro-mobility: Encourage awareness and participation in these programs. Engage and educate critical stakeholders on the benefits of shared, electric mobility.

8 Conclusion

This urban mobility alternative is still in its early stages and needs to align with government frameworks for achieving global sustainability goals, sustainable mobility, equitable access, and integrated transport. Electric mobility in India is seen as a transformational strategy towards sustainable urban mobility and thus, requires investments and long-term collaborative efforts. India has a relatively low penetration of electric vehicles, at less than 1% of the total number of vehicles, but an increase has been observed, which in turn may boost the use of micro-mobility vehicles.

It is possible to transition to low-carbon urban mobility with the use of MMVs. For shared micro-mobility to be fully realized, the government must first make the roads safer, create a reliable network of places for riding, and create a bicycle-friendly infrastructure and regulations. By regulating micro-mobility, the government can realize net welfare benefits while at the same time limiting any negative impacts on society and individual users.

There are very few cities that have enacted micro-mobility regulations including Santa Monica, Chicago, and Washington. When micro-mobility is well integrated with public transport, it can significantly contribute to last-mile connectivity and mobility policy at large. The Indian government needs to issue directives to standardize regulations and deal with micro-mobility as an efficient transportation system.

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Joint Mode and Shipment Size Choice for Interregional Transportation of Fruits and Vegetables in India

K. A. Gayathri, V. Ansu, and M. V. L. R. Anjaneyulu

Abstract Freight transportation is the backbone of a nation's economy. The mode of transport chosen is a significant logistical decision influenced by the size of the shipment. The shipment size and mode of transport are both logistics decisions that must be made simultaneously. However, studies that combine both of these logistic choices are limited. There have been no such studies in India, where the characteristics of the transport mode, shipments, and traffic are vastly different. Also, freight transport modelling studies are rare in India because disaggregate shipment data is unavailable, and data collection is costly. This study develops a joint transport mode and shipment size choice model for the interregional transportation of fruits and vegetables using the logit model. Shipment weight was divided into discrete classes using cluster analysis and combined with the mode of transport as discrete choices in modelling. The data was gathered through an extensive revealed preference survey conducted among shippers in Kerala, India's southern state. The study revealed that the shipment size affects the transport mode choice. As shipment size increases, the choice of rail is significantly increased by the increase in shipping frequency. The speed, shipment frequency, and the number of trucks were significant in joint choice modelling. Rail mode share could be increased to reduce energy consumption by improving the speed and increasing the shipment frequency.

Keywords Joint choice model · Freight transport · Cluster analysis

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1 Introduction

A nation's economic development heavily depends on a reliable and smooth supply of goods. As the goods transported increase in a country, trade and industrial activities also increase. In India, freight transport modelling lags because disaggregate shipment data is challenging to obtain, and data collection is expensive. Mode choice modelling identifies significant variables to plan strategies for reducing fuel consumption. In contrast to passenger transport, the shipper chooses the mode of transportation for freight. Freight mode choice modelling is challenging due to differences in the characteristics of the commodities, shippers, and distance.

Road and rail are India's primary modes of interregional freight transport, although other modes like airways, inland water transport, and shipping along the coast are also available. Road and rail account for 91% of total interregional freight traffic by weight, with road accounting for 61% and rail accounting for 30% [15].

Freight mode and shipment size are integrated logistics decisions, but most studies looks at them separately. However, it is seen that a close relationship exists between these two logistics decisions. Shippers select the mode of transport depending on the shipment size. These two choices go hand in hand, but simultaneous choice models are not often developed. As a result, this study attempts to construct a joint choice model of the freight mode and shipment size.

Discrete choice models estimate the probability of choosing a discrete alternative. For mode choice modelling, discrete choice models like logit or probit models are generally used. Ghareib [6] opposes using complex models such as probit for studying binary mode choice situations since the logit model makes more accurate predictions. Wang et al. [22] compared logit and probit modelling techniques for freight mode selection and concluded that the results of both models are similar. The logit models are commonly employed for freight mode choice modelling [4, 7–10, 13, 18, 19, 23]. As a result, logit modelling is used in this research for joint choice modelling of mode and shipment weight. The continuous shipment weight is classified into discrete classes using cluster analysis, combined with the mode of transport, to form discrete choices for joint modelling of transport mode and shipment weight.

This study is done to derive the methods to improve the effectiveness of interregional transportation of fruits and vegetables in India. India's annual interregional freight traffic of fruits and vegetables by road and rail is 36.5 and 3.1 billion tkm, respectively [15]. The research focuses on shippers' mode and cargo weight joint choice for transporting fruits and vegetables.

2 Literature Review

Many studies have modelled the mode choice of freight transport [1–3, 12, 14, 17, 21, 22]. However, studies on the joint modelling of the transport mode and shipment size are few. de Jong and Ben-Akiva [5] developed a joint transport mode and shipment size model employing multinomial logit modelling. The shipment weight was classified into different categories and combined with transport modes to form discrete combinations of transport mode and size of the shipment. Pourabdollahi et al. [16] modelled mode choice and shipment size after classifying the shipment weight into discrete groups. The joint distribution was created by using the copula function to analyse how the two choices interacted with one another. Stinson et al. [20] developed a joint mode and shipment size choice model by constructing nesting structures of the mode and shipment size categories. Keya et al. [11] developed a joint model of transport mode and shipment size using a closed-form copula-based framework considering six different copulas, and the most suitable copula model was determined.

However, no such studies of joint modelling of the mode of transport and size of shipment are available in India, where the vehicle characteristics and shipment sizes are entirely different from other countries. As a result, this study was carried out to comprehend the joint selection of transport modes and shipment sizes in India.

3 Methodology

The study is limited to India's interregional freight transport, excluding urban deliveries. Only road and rail are considered in this study, as they account for 91% of interregional freight transportation in India. Rail transport also includes a small portion of road transport at both ends of the journey to provide first/last mile connectivity and is referred to as rail for simplicity. The simultaneous selection of transport mode and shipment weight is considered the decision-making process.

A revealed preference survey was employed to collect data from shippers through a face-to-face interview. Data were collected from the shippers of fruits and vegetables in major districts of Kerala, India. The preliminary data analysis was done to understand the trends in data.

The continuous variable, shipment weight, is divided into discrete classes using cluster analysis. These discrete classes of shipment size are combined with the modes of transportation to obtain the joint selection of transport mode and shipment sizes for choice modelling. A multinomial logit model is developed as the study considers road and rail and the discrete shipment size classes constituting multiple combinations of choices.

4 Data Summary

Table 1 depicts the data summary of the continuous variables. Analysis of discrete variables found that 75% of road shippers and 68% of rail shippers believed their mode of transport was safe. 99% of road shippers believed that the transport mode was flexible. On the other hand, shippers disclosed that rail transport is not flexible.

Table 1 Data summary

Observed variable	Minimum	Maximum	Mean	Std. deviation
Shipment weight (tonnes)	0.02	25.70	8.19	6
Transportation cost (INR)	40	250000	31832	45819
Handling charges (INR)	15	14000	4234	3088
Cost of loss (INR)	0	55000	1331	5137
Transportation time (h)	2	336	46	52.9
Distance (km)	77	3754	1001	967
The capacity of mode (tonnes)	1.50	40	15.2	10.8
Shipment value (INR)	600	3855000	332971	428792
Number of trucks	0	36	2	6.4
Number of employees	0	30	6.2	5.8
Age of shipping firm (years)	1	100	19.7	13.1
Shelf life (days)	2	30	7	5
Shipment frequency per month	1	30	8.6	9

Shippers believe that road transport has a higher overall service quality, tracking capability, and reliability than rail.

5 Cluster Analysis

The K-means clustering method was adopted for clustering shipment weight, and the best number of clusters was selected, taking care that a sufficient number of observations were present in each cluster for modelling. Three clusters were chosen. Table 2 presents the number of responses in each cluster. In total, there were 601 responses in the data. Shipment weights up to 5 tonnes (small) formed the first cluster, weights above five to 12 tonnes (medium) formed the second cluster, and shipments above 12 tonnes (large) formed the third cluster. These classes of shipment sizes were combined with the mode of transportation and treated as discrete choices in joint choice modelling.

Table 2 Number of cases in each cluster

Cluster number	Shipment weight, t	Shipment size	Number of observations
1	0 to 5	Small	214
2	Above 5–12	Medium	258
3	Above 12	Large	129

6 Mode and Shipment Weight Choice Modelling

Multinomial logit models were developed for the combined transport mode and shipment weight modelling. The road and rail were combined with discrete classes of shipment weight obtained from cluster analysis to form the joint choices. The dependent variable is the combination of freight transport mode and shipment weight. The data collected shows that all the rail shipments of fruits and vegetables fall into the first cluster of low shipment sizes. This study does not consider the whole train load of a single shipment, as there won't be a mode choice. Hence, there were four choices; road with the three classes of shipment sizes and rail with small shipment size referred to as rail in further description. Rail is taken as the reference category. The confidence interval is taken as 95%. Significant variables have a significance value of less than 0.05. Variables were introduced one after the other to eliminate insignificant variables and to find the best model. The model's Chi-square value is 509.7, which is significant and suggests that the dependent and independent variables have a strong association. The Nagelkerke R^2 value obtained for the model is 0.618, and the prediction accuracy is 61.6%.

Table 3 presents the joint choice model developed. The choices considered are road with lower shipment weights (Road-Small), road with medium shipment weights (Road-Medium), road with higher shipment weights (Road-Large) and rail. Coefficient B is the parameter used for analysing the influence of each variable on the joint choice decision of mode and shipment size. If coefficient B is positive, the modelled joint choice is more likely to happen with an increase in the value of the variable. If the coefficient is negative, the likelihood of modelled choice reduces with an increase in the value of the variable.

The significant variables are speed, shipment frequency, and the number of trucks. The coefficient for speed and shipment frequency is negative and positive for the number of trucks. Hence, increasing speed and shipment frequency increases the mode selection of rail. It is also found that with an increase in shipment size, increasing shipment frequency increases rail choice more significantly. That is, shipment frequency has a more significant effect on mode choice for higher shipment weights. These mode choice variations depending on the shipment sizes, can be made only by developing joint choice models.

7 Conclusions

Developing joint models for mode and shipment weight is an exciting step in freight transport studies. The mode of transportation and cargo weight are the two most interconnected logistics decisions, but they are typically addressed separately. This paper discussed the necessity of studying these two logistics decisions simultaneously and examined the factors influencing the combined choice decision.

Table 3 Joint mode and shipment weight choice model (reference category: rail)

Choice	Variable	B	Std error	Wald statistic	Sig.	Exp (B)
Road-small	Intercept	12.926	1.907	45.954	0.000	
	Speed	−0.319	0.042	56.857	0.000	0.727
	Shipment frequency	−0.086	0.031	7.504	0.000	0.918
	Number of trucks	0.617	0.253	5.941	0.000	1.853
Road-medium	Intercept	13.610	1.904	51.067	0.000	
	Speed	−0.281	0.041	45.809	0.000	0.755
	Shipment frequency	−0.204	0.032	39.225	0.000	0.816
	Number of trucks	0.679	0.253	7.231	0.000	1.972
Road-large	Intercept	13.205	1.916	47.512	0.000	
	Speed	−0.267	0.042	40.718	0.000	0.765
	Shipment frequency	−0.336	0.046	53.653	0.000	0.715
	Number of trucks	0.664	0.253	6.908	0.000	1.943

The shipment weight, a continuous variable, is divided into discrete classes using cluster analysis. Three clusters of shipment sizes were combined with the mode of transport chosen to form discrete joint choices of transport mode and shipment weight. The modes of transportation considered are road and rail, as these modes take 91% of interregional freight in India. Four alternatives were considered for the choice modelling: the road with the three classes of shipment size and small shipments by rail.

The speed, shipment frequency, and the number of trucks were significant when the joint choice of the mode of transport and shipment sizes were considered. The model results revealed that increasing transportation speed and shipment frequency could increase the rail mode share. Hence, for transporting commodities like fruits and vegetables, implementing policies that increase the speed and shipment frequency can increase the mode share of rail compared to road, eventually decreasing the energy consumption and pollution associated with freight transportation. The increase in shipment frequency increases rail choice significantly as shipment size increases. These variations of the effects of the factors on mode choice depending on the shipment sizes can be found only by developing joint choice models. In further studies, similar models can be developed for other commodities to identify the factors influencing the joint choice decision.

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Model for Estimating On-Street Night Parking Demand: A Case Study at Roorkee

Ashwani Bokadia, Mokaddes Ali Ahmed, and Prasenjit Das

Abstract Nowadays parking is a challenging issue all around the world, especially in the central business district (CBD) during the day and residential areas during night. Roorkee is one of the cities in India that has parking issues. Inadequate off-street parking and a tendency to park vehicles close to the destination result in high parking demand. The gradual increase in vehicle ownership per family is also the reason for increase in parking demand. These causes lead to a reduction in main carriageway width, a fall in flow speed, and unnecessary congestion and delay. The parking demand is also increasing in residential areas during night time. Urbanites do not have sufficient parking space in their possession and they park their vehicle on street side during the night. To control parking, a proper parking management policy should be adopted for the on-street night parking demand. In this study, one residential area-Ramnagar, Roorkee has been selected as the case study area. To estimate parking demand, a parking demand model is developed. Use variables such as vehicle ownership, the purpose of the trip and the presence of backyard garage are used for generating the demand model. The data were collected from in-out and questionnaire surveys. The linear regression analysis was done for the parking demand model using the statistical package for social sciences (SPSS). The R square value was 0.622, found out by Regression analysis. And the sensitivity analysis was carried out to investigate the effect of independent variables on parking demand.

Keywords On-street night parking · Accumulation profile · Parking duration · Parking demand · SPSS · Sensitivity analysis

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1 Introduction

The gradual increase in the vehicular ownership and non-availability of parking space in buildings and apartments is increasing the on-street night car parking demand in residential areas and is bringing the parking system to a critical stage [3]. It has been observed that in metro cities like Delhi, Kolkata and, Chennai car owners park their vehicles on-street during night time. Roorkee is one of the cities that is also going through the same parking problem in residential areas. Free night parking users are using government property (road/land) without paying a single penny. If the on-street night car parking demand is not controlled with immediate effect, it might reach an irreversible position. Hence, a systematic study of parking characteristics, parking demand, and controlling measures is very important for a town planner. This study is undertaken for the improvement safety of both people and their vehicles. Through this research, the main factors affecting parking demand have been identified. The objectives of the studies are to study the present parking condition in residential areas, to develop a night parking demand model, to carry out a sensitivity analysis to obtain the most sensitive parameter for demand estimation and to obtain parking different parking characteristics.

2 Literature Review

Wong et al. [11] has made the assumption that each land-use variable's corresponding parking activity has a distinct parking accumulation profile. These profiles were determined using surveys. Models of parking demand have been created in Hong Kong using the findings of these surveys. Tong et al. [10] developed a cluster analysis-based approach for building profiles of aggregate parking buildup at parking lots to improve the effectiveness of survey data gathered. According to the authors, accumulation profiles are possible to support transportation decisions of professionals -making and support models of parking demand. These profiles may also make it easier to evaluate various traffic management techniques and construct real-time parking information systems. Ottosson et al. [9] used automated transaction data from Seattle-area parking pay stations to analyse how parking charges affected statistics, including Parking turnover rate, duration of parking, and parking accumulation. Chen et al. [5] examined parking characteristics for various land uses and parking facilities in Shanghai which include parking indices, saturation, peak-parking ratio, turnover rate, and parking time are all factors to consider. Abbott et al. [1] have hypothesized that by adding On-street parking congestion might be considerably reduced by adding a modest number of off-street stalls from chosen residential buildings to the residential parking programme. Alemia et al. [2] examined the parking search distance model, it was discovered that the "day of the week" variable was significant. Das et al. [6], Dave et al. [7], give a fundamental understanding of how demand for on-street parking is assessed using various data collection intervals, and can subsequently assist policy-makers and planners in creating on-street parking demand management strategies [6, 7]. Zheng et al. [12], Hamad et al. [8] the parking prediction interval determination method is examined in this research. Using the variable parking prediction interval, a parking demand prediction model was developed. Presented taking into account vehicle arrival and exit patterns in the parking lot, and numerical tests were conducted under various circumstances.

3 Study Area

Ramnagar is located in Roorkee tehsil of Haridwar district in Uttarakhand, India shown in Fig. 1. The latitude and longitude of the city are 29.8543 °N, 77.8880 °E and it is situated 20 km away from sub-district headquarters Roorkee and 49 km away from district headquarters Hardwar. Ramnagar is also a residential area of Roorkee City and is under the municipal corporation of Roorkee shown in Fig. 2. In this residential area, most of the people are having own cars and most of the car owners park their vehicles near buildings or apartments at night time on the street. Due to on-street night parking sometimes people face many types of problems like congestion, accident, delay, etc. in this area during the night.

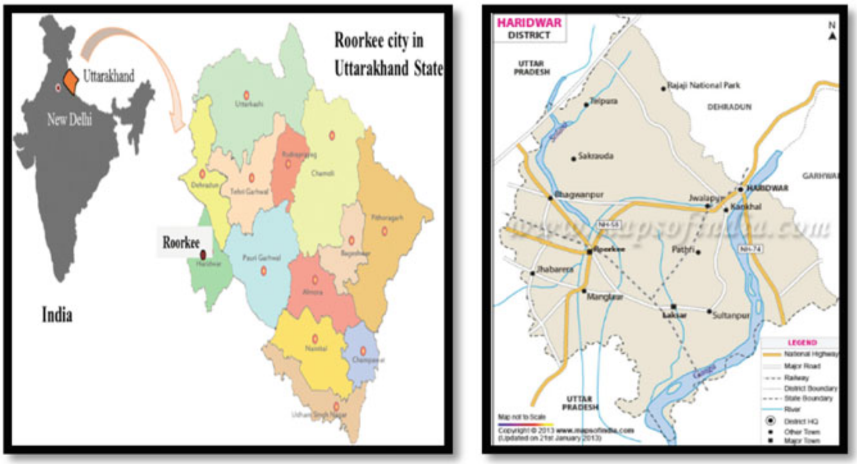

Fig. 1 Location map of Roorkee city. *Source* Google site, maps of India

Fig. 2 Map for Ramnagar, Roorkee

4 Methodology

Two types of surveys have been conducted for this study i.e. in–out and questionnaire surveys and the overall methodology is shown in Fig. 3.

To gather data, a survey of accumulation during peak hours from 1 to 3 Am night time was conducted. The parking demand model is created by analyzing the data gathered from this survey, which offers information about the total parking accumulation within the survey area at any given time.. Parameters like age, gender, family income, family size, number of vehicles parked at night time, garages presence in the backyard, distance between origin and destination, and the average duration of parking (in hours) parking fee, parking duration, search time, walking time, walking distance, waiting time, travel time, travel cost, search time, parking demand, trip generation, access time, vehicular and population growth is important in estimating parking demand. To create the model, all parameters are initially evaluated for the investigation. A regression analysis is carried out by IBM SPSS Statistics 26. The variables which have less significant are eliminated. Any backyard garage presence, the purpose of your trip and vehicle ownership is found to be significant. The regression equation is represented in its general form below by including the aforementioned parameters.

The demand equation is given by the following equation (Eq. 1).

$$Y = a_0 + a_1X_1 + a_2X_2 + a_3X_3 \tag{1}$$

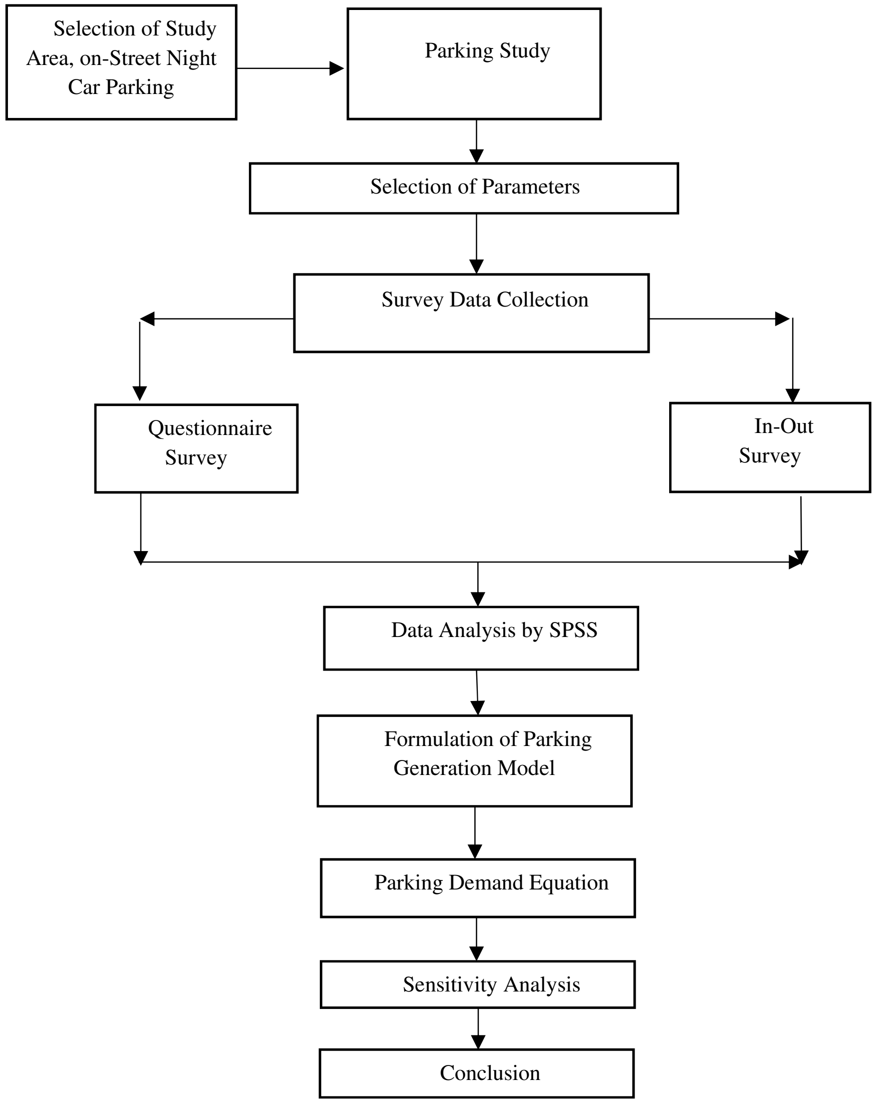

Fig. 3 Methodology flow chart

where,

Y Number of cars parked at night time.
X_1 Any backyard garage presence.
X_2 What is the purpose of your trip.
X_3 Vehicle ownership.

5 Survey and Data Collection

5.1 In–Out Survey

In which the number of cars entering the parking lot within a specific time frame is counted and the number of cars leaving the parking lot is also recorded. In this study, peak hours and parking accumulation are calculated from this survey. The number of vehicles entering the parking area within a specific period of time is then counted and also recorded how many cars exit the parking area in this study in–out survey was carried out overnight between the hours of 10 p.m. and 9 a.m. at Ramnagar to certain the peak parking accumulation. To finish the survey, manual counting was done [4].

5.2 Questionnaire Survey

A questionnaire survey has been conducted to know the personal information of the respondents, trip characteristics, and parking characteristics. A total of 104 sample size has been taken for this study. Personal information includes age, gender, family size, family income, vehicle ownership, presence of a backyard garage, and the number of cars parked at night time. Similarly, trip purpose, trip duration, frequency of visit, parking duration, average parking fee, search time, time to walk, and safety level belong to trip and parking characteristics. The data are extracted to develop the parking demand equation (Eq. 1) using linear regression analysis in SPSS.

Figure 5 shows the Purpose of your trip which explains that 64% of people go for a working trip, 18% for a business trip, 14% for an educational trip, 3% for shopping, and 1% people go for another trip and Fig. 6 shows the vehicle ownership percentage which is having the value of 88% for 1 car ownership, 10% people having no cars, and 2% people having 2 cars, which shows that in Ramnagar, Roorkee areas most of the people have their own car. On the other hand, it shows that 26% of people have their own garage but 74% do not have same, which is shown in Fig. 4. Lack of personal garages people used the street to park their vehicles during night time.

6 Results and Discussion

6.1 Parking Accumulation

The amount of cars parked at any particular moment is referred to as parking accumulation. A parking accumulation survey has been conducted during the night time between 10 pm and 9 am. The survey result shows that the peak time of vehicle

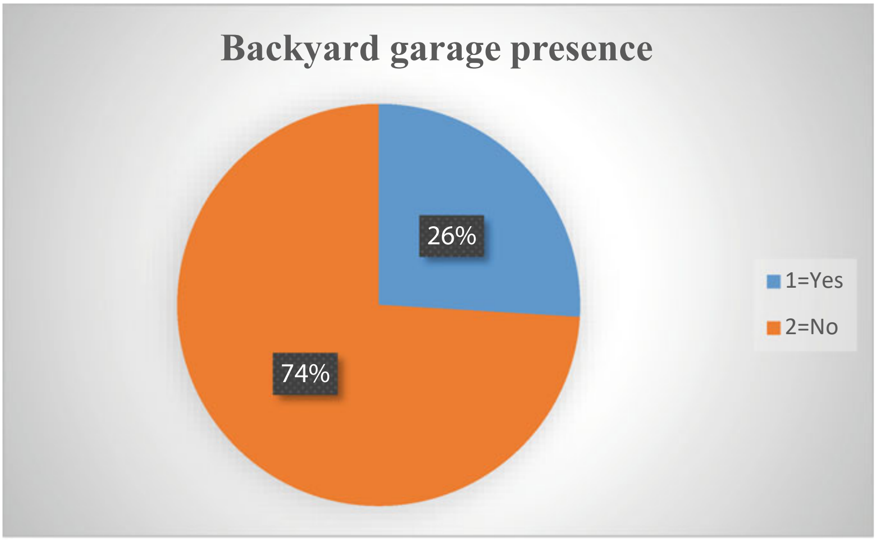

Fig. 4 Backyard garage presence

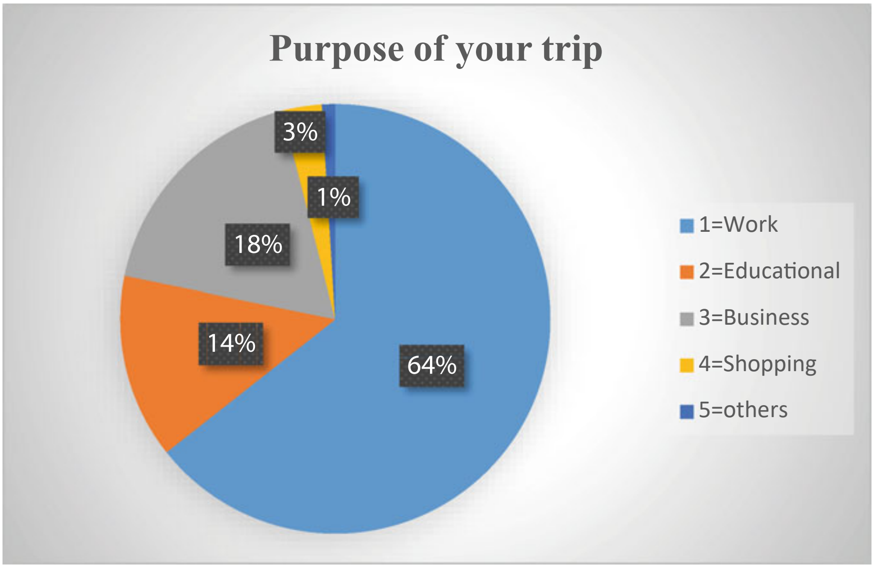

Fig. 5 Purpose of your trip

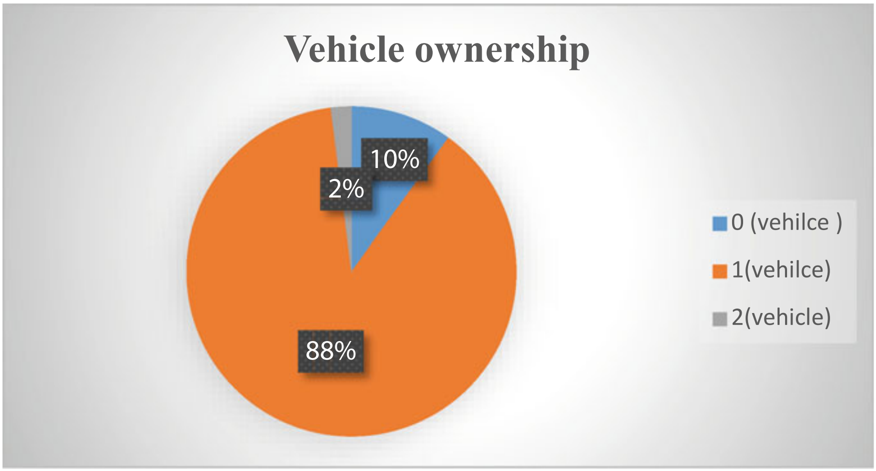

Fig. 6 Vehicle ownership

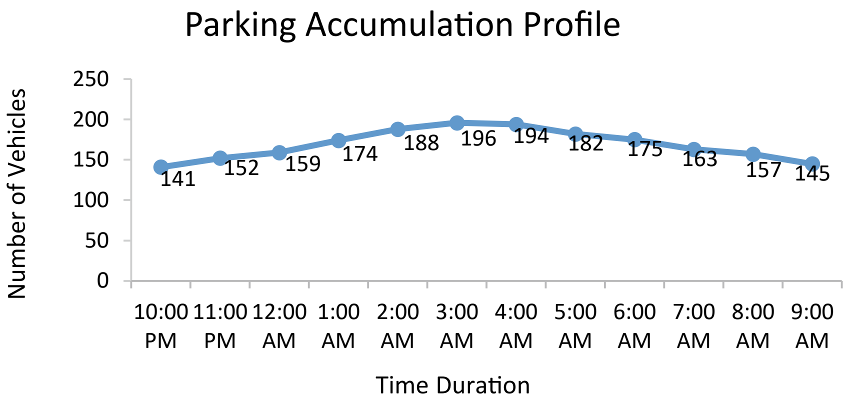

Fig. 7 Parking accumulation profile for Ramnagar, Roorkee

accumulation has been found to be at 3 a.m. and 196 vehicles are parked during this time. Figure 7 shows a parking accumulation profile of the vehicles.

6.2 *Parking Demand Model*

Data obtained from the questionnaire survey has been used to evaluate the demand model equation (Eq. 1) using linear regression analysis.

Table 1 shows that the number of cars parked is a dependent variable and the presence of a backyard garage is a significant factor along with the purpose of your

Table 1 Result of regression analysis by SPSS

Model	Unstandardized coefficients		Standardized coefficients	t	Sig.
	B	Std. error	Beta		
(Constant)	0.188	0.425		0.443	0.659
Any backyard garage presence (X_1)	−0.104	0.045	−0.178	−2.287	0.025
What is the purpose of your trip (X_2)	0.055	0.027	0.190	2.043	0.045
Vehicle ownership (X_3)	0.838	0.118	0.561	7.121	0.000
Dependent Variable: Number of vehicles parked at night time					

trip and vehicle ownership is also significant enough to influence parking demand. The R-square value of the equation is 0.62, That is, independent factors may account for 62% of the variance in parking demand. Other independent variables that were left out of this equation account for the remaining 38%. The demand model for Ramnagar, Roorkee is as follows.

$$Y = 0.188 - 0.104 \times X1 + 0.055 \times X2 + 0.838 \times X3 \tag{2}$$

6.3 *Sensitivity Analysis*

Sensitivity analysis was done using One Factor at a Time (OFAT) analysis to analyse the changes in the dependent variable by changing the independent variable. Each of the three significant variables was changed by 5, 10, 15, 20, 25, 30, 3%, and 40% and keeping the other two significant variables constant. It was observed that increasing the parking availability in the household decreases the parking demand on the street. On the other hand, an increase in vehicle ownership increases parking demand. The trip purpose does not affect the parking demand as significantly as the other two independent variables, as shown in Fig. 8. The constant values were also considered in the analysis as an independent variable with a value of 0.188.

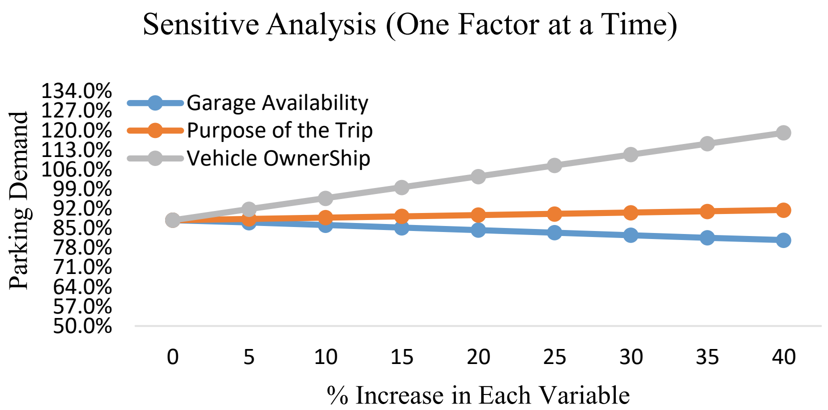

Fig. 8 Sensitivity analysis output for Ramnagar, Roorkee

7 Conclusion

Car parking demand has been evaluated using the demand equation and a multilinear regression model has been used to find out the parking demand during the night time. The result shows that the maximum number of parked vehicles has been found to be 196 vehicles and peak time 1–3 am respectively. Factors that affect the on-street parking demand have been analyzed. Presence of a backyard garage, trip purpose, and vehicle ownership are the parameters that are used to generate the demand model and the result shows that all of these parameters are found to be significant. This model and demand equation will help to reduce and control future on-street night parking demand in residential areas. The approach used in this study, as well as the demand equations, may be applied to estimate parking demand in other similar residential neighbourhoods. As a result, the current research not only expands the scope of this work to ensure future planning, but also allows the government to collect monthly fees from on-street night parking users.

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A Framework on Understanding the Barriers of Smart Cities and Intelligent Transportation System in India

M. B. Sushma, Y. Rashmitha, and Sandeepan Roy

Abstract The advancement in information technologies has eased human life by introducing smartness to our daily objects. The paradigm of smart cities arises with moving towards advancement and making the cities more futuristic. The idea of smart cities is spreading across the globe at a phenomenal pace, and India is not left behind. In the rapidly increasing economy India has also commenced an ambitious mission to transform 100 of its cities into smart cities with integrating the system with the advanced technologies, and Intelligent Transportation System. The research addresses the complex planning and the findings that indicate that are most significant category of barriers in achieving the target of smart city mission is followed by social, financial, technological, environmental, and legitimate issues. Reviews and analysis were done through various primary and secondary resources, such as existing literature, and views of experts given by conducting various critical SWOT analysis on the smart cities, government websites and media publications. The paper also adds the insights on understanding mobility in a smart city. This research will be useful for the government as well as policymakers for eliminating the potential obstacles in achieving the 100 smart cities mission and developing a sustainable transportation system in India.

Keywords 100 smart cities mission · Sustainable · Barriers · Urbanization · Intelligent transportation system

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1 Introduction

The idea of smart cities is enhanced through the need of development due to urbanization. People's shift from hinterlands to big cities is mostly in search of opportunities for social and economic benefits. As per a report by UN, the decadal growth rate of urban population was around 31.2% in 2011 and increased to 35.4% in 2021. It is estimated to rise to 40% by 2030 [1]. However, most of the cities lack basic infrastructural services due to poor administrative service, improper planning vision, shortage of funds, improper management, and to some extent the change of lifestyle. The issues associated with the burgeoning population in cities urgently need a solution to address the issues and ensure the sustainable harmonization of society, economy and environment. Thus, the evolution of the "Smart City" concept comes to identify ways to adapt to the rapidly varying dynamics of the current cities. However, there exist certain key challenges that hinder the smart city's pathway, such as urbanization and natural environment [2]. This mitigation depends on the strategy adopted to map out various urbanization issues by monitoring public health, education, communication, transport, environment, etc., and developing a systematic assessment and an integrative view.

The idea of the smart city is to solve the urban problems with the application of technology. Lately, the concept of smart city was extended by including the sustainability, quality of life and the serviceability to the citizens. It is a multidisciplinary model, and is still a difficult challenge to tackle the appropriate definition for the term 'Smart'. In literature, there are numerous studies which attempt to define smart city, as shown in Fig. 1.

2 Barriers in Smart Cities

As per the literature, in total of 31 barriers are listed that are broadly categorized into six major criteria, listed in Table 1.

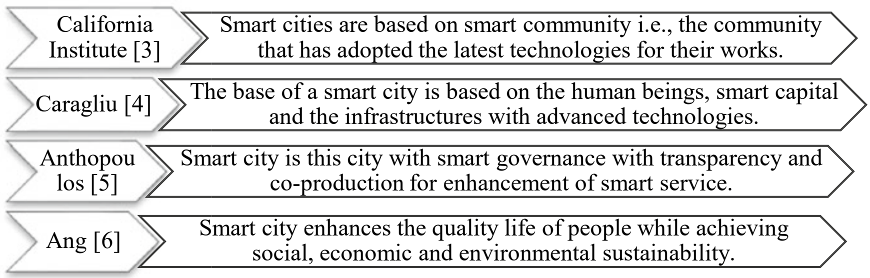

Fig. 1 Various definitions of smart cities [3–6]

Table 1 Barriers in the Smart City development

Key category	Barriers	References
Governance	Lack of coordination between city's operational networks	[7]
	Unclear IT management vision	[8]
	Instable state of politics	[7]
	Lack of trust in government and PPP	[9]
Economic	Dearth of high IT infrastructure and intelligence	[9]
	No competitiveness	[9]
	Higher cost of IT skill development programme for technology operation	[10]
Social	Lack of public awareness and their involvement	[7]
	Higher degree of inequality among the citizens	[10]
Technology	Lack of advanced technologies and skill	[8]
	Lack of integration and convergence of technologies across IT networks	[7, 11]
	Poor availability of data and scalable methods	[12]
Environment	Lack of ecological view, sustainability	[7]
	Rapid increase in population	[13, 14]
	Shortage of resources	[10]
Legal and ethical	Lack of creativity and cultural preservation	[8, 10]

3 Challenges of Smart Cities in India

The Government of India rundown to smart city mission in 2015 for which 100 cities were selected with an aim to improve the country's economic growth and enhance the living standards of the people through 'bottom up' approach [15, 16], shown in Fig. 2. The major problems associated are sustainability, governance, and infrastructure shown in Fig. 3. The vision of the smart cities is increasing with increase of urban population and rapid expansion of urbanization. The city should be efficient enough to manage the complexities and provide better service and enhanced quality of life. The objective of the smart cities is to drive sustainability and provide a clean and healthy environment.

Based on literature review and factors, a conceptualized framework is outlined that explains the relationship of the factors more coherently (Fig. 4). Some of the main infrastructure elements that are essential for smart city are mentioned below.

- Efficient public transport, mobility, safety, and security to the people.
- Housing, health, and educational facilities.
- Infrastructure to promote low carbon emission and green buildings.
- 24×7 water and electricity supply, proper sanitation, and solid waste management

Fig. 2 Representation of 100 cities in India selected for the smart city mission

Infrastructure

- Serious deterioration of quality of life, public heatlh and service of infrastructure
- Roads, public transportation

Governance

- Limited revenue base and dependent fiscal Jurisdiction.
- Mismanagement of resources
- Improper division of domain/work
- Absence of transparency and vigilance, etc

Sustainability

- Pollution challenges, congestions, inefficient energy transition
- Deteriorating quality of infrastructure's life.
- Inadequate share of renewable energy
- Overpopulation and waste management

Fig. 3 Challenges and issues of smart cities

4 Urbanization: Smart City Challenge for Transportation

In India, the rate of urban population is growing rapidly leading to a continuous demographic and spatial increase in the number and size of urban centres bringing up myriad challenges such as increase in traffic density, pollution, congestion and inefficient transport infrastructure. This demands the need to tackle the efficient strategy by utilizing the available infrastructure.

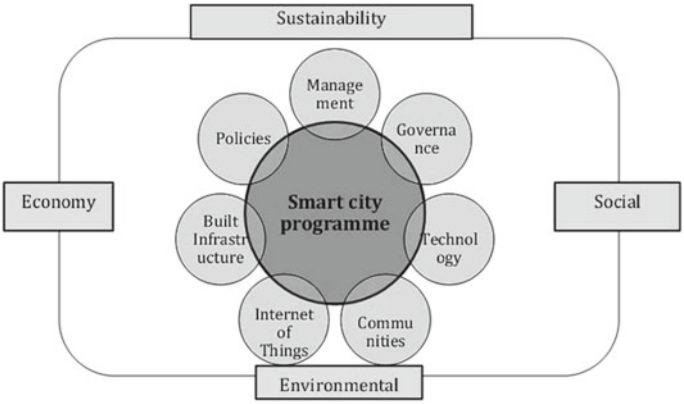

Fig. 4 Framework for smart city programme

4.1 *Intelligent Transportation System Architecture*

ITS are an essential part of smart cities. It utilizes various technologies ranging from core applications and decision-based information systems, as shown in Fig. 5. Intelligent vigilare system (IVS) is one of the applications of ITS that utilizes various forms of communication technology in line with data sensing, information processing and cloud storage being the key components. It is defined as cohesion of sensors, controllers, and transmission systems to enhance safety, mobility and traffic efficiency. ITS requires vehicles, roads and users' information and can help in improving the traffic management system, minimizing traffic congestion while minimizing the environmental impacts leading to an enable environment. Internet of Things (IoT) has majorly influenced the entire ITS.

Fig. 5 Integration of intelligent transportation system and smart city [17]

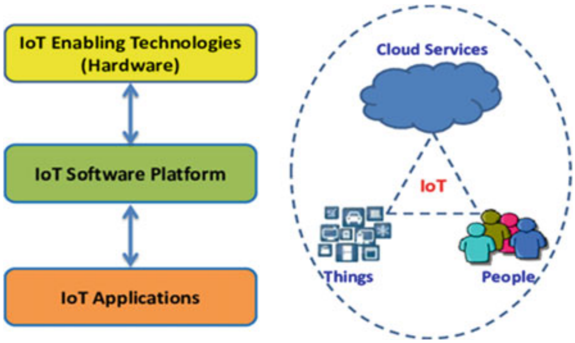

Fig. 6 The base IoT model local [19]

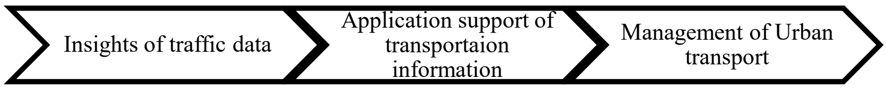

Fig. 7 Representation of ITS architecture

The major application of IoT in smart cities are smart manufacturing units, automotive, smart health care, etc. Integration of IoT with ITS has helped in improved efficiency to sense, communicate and analyze data and further transmit it to different users and/or devices through Internet [11, 18], shown in Fig. 6. ITS can help in enhancing planning objectives to build a sustainable transportation system. The architecture of ITS can be classified into three various parts, as shown in Fig. 7.

A driving anticipatory energy saving assistant is developed to examine traffic information mainly pertains to monitor vehicle speed, direction, acceleration and break known as operating conditions and other status attributes like type, colour, maintenance, vehicle number etc., and provide driving assistance to the user to improve vehicles efficiency [20]. However, anticipating vehicle position and motion information alone might not be enough to deal with the complicated road environment. Therefore, a modification was incorporated where components like vehicle information, road information, weather, road probing, and vehicle traffic sensing are collected concurrently via GPS and stereoscopic system [21], as shown in Fig. 5. The traffic sensing is further classified as fixed and mobile acquisition of raw data is one of the key components. In mobile collection technologies, sensors are responsible for data collection through feature extraction and high exactitude methods for laser range data scanning and visual detection respectively [22]. Another primary component understanding the pedestrian movements through some developed methods such as wavelet-based AdaBoost cascade [15], and combined shape–texture detection [22].

The next parameter is the merging traffic information, which is based on wired and non-wired transmission technologies of ITS. Optical fibre transmission is mostly used for trunk transmission and is a prime pillar in the development of wide area networks (WAN) and local area networks (LAN) roads. The other major FM radio communications are radio data system (RDS). Satellite communication technology is the only medium for dynamic location-based traffic monitoring. Some of the effective and widely used cellular communication are GSM, GPRS, 4G and 5G. The popular short-region wireless communication modes are Wi-Fi, ZigBee, Bluetooth, IEEE 802.11, 802.15, 802.16, and 802.209 [16]. The most commonly used approaches for understanding the traffic behaviour are probability statistics, hidden Markov model, and ANN [23].

The other most important component of ITS is urban traffic management and control assisted through traffic data collection system, and efficient traffic monitoring system. Effective data collection is ensured when traffic control system is properly unified with traffic signal control, urban traffic management and vehicle guidance system (VGS). Finally, urban traffic flow guidance system (UTFGS) is developed that is comprised of VGS, data analyzing platform, and a data distribution system for traffic assistance. The traffic organization can be optimized statically through resource management i.e., the capacity and the right-of-way distribution at various stages and dynamically through object management i.e., dispersion of the traffic on the roads [16].

4.2 An Outlook of ITS in India

In 1980s, the existence of intelligent transportation system came into account by the Japanese through intelligent vehicle system (IVS) programme. Further, the USA and Europe adopted similar technology and named it as Intelligent Vehicle Highway System (IVHS) and Advanced Transport Telematics (ATT) respectively. In India it was in 2006, ITS was introduced as National Urban Transport Policy (NUTP) and it mainly focussed on people's movement and not vehicles promoting ITS, clean fuel and other technologies. From 2006, there were various Strategies developed under the scheme of ITS. These are Jawaharlal Nehru National Urban Renewal Mission (JNNURM 2006), Sustainable Urban Transportation System (SUTP 2010) in coordination with United Nation Development Programme (UNDP), Atal Mission for Rejuvenation and Urban Transformation (AMRUT 2015)–the smart city mission, The National Transit Oriented development (NTOD 2017), Green Urban Mobility Initiative (GUMI), National Electric Mobility Mission Plan (NEMMP 2020) [24].

Enhancing traffic conditions through ITS includes few parameters such as traveller information and freight transport which can assist in improving safety and efficiency. Also, for the safety of commercial vehicles, weight-in-motion technology (WIM) is integrated in ITS that can be used to calculate the weight of vehicles without waiting on a scale. Furthermore, if there is a considerable reduction in overweighted vehicles, people would still be benefited through improved road conditions and less accidents. The other important thing, ramp metering which can aid in enhancing traffic conditions, however, it needs a detailed investigation before adopting.

Table 2 SWOT analysis of smart cities for Indian condition

Strengths	Weaknesses
• High energy efficiency of transport system • Accessibility • Emission legislations • Improvement of quality life • Potential shared mobility • Possibility of potential hybrid vehicles • Use of biofuels • Low Carbon emission	• Economically weak, inadequate maintenance of existing services • Lack of lane discipline, traffic regulation, public awareness • Increasing urban sprawl • Fragmented political and administration structures • Lack of connectivity of roads
Opportunities	**Threats**
• Usage of Clean fuel • Encouraging and supporting shared mobility and decreasing ownership of cars • Advancement in digital technology, Encouraging electronic payment • Development of ITS architecture as per the Indian context	• Improper and inactive stakeholders • Inactive government retribution • Unstable political situation • Integration of developments with the existing system

4.3 SWOT Analysis of Implementing ITS in India

It is a strategic planning and management technique for identifying the strengths, weaknesses, opportunities, and threats related to planning and selecting of ITS as per Indian context, as shown in Table 2. Analysis was carried out by considering existing literature, views of experts, government websites, media publications and surveys such as SWOT analysis for smart city Varanasi, Karnal, Indore, Public consultation for smart city Itanagar and a critical SWOT analysis for smart city planning [19]. Application of ITS can improve the comfort, safety and security of transportation through sustainable development policies.

4.4 Policy Recommendations for Better Implementation of ITS in India

A policy think-tank of Indian government, National Institution for Transforming India (NITI aayog) worked on developing and implementing Intelligent Transportation System (ITS) through introducing smarter mobility. For which it has setup a national level committee including members from various states and ministries. Diversity in committee members favoured in building up different subjects. They are electric mobility, traffic monitoring, fleet and parking management, enforcement of traffic rules, setting up ITS technical standards, generating awareness among the public and encouraging pilot projects. Considering the Indian scenario, the most challenging task seems to be integrating those developments with the present situation. Hence, focusing on developing affordable ITS for Indian driving conditions at the initial stage is recommended. Thus, substantial developments can be taken up with the growing awareness and acceptance among the people. They include moving away from "one size fits all" approach and working on distributing vehicular traffic. This can ultimately enhance traffic conditions and easy implementation of other policies in addition to these.

4.5 Status of Smart Cities in India

The Indian govt. has allotted a budget of US$ 868 million under the FY 2021–2022 for the Smart Cities Mission, which is 89% higher than the budget in FY21 which was US$ 457 million. As of March 2021, a total amount of US$ 27.60 billion was invested on 5,614 projects which were tendered worth US$ 23.29 billion, 4,912 projects work orders were issued worth US$ 18.84 billion and 2,421 projects worth US$ 5.41 billion have been achieved [24].

Effective plans, proposals and strategies were made for both area-based and pan-city development in 100 smart cities mission in India. Area-based development includes the rehabilitation of cities whereas pan-city projects which include retrofitting the cities, greenfield development at city extensions, redevelopment of the existing infrastructure and pan-city development visualize the application of the smart solution in the prevailing infrastructure. However, few projects were unsuccessful due to lack of planning and need for the implementation. Table 3 shows few initiated/completed in Urban transport sectors in various cities.

Apart from the urban transport and mobility, the core elements important for the development of the smart city is infrastructure, but the investment for building up these infrastructures is one of the major concerns in India, basically the smart city mission is sponsored centrally and also assisted by state government, ULB and also investments from foreign countries namely Spain, USA, Germany, Japan, France, Singapore, Israel, the Netherlands, UK, Hong Kong, Sweden and United States Trade and Development Agency (USTDA) assisted in development of states like Delhi, Allahabad, Ajmer, Vishakhapatnam, Bhubaneswar, Kochi, Coimbatore, Chennai, Ahmedabad, Varanasi, Amravati. Apart from investing in smart city mission countries like Italy and UAE contributed in terms of design, technology, and industrial services.

Here are few research opportunities on smart city initiative, effective data capturing, framework for successful implementation in developing countries, working to maintain collected data confidentially.

Table 3 List of some selected transportation projects initiated in smart cities

Smart cities	GOI funds (in crores)	Projects initiated/completed	Sector
Visakhapatnam	490	Supply, installation, commissioning, operation maintenance of adaptive traffic signalling system	Urban transport sectors
Chennai	490	MLCPs & ICT application-Thanickachalam road	Urban transport sectors
		Cycle sharing system with ICT application	
		Bicycle lanes	
		Footpaths for especially abled-Pedestrian Plaza	
		Traffic parks	Area development
Nashik	196	Concretization of surface for smart parking	Urban transport sectors
		Pilot smart road	
		Public bicycle sharing	
		National highway road project	Area development
New town Kolkata	294	Non-motorized transport and walkability, Construction of cycle track and barrier free footpath	Transportation and mobility
		Mixed land use and compactness	Urban transport
Udaipur	490	Public bicycle sharing PBS operations	Transportation and mobility
		Multilevel parking at chandpole phase—II	
		Construction of parking at MB hospital, Udaipur	
Amritsar	196	Development of open spaces	Area development

5 Conclusion

In the era of increasing urbanization, smart city turns up a solution. As the definition suggests a smart city should include basic requirements for the people. Safe transportation system becomes one of the primary requirements. This is because of urbanization growing at an alarm rate. Integration of ITS and other sophisticated technologies can help in making the transport system move smart, intelligent, and sustainable which boosts the development of economic sectors. This paper helps policymakers through a comprehensive review on integrating smart transportation systems and emerging technologies This shows the need for Indian cities to reframe their policies. The change should include vertically spaced institutional setting and horizontally linked sections with the local governance bodies. The political economic grants are one of the important contributors of successful implementation. However, the cities should rather focus on developing a strategic plan for strengthening its ULB. India needs to plan for a more integrated system for the core infrastructures to make the Smart City mission successful.

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Stepping Towards Environment Friendly Roads: Government-Initiatives in India

Yogesh Manoj Biyani

Abstract Road construction is a lengthy process that takes a long time for completion and has a variety of environmental concerns that, if ignored, can have negative effects. Additionally, since the raw materials used in road construction are typically putrescible and their extraction releases pollutants that have a negative impact on the environment, it can be expensive and require a lot of raw materials. Therefore, a phenomenal and ongoing increase in highway and all road construction throughout India under the current leadership has attracted the attention of various environmental critics. Recent international conflicts have raised market prices for many of these raw materials, but historical incidents have also demonstrated how such conflicts can disrupt the supply chain in global markets. In order to address this issue, the Indian government has implemented a number of initiatives through the MoRTH and other departments to ensure that the nation's infrastructural development isn't brought to a standstill. This article discusses the challenges that the government has to face in infrastructural development and the initiatives that MoRTH has taken, such as dredging, the use of repurposed and alternative materials, and the use of precast panels for building roads and highways.

Keywords MoRTH · NHAI · Dredging · Industrial by-products · Reprocessed material · Prestressed precast concrete pavement

Abbreviations

MoRTH	Ministry of Road Transport and Highways
NHAI	National Highway Authority of India
TCM	Thousand cubic metres
PPCP	Prestressed Precast Concrete Pavement
PPC	Portland Pozzolana Cement

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1 Introduction

The environmental consequences of highways continue to be widely discussed, particularly with expansion of road system, as roads are a result of changes in how people interact with their surroundings. Building roads is necessary to connect human communities, facilitate efficient access to natural resources, move goods to markets, and transport people to their places of employment. Whatever the goal, roads and ecosystems can interact in a variety of ways. Additionally, there is the use of combustible resources, the extraction of which frequently results in the accumulation of heavy metal compounds, the disruption of soil structures, the disruption of hydrological channels, the further disruption of plant and animal ecosystems, and even the interference with the migration of exotic organisms. In India, bituminous roads and rigid cement concrete roads are the two main types of pavement used in road construction. It is considered that flexible pavements are usually pervious compared to rigid pavements. However, some studies have said that there is little difference between the direct effects of the materials used to build cement concrete or bituminous roads because both kinds of pavements seem to be impervious by nature and therefore would experience a similar type of runoff [1].

Due to the construction activities, there is significant release of particulate matter into the atmosphere which heavily impacts the air quality. Further land clearing, ground excavation, cut-and-fill work, and the construction of a particular facility also adds up to the severity. These activities are major sources of airborne ultrafine particles [2], which are known to pollute natural water bodies [3]. People living and working near highways experience significant health and wellbeing effects from dust as well as other airborne pollutants from infrastructure projects. Additionally, being close to busy roads has been associated with an increased risk of cardiovascular disease, respiratory problems, and other harmful health effects. Lung conditions and chronic blockages have also caused death of construction site workers [4].

Another issue with road construction is that routine road maintenance and repair causes the pavement to be deeper than necessary, which damages nearby structures. The severity of this can be understood by referring to a report from 2013 in which the Joint Secretary of the Ministry of Home Affairs voiced concern about how routine carpeting for road repairs on Rajpath led to the burial of several stair levels that were originally built as rising-alters for neighbouring buildings [5]. These issues are exacerbated when underground civic projects like subsurface drainage, subsurface electric power supply, and others are being built, repaired, or maintained. It has been noted that newly built roads suffer damage from excavation required to carry out underground civic work tasks, which are then simply filled-in after tasks are completed. As a result, traffic is hampered; road sections need to be rebuilt, which necessitates additional funding [6] and natural resources are improperly exploited for raw materials, resulting in the needless accumulation of waste in disposal sites.

To address these issues and lessen the negative effects that may result from material procurement or infrastructure system construction the Government of India is pursuing various initiatives. These are the usage of alternative materials and research for pursuing panel-based construction systems (PBCS), which will be covered in the following sections.

2 Usage of Alternative Materials

2.1 Through Dredging

The well-known "Buldhana Pattern Jal Kranti" initiative is a great example of involving dredging mechanism for extraction and usage of alternative materials. In this initiative, MoRTH and MahaPWD worked together to dredge or excavate deposits from water bodies for use in construction, changing the image of Maharashtra's drought-affected areas. Being the idea of a Union Minister, NHAI performed the excavation and dredging work for free in order to obtain materials for highway projects at a lower price. These materials were further characterised according to their usage either as a material for pavement or as a landfill material. Continuous settlement in these water bodies during monsoon runoff had reduced their storage capacity, escalating the crisis, whereas the initiative helped in increasing the capacity of water bodies as well as improvement in the NHAI network. Being a part of the Ministry of Water Resources' "Jal Kranti Abhiyaan" national initiative helped with water body cleaning and conservation with the goal of ensuring that everyone has access to a reliable supply of water. This assisted in the creation of numerous job opportunities, multiple cropping patterns year-round and less dependence on tankers for drinkable water thereby completely transforming Buldhana from being water-scarce region into an aggregate supplier with the availability of adequate water supplies. 5.21 million cum of construction material have been extracted during the initiative from ponds, rivers, and small irrigation projects, resulting in an additional 5510 TCM storage capacity in the district [7].

The development is the result of the establishment of effective communication channels and the participation of the villagers in the project [8]. This contributed to proper fund utilization, allowing the minister to issue a statement of accomplishment [9, 10]. Additionally, in order to reduce pollution, MoRTH made agreements with state and local governments to dredge river sands for the preparation of "Inland Waterways" that would also be beneficial for NHAI as highway construction material. Moving forward with the same project, the governments of Telangana, Jharkhand, and Maharashtra are gearing up DPR for 7000, 3000 and 2000 kms, respectively [11]. Working toward the same goal, a Union Minister advised the Manipur government to issue a notification allowing the use of dredged sand in the construction of roads and highways in the state to reduce costs and enhance waterways [12].

2.2 Industrial By-Products

Industrial processes produce a variety of mineral by-products that have the potential to be recycled, reused, or used to develop new materials or goods. These residues are known as industrial by-products. While a few of those by-products can be used as raw materials again in the manufacturing process, there are many other industries where they can be put to use.

Industrial Slag: Slag is a by-product of the thermo-chemical smelting of ores and scrap metal. In general, it can be divided into three major categories: ferrous, ferroalloy or non-ferrous/base metals. Despite recycling as well as up-cycling efforts, there has been a significant increase in slag production over the years due to the high demand for these materials. According to the World Steel Association (WSA), 600 kg of by-products in the form of slags are produced for every tonne of steel produced [13]. Slags are transported to "slag dumps" with slag tailings, where they would be exposed to weathering and the possibility of toxic compounds and hyper-alkaline industrial discharges into the water and soil, risking the local ecosystems [14]. This system is quite similar to waste dumping or landfill grounds observed in the case of major cities. Non-ferrous/Base metal slags, which have greater concentrations of toxic elements, are typically the source of leaching concerns [15]. However, they may also be present in ferrous and ferroalloy slags, expressing concern regarding highly weathered slag dumping sites and up-cycled materials [16]. The Indian steel industry produced about 36 million tonnes of slag in 2020, and that number could rise to 60–70 million tonnes by 2030 [17]. The government has decided to use slag in road construction in order to address these problems and the potential to increase concrete strength through the admixing or replacement of slag in concrete [18, 19]. Recent example for the same can be observed in road construction with the usage of slag in city of Surat [20]. While government is moving forward with more such projects, this would bring better material sufficiency in procurement, as slag could be procured from steel plants itself.

Fly Ash: Fly Ash is a by-product of coal combustion that is made up of the flue gases and the particulates which are driven out from coal-fired boilers. The constituents of It vary significantly depending on the source and makeup of the coal being ignited, but it contains significant amounts of silicon dioxide, aluminium oxide and calcium oxide, the major mineral compounds in the coal-bearing rock strata. They have a very diverse mineralogy and may contain a number of dangerous substances that pose health risks. Unknown to the same, prior to the implementation of air pollution control standards, they were typically released into the atmosphere; however, these days, it must be captured before release by installing pollution control equipment.

Due to rising landfill costs and the current interest in sustainability, recycling Fly Ash has come to be a growing concern in recent years. Its abundance as a thermal plant waste and the silica and calcium content it contains make it suitable for replacements in cement forming PPC, additives in concrete mixes and a variety of other uses in the construction industry. In order to utilize properly the NHAI has mandated the use of 30% Fly Ash in the construction of roads within 100 km of thermal plants using coal or lignite [21]. This has been asked by the Ministry of Environment, Forest and Climate Change (MoEFCC) to amend the rule by making the building of roads within a 300 km radial distance of thermal plants mandatory. According to MoRTH, this is disputed because many thermal plants frequently demand exorbitant prices for the Fly Ash and frequently lack the quality needed for construction [22].

In response, the government mandated the use of Fly Ash-based products across all government schemes and programmes, with the transportation costs for Fly Ash now being covered entirely by Thermal Power Station up to 100 km and shared equally between consumer and Thermal Power Station for further than 100 km and up to 300 km. Fly Ash utilization, however, is still only accounting for 83.28% of total generation, falling short of the 2017 goal of 100% utilization [23]. Owing to the presence of coal reserves close to radioactive deposits, another concern is generated making Fly Ash micro-radioactive. Some researchers have demonstrated that Fly Ash has higher radiation levels compared to soil, which is concerning because regular exposure to it can have negative health effects [24].

Bio-Bitumen: Bitumen, a by-product of oil production and a very viscous liquid form of crude oil, is being used to bind the surfaces of pavements. It is a glue-like substance that adheres to the pavement's structural integrity, the aggregate framework and the waterproofing of the entire system [25]. Hence in India, this material is frequently used in road construction ranging from village roads to national highways. However, some researchers have begun to develop bio-bitumen as an alternative because they are concerned about the effects bitumen has on the environment and its future supply in the event that production in the petroleum market declines. There are currently bio-based survey roadways and cycleways in the Netherlands that are paved with a material that resembles bitumen and is produced from the natural binder lignin [26]. These include a section of a road on an industrial site that is regularly travelled by cars and larger vehicles, some minor roads, and a cycleway at Wageningen University and Research that is divided into three sections and made with various types of lignin-based bio-bitumen [27]. One of the top industrial biotech firms in India, Praj Industries, has created novel technology to create bio-bitumen based on lignin, which has been acknowledged by Biobased Delta, Europe's leading consortium for bio-bitumen samples made from purified lignin [28]. Additionally, MoRTH and Indian Oil Corporation Limited are collaborating to create bio-bitumen that can be used throughout India [29, 30]. MoRTH is planning to use more bitumen in the form of bio-bitumen in NH construction in order to take further environmental protection measures [31].

2.3 By Repurposed Material

Repurposed materials are waste products that are gathered and processed in a manner that allows for their transformation or redeployment for alternative uses, much like the "Reuse" theory.

Plastic Waste: As per Union Environment Ministry, plastic wastes greater than 25,000 tonnes is generated on a daily basis all over India, with only about 60% of this being recycled. The remaining portion is burned, which pollutes the air, clogs drains, becomes micro-plastic and enters the ocean. Without an effective waste management system, recycled plastics are frequently dirty, which makes the recycling process expensive and water-intensive. Union minister announced in 2016 that plastic waste would be used in road construction to address the environmental problem caused by plastic waste [32]. Since then, over one lakh kilometres of roads have been built in 11 states using plastic waste. The initiative's pioneer was the Municipal Corporation of Gurugram (MCG), which established guidelines for the use of plastic waste in arterial road construction after testing the idea in 2018 [33]. A few notable examples in India include the 270-km-long Jammu Kashmir National Highway, a two-kilometre stretch of the Delhi-Meerut highway close to UP Gate and road construction in between Dhaula Kuan-IGI Airport [33]. In addition to increasing environmental sustainability, roads made of plastic are found to be more reliable and economical. A tonne of bitumen is saved per kilometre of road construction by substituting 6–8% of it with plastic; this lowers the cost of road construction and helps the foreign exchequer. Because both plastic and bitumen are organic compounds generated from petroleum, hence they engage in better bonding with each other. The combination improves the road's lifespan and increases its capacity to support weight, helping roads to exhibit greater resistance to rain damage.

Processed Waste: Waste which has been separated for recycling or waste diversion and is going to an end user is referred to as "processed waste". In the construction industry, processed waste is helpful for landfilling operations. Therefore, waste must be processed properly to become stable or inert in nature to obtain appropriate landfilling material. We would encounter problems like toxins, leachate, or greenhouse gases in the event of improper processing. Some waste materials that are disposed of in landfills contain toxic chemicals that, over time, are released and seep into groundwater and soil. These substances are significant environmental hazards and can linger for a long time. Chemical reactions occur and a toxic leachate "cocktail" is created when water seeps through the landfill and gathers decomposed waste components. While, organic waste is typically compacted during landfilling, this causes anaerobic breakdown and eventual creation of greenhouse gases. NHAI was recently contacted by South Delhi Municipal Corporation regarding the use of processed inert waste for landfilling in highway construction projects [34].

3 Panel Based Construction Systems (PBCS)

In addition to usage of alternative materials, government is also moving forward with induction of PBCS where we would be able to reuse existing pavement instead of constructing a newer one. As one of the most expensive and time-consuming activities undertaken by the transportation department is the construction and repair of concrete pavement. A proper construction of a 1 km long 7.5 m wide concrete pavement takes nearly 94 days to complete [35], during which time numerous safety precautions had to be taken to ensure a smooth flow of traffic. While on-site casting of rigid concrete pavements, the main hindrance is traffic obstruction because the time needed to complete a regular concrete pavement includes base preparation, concrete pouring, and concrete curing. Since India's climate is primarily tropical or subtropical, water loss during construction is another important aspect that needs to be addressed. Therefore, in on-site construction, water content is significantly increased to retain workability during construction in order to prevent water losses, which frequently results in lower quality construction, making it difficult to maintain quality of construction [36]. According to some studies, using flexible pavement can help with the issue of slow construction pace, but doing so necessitates frequent maintenance, which makes construction and maintenance work a regular process [37]. Hence it is necessary to have a quick and low maintenance construction as a result of the growth in global traffic. As a result, precast construction techniques have become increasingly common in the construction sector. Construction of bridges and buildings using precast concrete is nothing new. Precast panels have become very popular for use in paving over the past ten years, and numerous pilot projects using this technology have been carried out internationally as well as in India [35].

One such PBCS for concrete pavement is known as Prestressed Precast Concrete Panel (PPCP), where as the name suggests precast concrete panels cast in laboratory setting and prestressed for improved strength caring capacity. As pavement panels are cast in a laboratory setting it allows for providing proper curing schedules, helpful to achieve desired concrete strength. This can be explained by comparing it to the distinction between shop-welding and site-welding in steel construction, where the safety coefficient varies depending on the type of location. As a result, with lab-casted panels, we can have a reduced depth for a panel with the same load capacity, saving material and lowering the cost of construction. Furthermore, since it is frequently noted that existing concrete pavements sustain damages during underground civil works that tend to call for reconstruction, PPCP would be helpful for achieving low-cost repairs in cases of underground civil works. With PPCP it would become convenient to perform underground civil works as the work panel can be uplifted and replaced back post-completion of work instead of demolition of the existing followed by reconstruction in case of regular pavements [38].

PPCP have been used by various agencies around the world for constructing roads, one such form of PPCP is known as Super Slab and the following are some notable examples of its usage [37] (Table 1).

In India, CC Precast Solutions have performed some notable works in development and installation of PPCP, proving capability of PPCP usage in India. Some of which can be seen as [39] (Table 2).

Same was been recognized by government for further usage [40].

Table 1 Projects by super slab

Year	Project	Agency	City
2001	Sagamore bridge rehab	Sagamore resort	Bolton landing
2003	9A Ramp Tarrytown	NYSTA (NY Thruway)	Tarrytown
2007	Chicago, IL Trial	ISTHA (Illinois Tollway)	Chicago
2007	I-294 Chicago	ISTHA (Illinois Tollway)	Chicago
2008	I-88 Chicago	ISTHA (Illinois Tollway)	Chicago

Table 2 Projects by CC precast solutions

Date	Project	Agency	City	Lane Len (m)
02/2018	College canteen road	VNIT	Nagpur	25
12/2018	Inner ring road	PWD, Nagpur	Nagpur	900
02/2019	Ultratech road	Ultratech	Bengaluru	12
10/2019	Rajiv Nagar, Hingna	PWD, Nagpur	Nagpur	900
03/2021	Deolamethi	Sadique and company	Nagpur	260
01/2022	Pathan Chowk	Welspun Ent	Amravati	2200

4 Conclusion

Road construction operations induce requirements for construction, repair, maintenance, and restoration lead to a variety of environmental concerns that need to be addressed. Government agencies are working to address this issue as their top priority through initiatives under the MoRTH umbrella. To achieve these goals, MoRTH has already partnered with other government agencies, academic institutions, non-profit organisations, and private companies to develop a range of innovative solutions. The initiatives have yielded a range of positive outcomes, like reduction in waste accumulation for industrial as well as residential waste through usage of alternative materials. Reusing the existing pavements with PBCS or PPCP would be beneficial in reducing pressure on material procurement as it would help in reducing demand of construction materials. However, a number of issues still persist; resolving them in quicker manner would include the active involvement of government.

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A Review of Berth Allocation Problem in Bulk Terminals

Adnan Pasha and Rajat Rastogi

Abstract This paper reviews the berth allocation problem (BAP) at bulk terminals. Initially, BAP, its constituent attributes related to spatial aspects, temporal aspects, handling time and performance measures are discussed. A classification of formulation approaches and solving methods is presented. These are discussed with respect to quay layout, i.e., discrete, continuous and hybrid. Majority of the studies minimize the total service times of the vessels followed by completion times and delayed departure times. Discrete bulk BAP models appear to be more robust in terms of the formulation and integration with other processes. Continuous and Hybrid problems have not been considered in detail as much as the Discrete problems. Future works can be explored by incorporating uncertainty, earmarking of berths by cargo type, modelling multiple cargos on same vessel, and indented berths in bulk BAP. Environmental considerations and overall operational integration can also be considered. This can be extended to Inland Water Transport (IWT) terminals. These will have higher applications on terminals in developing countries which majorly deal with bulk cargo.

Keywords Berth allocation problem · Bulk terminals · Problem classification

1 Introduction

Maritime transport plays a vital role in boosting global trade and economy as more than 80% of the global trade by volume is transported by ships and handled at ports. Hence, improving operational efficiency and service quality of port terminals is necessary because they are the main connecting nodes between marine and land transportation. Various planning problems arise in a terminal related to seaside, yard-side and landside areas across strategic, tactical, and operational time horizons [3, 4]. A typical layout of a terminal (bulk) and its areas is shown in Fig. 1.

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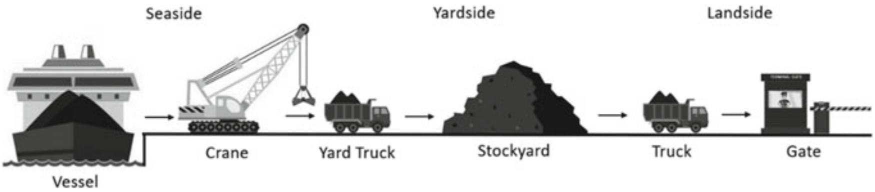

Fig. 1. Bulk terminal layout and operation areas

The planning problems also vary based on the terminal type, i.e., container, dry bulk, liquid bulk, break-bulk, roll-on-roll-off (ro-ro), and multipurpose terminals (MPT). The first five types of terminals are specialized terminals as they handle only a specific cargo type, while the MPTs handle all types of cargo. Terminals in the developed countries underwent specialization and containerization in the late 1990s and early 2000s. This has resulted in research focused on container terminals, even though bulk constitutes over 80% of maritime trade [35].

Since berths are the most critical resources for cost-effective terminal management, the BAP is the most critical optimization problem in terminal operations. BAP is the optimal assignment of berthing spaces and berthing times to incoming vessels at the terminal. Studies on BAP before the 1990s focused on queuing theory which failed to capture several attributes of the problem, mainly the spatial attributes. Since then, research in the BAP has mainly concentrated on mathematical programming. By the late 1990s, research in BAP was started in developed countries. Comprehensive literature reviews on BAP in the context of container terminals can be found in [3, 4].

This paper reviews the BAP in the context of bulk terminals. It first presents BAP and its variants, then provides a literature overview on BAP in bulk terminals, and finally indicates research gaps and future directions in this domain.

2 Berth Allocation Problem

BAP works with vessel data like arrival times, length, draft, cargo type, quantity of cargo etc. and terminal infrastructure data like quay length, berth/storage capacity, distance from storage, equipment, etc. The terminal operator must allocate berthing spaces and times to these incoming vessels at the terminal in an optimized manner. A feasible berth allocation is represented on a time-space diagram in Fig. 2(a).

The horizontal axis denotes the time horizon, while the vertical axis represents the quay length. The rectangles indicate the vessels berthing at the terminal. On the horizontal axis, the rectangle's width represents handling time, with left and right vertices representing berthing time and completion time, respectively. These times are also demonstrated in Fig. 2(b). On the vertical axis, rectangle's length corresponds to vessel length, with the two vertices representing the berthing position of the vessel.

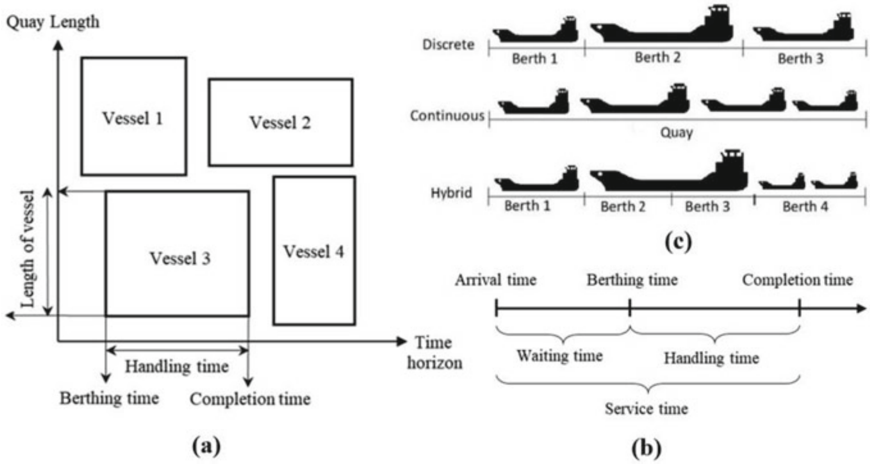

Fig. 2 **a**. BAP time-space diagram, **b**. Berth operation timeline, **c**. Berth layouts at terminal

For a feasible berth allocation, these rectangles (vessels) should not overlap in time and space simultaneously.

There are several BAP variants in the literature based on the assumptions being made on input attributes [3, 4]. The attributes are related to spatial aspects, temporal aspects, handling time and performance measures (see Fig. 3). Spatial attributes concern the quay layout, which can be *discrete*, *continuous* or *hybrid,* as shown in Fig. 2(c). Vessel *draft* is also a spatial attribute. Temporal attributes are related to limitations on berthing and departure times, the most important being the arrival time of vessels, which can be *static*, *dynamic*, or *stochastic*. *Static* means that all vessels are already at the terminal, *dynamic* means that they arrive over time, and *stochastic* means that the arrival times are uncertain. Sometimes vessels have predefined *due dates* or maximum waiting times also. Bulk carriers are usually larger than their container counterparts, and sometimes they can enter or leave the terminal only during certain *tide* windows. Handling times can be *fixed*, *position-dependent*, *equipment-dependent*, or *stochastic*. Performance measures relate to the minimization of a function comprising one or more of these attributes (Refer Fig. 3)

After a vessel arrives at a terminal, it may endure a *waiting time* before its berthing. After berthing, the time taken in loading and/or unloading is termed as *handling time*. *Completion time* or departure time denotes the time when the handling finishes, and the vessel leaves the terminal. Most studies consider minimizing vessels' total service time, which is the sum of their waiting and handling times. This timeline is demonstrated in Fig. 2(b). *Tardiness* refers to the delay in departure. *Position* refers to the minimization of horizontal transport by berthing vessels closest to their respective storage areas. *Earliness* denotes the rewards payable to the terminal operator for enabling the completion of handling before the stipulated time. *Order* represents the deviation between customer priority and berthing order of vessels. Sometimes utilization of terminal *resources* like vehicles, equipment, manpower, etc., is

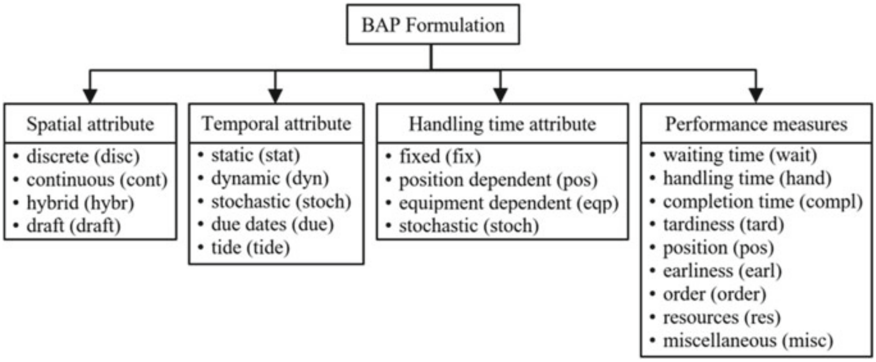

Fig. 3. BAP classification scheme (modified from Bierwirth and Meisel [3]

also optimized. Other performance measures are taken as *miscellaneous*. Combining different attributes can lead to a variety of BAP models, which are discussed in the next section.

3 Literature Overview

Table 1 lists the mathematical models for bulk BAP in lexicographic order based on the attributes discussed in Sect. 2. These relate to discrete, continuous and hybrid quay layout, and use solution approaches based on solvers like CPLEX, LINGO and Xpress to other exact algorithms, as well as heuristics and metaheuristics approaches which deliver high-quality solutions with less computational effort. The models are based on performance measures. All the objective functions listed in Table 1 are minimization functions. A Σ() function defines a summation for all vessels and Max() function is used for the worst-performing vessel. Weights, priorities, or costs are indicated by 'w'. A comma (,) is used to separate objective functions in a multi-objective model, and a slash (/) is used when there is more than one model. A tuplet notation [3, 4] is used to classify the mathematical models. For instance, disc| dyn, due | pos | Σ w (wait + hand) specifies that the model deals with discrete berths, dynamic arrivals with due dates, handling time is dependent on vessel position, and the objective function is to minimize the weighted total service time.

3.1 Discrete Problems

Extensive literature on BAP in the context of container terminals is available, but the research on BAP in bulk terminals started very late in 2011. Barros et al. [2] were the

Table 1 Classification of bulk BAP models

Model	References	Method
disc ǀ stat, tide ǀ pos, eqp ǀ Σ w hand + misc	[11]	Heuristic
disc ǀ dyn ǀ pos ǀ Σ (wait + hand) / Σ w(wait + hand) / Σ (w wait + hand) / max (compl)	[31]	PSO
disc ǀ dyn ǀ eqp ǀ Σ (wait + hand)	[24]	GA
disc ǀ dyn ǀ eqp ǀ Σ (wait + hand), Σ order	[25]	GA
disc ǀ dyn ǀ eqp ǀ Σ (wait + hand), Σ order	[26]	GA
disc ǀ dyn, due ǀ pos ǀ Σ w (wait + hand)	[30]	CS
disc ǀ dyn, due ǀ pos ǀ Σ (w wait + misc), misc	[22]	PSO
disc ǀ dyn, due ǀ pos ǀ Σ (w_1 tard - w_2 earl)	[27]	ALNS
disc ǀ dyn, tide ǀ fix ǀ Σ w (wait + hand) + misc	[20]	ALNS
disc ǀ dyn, tide ǀ fix ǀ Σ misc	[2]	SA
disc ǀ dyn, tide ǀ pos ǀ Σ misc	[14]	Heuristic
disc ǀ dyn, tide ǀ pos, eqp ǀ Σ w compl	[36]	BD
disc, draft ǀ dyn ǀ pos ǀ Σ (wait + hand)	[19]	CPLEX
disc, draft ǀ dyn ǀ pos, eqp ǀΣ (wait + hand)	[23]	CPLEX
cont ǀ dyn ǀ fix ǀ Σ (wait + hand)	[29]	CPLEX
cont ǀ dyn ǀ fix ǀ Σ (w_1 tard + w_2 misc)	[37]	Heuristic
cont ǀ dyn, tide ǀ fix ǀ Σ compl	[12]	CPLEX
cont ǀ dyn, tide ǀ fix ǀ Σ compl	[8]	VNS
cont ǀ dyn, tide ǀ pos ǀ Σ compl	[9]	ILS
cont ǀ dyn, tide ǀ eqp ǀ Σ w compl	[17]	CPLEX
cont ǀ dyn, tide ǀ eqp ǀ Σ w tard	[15]	VNS
cont ǀ dyn, tide ǀ eqp ǀ Σ w tard	[16]	RHOH
cont ǀ dyn, tide ǀ eqp ǀ Σ w tard	[18]	VNS
cont, draft ǀ dyn, due ǀ pos ǀ Σ (w_1 tard − w_2 earl + pos)	[7]	Xpress
hybrǀ dyn ǀ pos ǀ Σ (w_1 compl + w_2 res)	[13]	LINGO
hybr ǀ dyn, tide ǀ fix ǀ Σ (wait + hand)	[38]	GA
hybr ǀ dyn, tide ǀ pos ǀ Σ (wait + hand)	[21]	GA
hybr ǀ stoch ǀ pos, stoch ǀ Σ (wait + hand) + misc	[32]	RHOH
hybr, draft ǀ dyn ǀ pos, eqp ǀ Σ (wait + hand)	[34]	CPLEX
hybr, draft ǀ dyn ǀ pos, eqp ǀ Σ (wait + hand)	[33]	GSPM
hybr, draft ǀ dyn ǀ pos, eqp ǀ Σ (wait + hand)	[28]	B&P
hybr, draft ǀ dyn ǀ pos, eqp ǀ Σ (wait + hand)	[1]	CPLEX

(continued)

Table 1 (continued)

Model	References	Method
hybr, draft ∣ dyn, due ∣ pos, eqp ∣ Σ (w_1 tard − w_2 earl)	[6]	Xpress
hybr, draft ∣ dyn, due, tide ∣ pos, eqp ∣ Σ (w_1 tard − w_2 earl) / Σ compl	[5]	Xpress

Abbreviations used in the last column: ALNS—Adaptive large neighbourhood search, B&P—Branch and Price, BD—Benders decomposition, CS—Clustering search, GA—Genetic algorithm, GSPM—Generalized set-partitioning model, ILS—Iterated local search, PSO—Particle swarm optimization, RHOH—Rolling horizon optimization heuristic, SA—Simulated annealing, VNS—Variable neighbourhood search

first to address this as well as the influence of tides in BAP. They developed a mixed-integer linear programming (MILP) formulation based on transportation problems to minimize the overall operational cost with tide and stock level constraints. Simulated Annealing (SA) based algorithm was used as a solution procedure. Junior et al. [14] used a greedy heuristic procedure to solve the same problem without considering stock level conditions.

Pratap et al. [25] presented a bi-objective formulation of BAP, minimizing the vessels' waiting time and the deviation of berthing order from vessel priority. The problem was solved using a modified non-sorting genetic algorithm (NSGA-II) and data from an Indian port. Along similar lines, Pratap et al. [24] integrated BAP with quay crane allocation for minimizing the total service time. They solved the model using a block-based genetic algorithm (BBGA) and a genetic algorithm (GA). With the same objectives as Pratap et al. [24], Pratap et al. [26] presented a decision support system (DSS) integrating BAP with ship unloader allocation. The problem was solved sequentially as well as simultaneously, and the results indicated that the latter gives better results than the former. A controlled elitist non-dominated sorting genetic algorithm (CENSGA-II) and chemical reaction optimization (CRO) were adopted as solution approaches. Ribeiro et al. [27] extended the formulation of Cordeau et al. [10] and minimized demurrage (extra fees) and maximized despatch (reward amount) in an ore terminal while considering the maintenance to be carried out at berths. They proposed an adaptive large neighbourhood search (ALNS) heuristic, which yielded good solutions in reasonable computational time. Tang et al. [31] adopted multi-phase particle swarm optimization (PSO) to minimize total service time with and without vessel priority and makespan (maximum completion time) (refer Table 1). The models give varied results in terms of total and maximum service time, which can be useful for different situations in a terminal. Rosa et al. [30] also extended the Cordeau et al. [10] formulation by considering multiple cargo types on vessels with different priorities. A clustering search (CS) metaheuristic was employed to solve the problem instances generated from a Brazilian terminal.

Various studies integrated the BAP with other terminal operational problems. Unsal and Oguz [36] took the integrated approach to the next level by considering reclaimer scheduling and yard allocation in addition to BAP taking tidal effects into account. They minimized the weighted completion time and developed an exact benders decomposition (BD) algorithm to solve the problem using MILP and constraint programming (CP). Perez and Jin [23] proposed an integrated cargo-specific BAP and specialized handling equipment (grab cranes, conveyors, pipelines, etc.) assignment to minimize the total service time. The commercial solver CPLEX was used for solving the MILP model. Liu et al. [20] integrated the BAP with vessel sequencing in a navigation channel with tidal conditions. Their MILP formulation minimized the weighted dwelling time of the vessels. A tailored ALNS approach was used which outperformed the solver, column generation (CG heuristic), SA and GA. de Andrade and Menezes [11] integrated BAP, yard allocation, planning and scheduling problems by minimizing penalties regarding handling, transportation, and not meeting supply of products or demand of vessels. They developed a relax-solve-and-fix heuristic (RSFH) whose outputs were satisfactory. Peng et al. [22] integrated BAP with shore power allocation achieving a tradeoff between installation and usage cost of shore power and greenhouse gas emissions. The multi-objective model was solved using a multi-objective PSO.

3.2 Continuous Problems

Xiaona et al. [37] first presented the bulk BAP with a continuous quay layout. They minimized the total deadweight and delayed departure of vessels and used a heuristic algorithm to solve the problem. Rodrigues et al. [29] considered a BAP where certain continuous segments along the quay were earmarked for specific types of cargo, as is the case in some Brazilian ports. They developed a MILP model minimizing total service time and solved it using CPLEX. Ernst et al. [12] presented two MILP formulations for the BAP considering tidal time window constraints. The first was a model based on sequence variables, and the second was a time-index-based formulation. The results from CPLEX indicated that the latter was more suitable for large sized instances. Cheimanoff et al. [8] solved the MILP formulation of Ernst et al. [12] using a reduced VNS and utilized machine learning for parameter tuning. Cheimanoff et al. [9] extended this model to multiple continuous quays and used an iterated local search (ILS) as a solution procedure.

Krimi et al. [15, 16, 17, 18] integrated BAP with crane assignment in multi-quay bulk port with tidal and availability constraints. Krimi et al. [17] minimized the weighted completion time, while Krimi et al. [15, 16, 18] minimized weighted tardiness (delayed departure). CPLEX, variable neighbourhood search (VNS) and rolling horizon optimization heuristic (RHOH) were used as solution procedures. With a similar quay layout, Bouzekri et al. [7] integrated the BAP with Laycan Allocation Problem (LAP). The latter assigns berthing time windows to vessels based on the availability of port resources. The objective function maximized the difference between dispatch and demurrage and favoured berthing close to the respective storage yard. Xpress solver was used to solve the problem.

3.3 Hybrid Problems

Umang et al. [33, 34] were the pioneers in truly distinguishing the BAP in bulk terminals with respect to container terminals. The key differences include incorporating different cargo types and heterogenous equipment facilities like mobile harbour cranes (MHC), conveyors and pipelines in the BAP. Umang et al. [34] developed a MILP model minimizing total service time and solved it using CPLEX. Umang et al. [33] proposed an additional generalized set-partitioning problem (GSPP) formulation, which proved to be more efficient. The MILP formulation was also solved using a squeaky wheel optimization (SWO) algorithm. As an extension to these, Robenek et al. [28] integrated the BAP with yard assignment and used a branch and price (B&P) approach to solve an improved MILP formulation. Al-Hammadi and Diabat [1] too extended this work by adding constraints related to yard storage capacity and heavy-weight cargo. Grubišić et al. [13] developed a MILP model to solve BAP in inland waterway ports minimizing the completion time of barges/convoys and workload of port operators, taking cargo types into consideration. Finally, Umang et al. [32] considered BAP under uncertainty wherein they developed a real-time formulation with stochastic arrival and handling times, minimizing the total service cost of vessels.

Liu et al. [21] developed a GA to solve the BAP considering tidal time windows to minimize total service time. Following Bouzekri et al. [7], Bouzekri et al. [6] maximized the difference between despatch and demurrage, considering a conveyor system with routes sharing one or more conveyor belts. Finally, Bouzekri et al. [5] combined these two models to solve an integrated BAP and LAP considering conveyor routing constraints, preventive maintenance activities, multiple quays, different cargos, and tidal restrictions, Yao and Wu [38].

4 Research Gaps Identified

Following research directions on BAP in bulk terminals can be pursued in future:

(a) Uncertainty in the arrival or handling times, and/or due to material handling equipment breakdowns, unscheduled vessels, and general disruptions can be incorporated in the bulk BAP models. This will make the models more robust.
(b) The bulk BAP models shall include earmarking of berths based on cargo type, modelling multiple cargos on same vessel, use of indented berths, etc. These are the real scenarios which may have application in the developing countries.
(c) Environmental considerations like fuel consumption and emissions of berthing vessels can also be considered in the models.
(d) For overall optimization of terminal operations, the bulk berth allocation shall be integrated with yardside and landside operations.

(e) BAP can be explored in IWT terminals which may be either specialized or multipurpose in terms of cargo handling. IWT terminals may involve direct transhipment of cargo, parallel berthing or ro–ro cargo which would impact the berth allocation.
(f) To evaluate the efficiency and effectiveness of BAP models, proper benchmarks need to be established which must be unbiased and comparable.

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MCDA Approach to Evaluate Planning and Consolidation Freight Strategies for Sustainable Goods Distribution: Case of Indian City Jaipur

Pankaj Kant, Sanjay Gupta, and Ish Kumar

Abstract This paper aims to rank and weights the planning and consolidation freight strategies from the literature in the city of Jaipur in India. Two multi-criteria decision-making (MCDA) techniques, the Analytic Hierarchical Process (AHP) and the Best Worst Method are used to analyze the weights of various planning and consolidation freight strategies (BWM). The hypothesis for using AHP and BWM is assumed that both methods produce equivalent results and are suitable techniques to select freight strategies in urban areas keeping the concerns of stakeholders of city logistics. This study includes wholesalers, shippers, transport operators, policy planners, and city administrators as key stakeholders in the case city for the evaluation of the planning and consolidation strategies for sustainable urban goods distribution. Face-to-face paper and pencil survey methods were used with key stakeholders to determine the weight of strategies. The study's policy implications show how MCDA techniques could be applied in India's city logistics domain as a decision-making instrument.

Keywords Urban freight · Sustainability · Strategies · AHP · BWM

1 Introduction

Many nations have made sustainability in urban goods transportation a top political concern [1]. Sustainable development overall depends on sustainable transportation of passenger and goods movement [2]. In the urban setting, freight strategies and key performance indicators (KPI) are crucial for the decision-making process

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[3]. To achieve sustainable development goals in urban areas, decision-makers can employ sustainable freight distribution methods [4]. Policy planners dealing with urban freight management must evaluate how freight vehicles affect urban mobility and sustainability [5]. It is difficult to predict how freight rules and strategies will play out due to the complexities of urban freight stakeholders' interaction and their priorities [6]. The relationship between stakeholders' behaviours and freight policies and strategies is still being researched in the city logistics domain [7]. It is necessary to increase awareness of freight issues while taking into account all key stakeholders [8].

Urban freight is regrettably overlooked in both policy discourse and recommendations for the development of citywide transportation systems in India [9]. Even the Government of India's flagship smart cities initiative includes only a few concepts for urban freight transportation and management that are being taken into consideration for implementation in these cities [10]. The purpose of urban planning in particular is to allow the efficient circulation of goods at predetermined levels to maximize efficiency [11]. Policymakers face a significant problem in selecting effective freight strategies for sustainable commodities distribution because of the diverse nature of urban freight stakeholders and the characteristics of goods movement in the city.

The AHP and BWM approaches are used in this research article to assess the relative importance of various planning and consolidation freight strategies for the distribution of urban commodities in the Indian city of Jaipur. AHP and BWM methods can be useful tools which can assist the decision maker in making optimal choices in city logistics [12, 13]. The research methodology for this research paper was developed in the context of wholesalers, transportation providers, and local policymakers involved in the distribution of urban goods in urban settings.

The following section provides a review of the literature on freight distribution strategies in urban areas. Section 3 discusses the characteristics of the case city and the selection of the wholesale commodities market. In Sect. 4, the research approach is covered. The AHP and BWM models' formulations are discussed in Sect. 5. Section 6 presents the evaluation of planning and consolidation of freight strategies utilizing AHP and BWM, and Sect. 7 complies with the conclusion with policy recommendations.

2 Literature Review

Urban freight deliveries, transportation fleet, economy, environment, and safety are some of the metrics that can be used to assess the effects of urban freight transport in cities [14]. Various urban freight strategies were adopted based on the results of a thorough analysis of urban freight data from 56 cities in 32 nations [15]. For sustainable urban goods distribution, three verticals of urban freight systems have been developed: last-mile in urban freight, freight impact on the urban environment, and trade in cities. 21 urban freight strategies that have already been used in numerous cities throughout the world are divided into these three categories [5, 16, 17].

Policy initiatives for adopting freight strategies for sustainability in urban freight transport are categorized for policy planners, wholesalers, shippers, receivers, and transport operators level [18]. City administrators and policy planners struggle to interact with other urban freight actors, such as transportation companies and shops, while designing and implementing freight strategies [19]. Key concerns of city policy planner's concerns about freight demand and supply should be supplemented by city logistics freight policies and strategies [20].

For municipal authorities and corporate partners, a comprehensive compendium of global best practices in sustainable urban freight strategies based on their advantages and disadvantages has been already compiled and available in the literature especially in developed countries [21]. The implementation of urban freight strategies is hampered by freight stakeholder's priorities hence there needs to be a dialogue among those involved in urban freight distribution [22]. When designing and developing transportation projects, different freight operation strategies need to be taken into account for a collaborative and systematic assessment of management and operations (US-DOT, 2013) [23]. When creating a freight transport plan, many important factors must be taken into account, such as design standards, infrastructure design, land use zoning, and vehicle control (NCFRP report 14, 2012) [24].

Sustainable transportation in metropolitan areas necessitates a mix of freight strategies and good collaboration between administration and private stakeholders. Many times, transportation management plans (TMP) are adopted without a complete understanding of their potential benefits and drawbacks [6]. Due to the widespread usage of large vehicles in commodity distribution, even a minor increase in freight efficiency can have a significant positive impact on city sustainability [25].

Sustainable freight transportation necessitates a framework for the development and selection of freight solutions that take stakeholder concerns into account [3]. The multimodal freight transport system's alternatives were evaluated using the AHP technique [26]. Planning, resource management, company policy, public policy, and performance issues can be evaluated from the MCDA technique [27].

The AHP approach in the city logistics and transport sector can be combined with a stakeholder-driven or institutional approach [27]. For evaluation and supplier selection in the context of social sustainability, the BWM technique was utilized [28]. The many variables affecting building energy efficiency, as well as the challenges to minimizing these variables, were examined using the BWM tool [29]. BWM is a suggested method for setting parameter weights since it is clear-cut and simple to understand [30].

Due to the varied characteristics of the stakeholder's involvement in the distribution of urban products, the effects and advantages of freight strategies can vary. Policy intervention in urban goods distribution will be helped by an evaluation of freight strategies in the context of sustainable urban development by local city administrators.

3 Methodology and Data Collection

This study aims to evaluate the relative significance of freight strategies in two wholesale markets in Jaipur city of India. These two wholesale markets are the building hardware market and the electronics market. The hypothesis behind the use of AHP and BWM techniques is that both approaches yield comparable outcomes and are viable methods for choosing freight strategies in urban areas while considering city logistics stakeholders' concerns.

Planning and consolidation freight strategies for this research study are primarily based on the recommendations of NCFRP Report 23, 2012 by the Transport Research Board of the US on the Synthesis of freight research in urban transport planning and CIVITAS, 2015 guide for the urban transport professionals by the European Commission [21, 31, 32].

Figure 1 shows the framework used for evaluating planning and consolidation strategies and their variations among two commodity distributions in the city of Jaipur. The geometric mean of individual replies with consistency ratios under 10% makes up the final weights of freight solutions. In the last stage, the weights obtained by the AHP and BWM methods were statistically analysed using IBM SPSS software to conduct Kendall's tau-b correlation test to assess the ordinal relationship between the results obtained by both methods.

Table 1 shows the sample size collected for various stakeholders.

A list of wholesalers for both markets was obtained from respective market associations to initially select the wholesalers. Telephone conversations were utilized to set up in-person paper-pencil surveys on certain dates and times with all stakeholders. Only wholesalers who have been in business for more than five years were

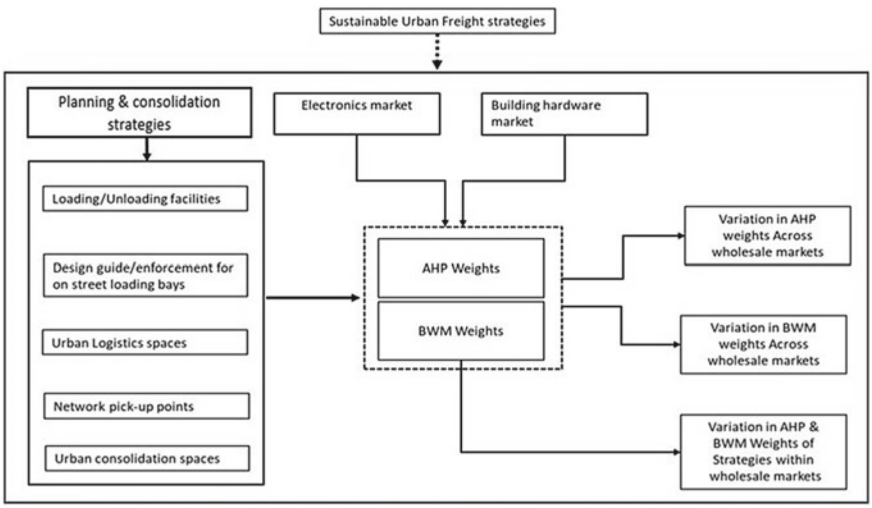

Fig. 1 Framework for evaluating planning and consolidation strategies

Table 1 Sample size

S.no. Stakeholders	Sample size (AHP)	Sample size (BWM)
1 Wholesalers-building hardware	10	10
2 Wholesalers-electronics market	10	10
3 Traffic management /enforcement	2	2
4 Urban planners	2	2

selected for the survey. A list of traffic police personnel with more than two years of experience in and near these two wholesale markets was provided by the Jaipur traffic police department for the survey. Depending on their availability, the core urban planning experts of the Jaipur Development Authority have been selected for the interview. Four trained enumerators were assigned the duty of gathering four samples each day. The data from enumerators were analysed on the same day via BWM and AHP spreadsheet-based calculator to check the consistency of individual results. The process of conducting the survey was repeated with other stakeholders in the list until the required sample size was gathered.

4 Case City Profile

Jaipur city is the capital of Rajasthan state in India. The total area of Jaipur city is 2939 km^2 out of which the walled city area constitutes 17 km^2. Jaipur city has a total population of 30.5 lac people (yr. 2011). The land use distribution is shown in Fig. 2.

Figure 3 below shows the location of significant wholesale markets in Jaipur city.

For the evaluation of planning and consolidation strategies, the building hardware and electronics market was selected. The goods distribution in the building hardware market was weight-based whereas in the electronics market, it is based on item numbers. The location of electronics market is in a walled city where warehouse and storage are critical due unavailability of required space. Connectivity to the electronics market is also limited due to its location in a walled city. The building

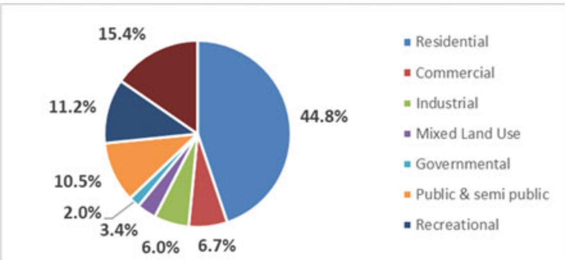

Fig. 2 Land use distribution

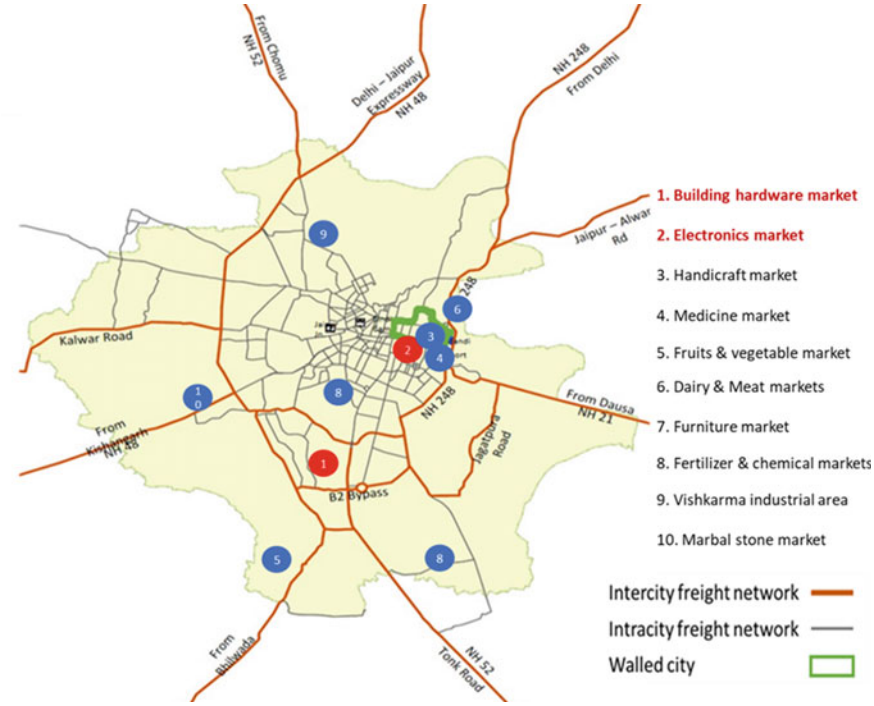

Fig. 3 Location of wholesale markets

hardware market is a planned market recently developed by the Jaipur development authority with all required facilities in the periphery of the city on the national highway.

5 AHP and BWM Methods

To better comprehend the fundamental decision-making processes, multi-criteria Analytical Hierarchical Process (AHP) is useful. The AHP method is useful when there is the potential for conflicting expert opinions [33]. AHP is an extensively utilized mathematical tool, particularly where subjectivity could affect how a choice is made [34]. For paired comparison, a nine-point scale is used. According to the relative relevance of each option, each point on this scale is assigned. Point 1 on the scale denotes equal significance when comparing both options (A and B). When later alternative (B) is marginally preferable to the first choice then scale point 3 is assigned. When the latter alternative (B) is significantly more favourable than the first alternative, then scale point 5 is assigned. Point scale 7 is assigned, where the latter alternative (B) is strongly chosen due to its perceived superiority. Point 9 on the scale was reached when a subsequent decision (B) was confirmed in the highest order achievable. Even scales (2, 4, 6, and 8) are used to compromise between successive odd scales. If option (A) is the preferred choice compared to the alternative (B), then a reverse scale is to be used (1/3, 1/5, 1/7, 1/9) [35].

Results from AHP are shown in a ratio scale for each criterion. Each alternative's weight is proportional to the hierarchy's overall score for all of its criteria [36]. The results of the pairwise comparison on the 'n' criteria can be summarized. This pairwise comparison can be represented using a square matrix. The comparison matrix is normalized in the final phase of AHP to obtain relative weights. The final check for consistency of results in AHP is denoted by consistency ratio (C.R.). A C.R. check's acceptable upper limit is 0.1. (10%).

The Best–Worst Method (BWM) is a multi-criteria, vector-based decision-making method. BWM also do the pairwise comparison of alternatives, such as AHP. A set of selection criteria must be chosen for comparison before the selection maker may determine the best and worst criteria without comparing them, which is how a BWM analysis is performed which is a significant difference compared to the AHP method. The best criteria are then compared against other criteria and similarly, the worst criteria are compared with other criteria. The ideal criterion weights are determined in the final stage. On a scale of 1–9, alternatives are compared, with 1 denoting equal choice and 9 denoting that something is nine times more important. Comparisons with higher consistency ratios are less reliable. BWM produces more reliable results since it requires fewer comparison data (alternatives) and more consistent comparisons. BWM is interchangeable with other MCDM approaches. BWM can replace multi-criteria decision-making techniques, which are more comfortable [13].

6 Results and Discussion

AHP and BWM techniques were used to evaluate five planning and consolidation strategies for the city of Jaipur for two wholesale markets. Table 2 shows the results of freight strategies weights obtained by AHP and BWM techniques for individual markets. All results confirm the consistency ratio of less than 10%, and the final weights are the geometric mean of all the results from each stakeholder.

The outcomes of both techniques show that there is variation in freight strategy weights within and across wholesale markets. In the electronic market, the weights of strategies vary by both AHP and BWM techniques, but the ranking of strategies remains consistent.

6.1 Electronics Market

With both methods, the urban logistics spaces strategy is the most desired in the electronics wholesale market. Loading–unloading facilities and urban consolidation

Table 2 Weights of planning and consolidation strategies

Strategies	Electronics market		Building hardware market	
	AHP weights	BWM weights	AHP weights	BWM weights
Loading/unloading	20%	21%	22%	24%
Design enforcement for on-street loading bays	14%	11%	13%	10%
Urban logistics spaces	27%	29%	32%	35%
Network pick-up points	18%	17%	14%	15%
Urban consolidation spaces	21%	22%	19%	16%

spaces have nearly identical weights but are more crucial in the electronics market. The Jaipur electronics market is located in a densely populated walled city with no scope for expansion with limited parking and logistics amenities. This explains why urban logistics spaces (27–29%) are preferred over urban consolidation spaces and loading–unloading strategies. In the electronic market, the network pick-up points strategy (18–17%) is second to least preferred by wholesalers. Electronics market wholesalers receive their goods either at late night or early in the morning on the outskirts of the walled city. Wholesalers utilize small 4-wheel commercial vehicles to transport items to their different warehouses. There are network pick-up points, but not in the form of formally recognized locations by competent agencies. A maximum weight variation of (3%) by both AHP and BWM techniques is observed for enforcement of the on-street loading bays strategy. A (2%) variation in weights is observed for the urban logistics spaces strategy. The rest of the strategies have only (1%) variation in their weights by both methods.

6.2 Building Hardware Market

Variations in weights of various planning and consolidation strategies are also observed in the building hardware market using AHP and BWM techniques. Among all other strategies, the urban logistics space strategy has the highest weight (32–35%) by both methods, followed by the loading/unloading facilities strategy (22–24%) and urban consolidation spaces strategy (19–16%).

The design guide/enforcement strategy for on-street loading bays is the least preferred (13–10%) in the building market. Parking and loading/unloading are not a problem because there are enough parking spaces for freight vehicles in the building hardware market. Apart from time constraints on freight vehicles, there is little policy direction from local authorities in terms of distribution and delivery. There is no specialized freight parking demarcation in terms of time and space for loading and unloading in the city in general, this is why the design guide/enforcement for the on-street loading bay strategy is given the lowest priority. The second least recommended strategy is network pickup locations (14–15%). The network pick-up point strategy is only feasible in a walled city as there is significant space constraint for transporting goods.

The AHP and BWM approaches differ by the greatest percentage (3%) in the weights of strategies for urban logistics spaces, urban consolidation spaces strategy, and design guidelines/enforcement for on-street loading bays strategy. The loading–unloading facility strategy differs by 2% in weight, and the network pick-up point strategy differs by 1% in weight by both methods.

6.3 Weights Comparison Across Markets

A cross-market analysis of strategies finds that the urban logistics space strategy is the most preferred, but the building hardware industry has higher weights associated with it than the electronics market. In both wholesale marketplaces, the design guide/enforcement for the on-street loading bay strategy is least desired. In the building hardware market, the loading–unloading facilities strategy is slightly more favoured than in the electronics market. In the electronics sector, network pick-up points and urban consolidation space strategies have more weight and preference compared to building hardware markets.

6.4 Statistical Test for Weight Comparison

To examine the similarities of the obtained rankings/weights, Kendall's tau-b and Spearman's rho were utilized to analyse the ranks produced using the AHP and BWM techniques. Kendall's correlation coefficient for identical produced ranks is 1, whereas Kendall's correlation coefficient for fully differentiated produced ranks (totally dissimilar) is 0. Spearman's rank correlation coefficient is a monotonic function that describes the strength of the linear association between the ranks that are produced. Kendall's tau-b and Spearman's rho tests yield comparable results, according to the statistical analysis presented in Tables 3 and 4. The *p*-values for both statistical tests are less than 0.05, indicating that there is a statistically significant connection between the obtained ranks of the two contrasting methods. Both AHP and BWM techniques predict similar outcomes.

7 Conclusion

This research paper evaluates the planning and consolidation freight strategies weights for building the hardware market and electronics market in Jaipur city. The study results confirm that the weights of planning and consolidation freight strategies

Table 3 Correlation of ranks by BWM method

		Correlations	BWM_BH	BWM_EM
Kendall's tau_b	BWM_BH	Correlation Coefficient	1.000	0.800
		Sig. (2-tailed)		0.50
		N	5	5
	BWM_EM	Correlation Coefficient	0.800	0.800
		Sig. (2-tailed)	0.050	0.0
		N	5	5
Spearman's rho	BWM_BH	Correlation Coefficient	1.000	0.900
		Sig. (2-tailed)		0.37
		N	5	5
	BWM_EM	Correlation Coefficient	0.900	1.000
		Sig. (2-tailed)	0.037	
		N	5	5

*Correlation is significant at the 0.05 level (2-tailed)

Table 4 Correlation of ranks by AHP method

		Correlations	AHP_BH	AHP_EM
Kendall's tau_b	AHP_BH	Correlation coefficient	1.000	0.800
		Sig. (2-tailed)		0.50
		N	5	5
	AHP_EM	Correlation coefficient	0.800	0.800
		Sig. (2-tailed)	0.050	0.0
		N	5	5
Spearman's rho	AHP_BH	Correlation Coefficient	1.000	0.900
		Sig. (2-tailed)		0.37
		N	5	5
	AHP_EM	Correlation Coefficient	0.900	1.000
		Sig. (2-tailed)	0.037	
		N	5	5

*Correlation is significant at the 0.05 level (2-tailed)

are similar across both wholesale markets by using AHP and BWM techniques. In both markets, the design guide/enforcement strategy for on-street loading bay strategies is least preferred, while the urban logistics space strategy is most preferred. The weights and rankings for freight planning and consolidation strategies vary across and within both wholesale marketplaces. Both AHP and BWM approach consistently rank strategies inside individual wholesale markets, although with different weights. The selection of suitable urban freight strategy must be in such a manner that a particular strategy can have a similar impact on multiple commodity distributions as well as strategies for specific commodity distributions within a city. When compared to the AHP technique, BWM weighing scales are easier to convey to stakeholders. There is a further need for research to analyse the weights of planning and consolidation strategies for other freight handling wholesale markets with various sizes of cities in India for the transferability of results.

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Evaluating the Parking Characteristics and Parking Demand of On-Street Parking in Silchar City

Prasenjit Das, Ashwani Bokadia, and Mokaddes Ali Ahmed

Abstract Silchar is a major city in Cachar district, Assam with a population of 2,72,709 and an area of 26.88 km^2. Parking is an issue that creates high traffic congestion in city areas. The vehicle needs sufficient street space for parking for the smooth movement of traffic. With the expanding population and automobiles, parking has become a significant issue (Manville, Shoup, Bacon, 2005). The scarcity of parking space in the city has increased the demand for it, particularly in critical business areas. A thorough examination of parking demand and characteristics is carried out to control parking activities, which will be helpful to the site visitors, engineers, and town planners. Two roads have been chosen for this study, i.e., Central Road and Shyamaprasad Road, which have insufficient parking spaces. Surveys are being conducted at those two locations to determine the parking accumulation, occupancy profile, demand and supply of parking (Parmar, Das, Dave, 2019; Chen, Wang, Pan, 2015; Tong, Wong, Leung, 2004). Linear regression analysis is also used to build a parking demand model using the statistical package for the social science (SPSS) software. The R-square values for Central and Shyamaprasad Roads are found to be 0.97 and 0.98, respectively.

Keywords On-street parking · Parking accumulation · Parking occupancy · Parking demand

1 Introduction

Managing on-street parking in an urban area is difficult for a transportation planner [1–3]. A city's transportation system includes traffic mobility and good planning to allow a vehicle to stop at the desired spot [4]. Vehicles must stop at the specified place for people to alight or embark and for loading and unloading products. Parking temporarily stops a vehicle at a particular area until the user's purpose is completed. It is a necessary component of the transportation system.

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A gradual growth in the number of vehicles owned per family, as well as a preference for parking vehicles close to destinations, is boosting on-street parking demand in the Central Business District (CBD), which is reaching a tipping point. The number of vehicles intending to park is much higher than the available parking space, resulting in a mismatch between the parking demand and supply [5, 6]. The search time is increasing with the increased parking demand, which is increasing the unnecessary movement of the vehicle to find a parking space.

Because of on-street parking, the main highway width is reduced, reducing flow and increasing congestion and unwelcome delays in traffic movement [7]. If on-street parking demand is not immediately reduced, it may reach an irreparable point.

1.1 Need of On-Street Parking System

Smart on-street parking is an essential part of smart mobility and smart city concepts [8, 9]. It helps to reduce congestion, air pollution, and other aspects such as

- **Higher performance:** Users of the downtowns always select on-street parking areas over off-street surface lots and garage parking. The on-street areas enjoy the maximum use and the very best turnover.
- **Better land use:** Medium-sized city facilities can keep a mean of greater than acres of land by supplying street parking. This performance can permit a lot higher density for business improvement than the middle, depending totally on off-street floor plenty.
- **Increased safety:** Drivers generally tend to tour at appreciably slower speeds within the presence of capabilities along with on-street parking and minor construction setbacks. Slower automobile speeds offer pedestrians, cyclists, and drivers more time to react, and when a crash occurs, its life-threatening hazard is substantially reduced. The negative impact of on-street parking on traffic safety is as follows.
- Effects on road capacity: On-street parking reduces road capacity mainly in two ways. Firstly, it narrows the carriageway width by bordering the traffic stream. Secondly, continuous parking and unparking create congestion on the busy urban road.
- Effects on road user safety: On-street parking creates hazards and puts drivers at risk. The most dangerous and frequently observed accident prototypes are the collision with other vehicles while darting out between two parked cars and the crash while backing into the stream from the parking space.

The objectives of the study are:

- To assess the parking demand at selected area of the Silchar city.
- To determine the parking characteristics like parking Occupancy and accumulation.

2 Selection of Study Area

Following two locations—Central Road and Shyamaprasad Road in Silchar have been selected as the case study area.

3 Methodology

Four types of surveys were carried out to find out parking characteristics. Parking survey was carried out for 9 h including peak and non-peak hours (i.e.9.00AM to 6.00PM) at both the locations from December 17, 2021 to January 17, 2022. The methodology flow chart has been shown in Fig. 1. The various surveys conducted for the study are discussed below.

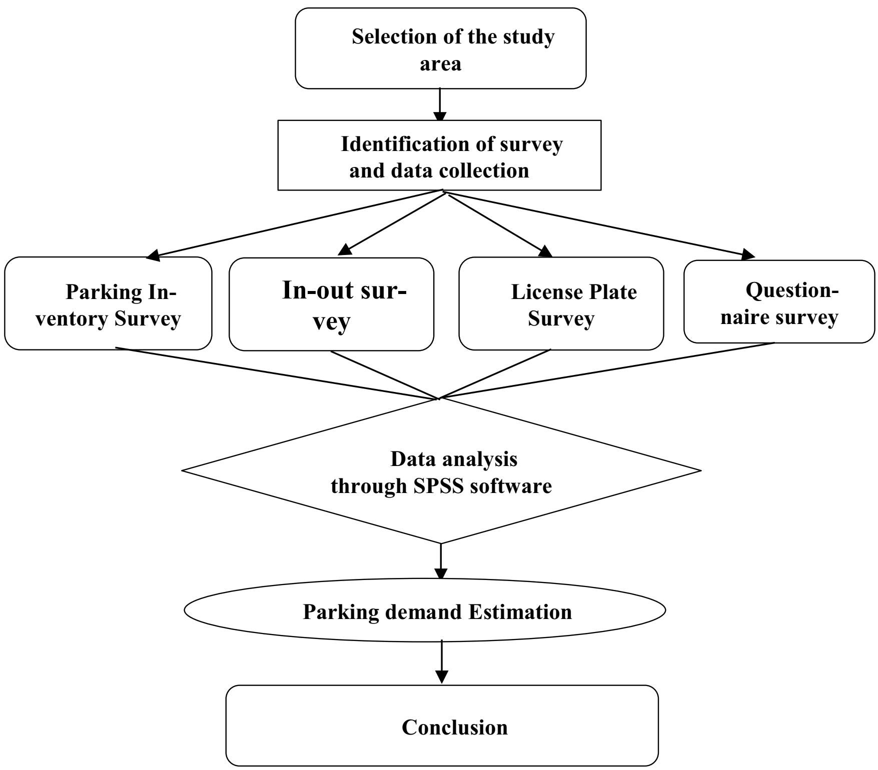

Fig. 1 Flow chart showing the methodology of the current study

3.1 Parking Inventory Survey

This survey is performed to find out the number of parking bays available in the study location.

3.2 In–Out Survey

This survey determines the various parking determinants like parking load, efficiency, accumulation, peak parking saturation, and parking ratio.

3.3 License Plate Survey

A license plate survey is carried out by taking note of the registration numbers of each vehicle entering and exiting the parking place [10].

3.4 Questionnaire Survey

Questionnaire surveys have been conducted to know the personal information of the respondent, their trip characteristics and parking characteristics. Total of 200 sample size have been taken for this survey.

4 Result and Discussion

4.1 Parking Accumulation and Occupancy Profile

Parking accumulation is defined as the number of parked automobiles at any particular time, and Parking occupancy is defined as the ratio of the number of bays occupied in a specific period to the total available space [11, 12].

The parking statistics were gathered from the in–out and license plate surveys. The accumulation curve (Fig. 2) and occupancy curve (Fig. 3) show that the number of parked vehicles steadily climbs at the start of the day, reaches a peak, and then falls.

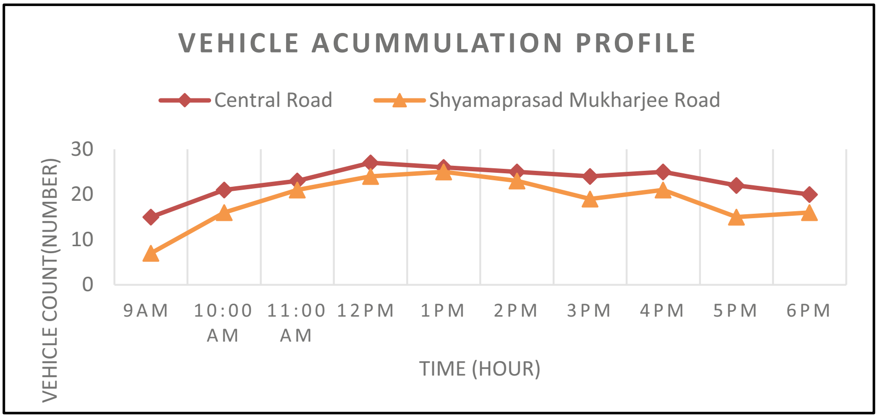

Fig. 2 Vehicle accumulation curve

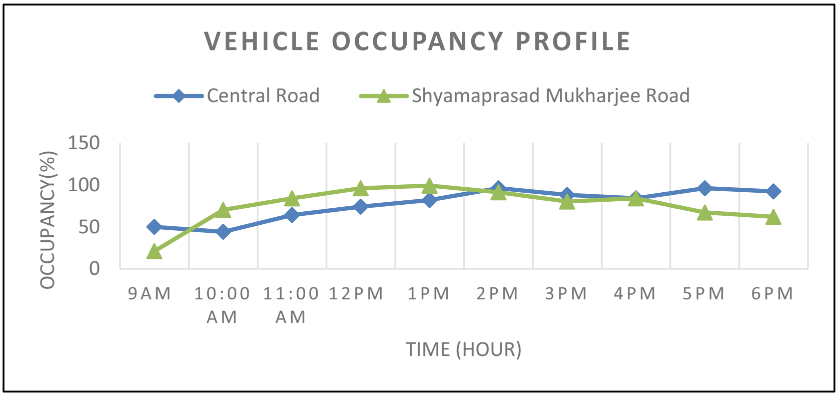

Fig. 3 Vehicle occupancy curve

4.2 Development of Parking Demand Model

Parking demand models have been developed for both areas with the help of linear regression analysis. SPSS was used to create models. Demand for parking is taken as the dependent variable in the model. As independent variables, parking duration, car ownership, income, frequency of visit, maximum ease time, and maximum search time have been taken and generated a demand equation (Eq. 1).

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 X_4 + \beta_5 X_5 + \beta_6 X_6 + \mu \tag{1}$$

Y = Parking Demand

β_0 = The Intercept

X_1 = Car owned

X_2 = Parking duration

X_3 = Income

X_4 = Frequency of visit (FOV)
X5 = Average ease time (AET)

X_6 = Maximum search time (MST)

$\beta_1, \beta_2, \beta_3, \beta_4, \beta_5, \beta_6, \beta_7, \beta_8, \beta_9$ = Regression Coefficients

μ = error term

For both roads, Pearson correlation has been found to check the correlation between the variables, shown in Tables 1 and 3.

Tables 2 and 4 show the coefficient, which helps to get the value of R-square and find out the parking demand.

Table 1 Pearson correlations of Shyamaprasad road

	Y	X_1	X_2	X_3	X_4	X_5	X_6
Y	1						
X_1	0.578	1					
X_2	0.300	−0.117	1				
X_3	0.206	0.664	−0.077	1			
X_4	0.096	0.167	−0.408	0.221	1		
X_5	0.365	0.160	−0.086	0.260	0.077	1	
X_6	0.798	0.382	0.145	0.313	0.097	0.832	1

Table 2 Coefficient of shyamaprasad road

Model		Unstandardized coefficients		Standardized coefficients	T	Sig.
		B	Std. Error	Beta		
1	Constant	−13.672	9.192		−1.487	0.234
	Car owned	8.967	2.797	0.370	3.205	0.049
	Parking duration	1.573	1.611	0.093	0.976	0.401
	Income	−3.506	1.321	−0.273	−2.655	0.077
	FOV	1.446	2.018	0.060	0.717	0.525
	AET	−17.236	3.937	−0.721	−4.378	0.022
	MST	31.209	4.184	1.323	7.459	0.005
Dependent variable: Parking Demand						

Table 3 Pearson correlations of central road

	Y	X_1	X_2	X_3	X_4	X_5	X_6
Y	1						
X_1	0.578	1					
X_2	0.300	−0.117	1				
X_3	0.206	0.664	−0.077	1			
X_4	0.096	0.167	−0.408	0.221	1		
X_5	0.353	0.158	0.068	0.094	0.066	1	
X_6	0.558	0.105	0.011	0.321	0.226	0.653	1

Table 4 Coefficients of central road

Model		Unstandardized coefficient		Standardized coefficient	t	Sig
		B	Std. error	Beta		
1	Constant	−35.397	12.239		−2.892	0.063
	Car owned	26.113	3.496	1.079	7.470	0.005
	Parking duration	6.518	1.877	0.385	3.472	0.040
	Income	−9.566	1.939	0.744	−4.932	0.016
	FOV	0.114	2.897	0.005	0.039	0.971
	AET	−9.778	3.847	0.376	−2.541	0.085
	MST	21.653	3.711	0.924	5.834	0.010
Dependent variable: Parking Demand						

4.3 Discussion

The R-square value for Central Road and Shyamaprasad Road is 0.97 and 0.98, respectively. On the Central Road, the frequency of visits and average ease time does not show significant value. Parking duration, income and frequency of visit do not show significant value in the case of Shyamaprasad Road. Variables having a significance value equal to or less than 0.05 are considered in the parking demand equation i.e., in Eq. 1.

Parking demand for Central Road is,

$$Y = -35.397 + 26.113X_1 + 6.518\,X_2 - 9.566\,X_3 + 21.653X_6$$

Parking demand for Shyamaprasad Road is,

$$Y = -13.672 + 8.967\,X_1 - 17.236\,X_5 + 31.209\,X_6$$

5 Conclusions

Peak hour on-street parking demand was found to be most uniform from 9 a.m. to 6 p.m. in both the study locations. Because of the diversified land uses, accumulation and occupancy are more or less constant throughout the day (offices and shopping). The average occupancy rate is around 75%. Central Road is mainly a shopping district.

1. Maximum accumulation for Central and Shyamaprasad Road is 26 and 24 and occupancy is 97% and 96% respectively.
2. In parking demand model R-square value shows good result and has a value of 0.97 and 0.98 for Central and Shyamaprasad Roads respectively.

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