Lecture Notes in Civil Engineering

Shriniwas S. Arkatkar
S. Velmurugan
Ashish Verma *Editors*

Recent Advances in Traffic Engineering

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Conference Tracks

The conference, named Recent Advances in Traffic Engineering (RATE), was held during 11–12 August 2018. This book is a collection of the contribution of research articles and deliberations along three tracks of **Traffic Engineering (Track I), Sustainable Transport (Track II) and Road Safety (Track III)**. Several inter-disciplinary themes are covered in each of these tracks keeping comprehensiveness of the field of transportation encompassing traffic flow modelling, simulation studies, pedestrian and vehicular safety, green technologies and innovations in data retrieval. The track for works related to themes, which are not mentioned directly under various tracks, was decided suitably according to the focus and objectives of the work. The details of the tracks are as follows:

Track I: Traffic Engineering: This track deals with the topics related to operations and management of traffic systems and multiclass & multi-category users. Human factor analysis, traffic flow behaviour, travel time reliability for transit operations, corridor and area traffic coordination systems, user-perceived level of service, inclusive design of intersections & transfer stations' real-time traffic data collection and analysis and vehicle dynamics are covered under this theme. The track would also consider research outcomes based on simulation studies on roadway operations, toll plaza operations, crowd management as well as intercity & urban freight movements.

Track II: Sustainable Transport: This track deals with the topics related to approach and practices towards achieving sustainability of decisions related to transport system improvement and enhancement. Issues related (but not limited to) to economic policy & planning, transport project evaluations in the multimodal environment, transport impacts, public transport planning and strategies for inclusive mobility including NMT, socio-economic impacts of transport projects and smart mobility options are the broad themes expected to be addressed by this track. Research outcomes related to short-, medium- and long-term transport climate change resilient policy options and case studies demonstrating path towards sustainable development goals are invited under this track.

Track III: Road Safety: This track deals with the topics related to approach and practices towards achieving a goal of safety on roads. Topics related to the study of safety performance indicators, safety measurement & programmes, pre- and post-crash studies, vehicle and pedestrian crash analysis and social cost involved with all of these are considered under this track.

Contents

About the Editors

Dr. Shriniwas S. Arkatkar is currently working as an Associate Professor in the Department of Civil Engineering at SVNIT Surat. Prior to joining SVNIT Surat, he has worked in the Department of Civil Engineering at BITS Pilani, Rajasthan. He has more than 10 years of experience in teaching, research, and consultancy in the field of traffic and transportation engineering. He obtained his Ph.D. from the Department of Civil Engineering, IIT Madras. He completed his bachelors in Civil Engineering in 1999 and masters in the area of Urban Planning in 2001, from Visvesvaraya National Institute of Technology (VNIT), Nagpur, India. Dr. Arkatkar has published more than 120 research papers in journals and conference proceedings. He has hosted a special issue as Guest Editor in Transportation Letters; The International Journal of Transportation Research. He has experience in diverse fields of transportation. They include: traffic flow modelling, traffic safety, intelligent transportation systems (ITS), transportation planning, traffic operations, and traffic simulation applications. He is actively involved as Executive Secretary, Transportation Research Group of India (TRG) and as member of SIGs of WCTRS, Indian Roads Congress (IRC), Governing Council member in the Institute of Urban Transport (IUT), MoUD.

Dr. S. Velmurugan is working as a Chief Scientist at CSIR - Central Road Research Institute (CRRI), New Delhi and is also the Former Head of the Traffic Engineering and Safety (TES) Division of CSIR-CRRI. He has completed his Ph.D. in Transportation Systems Engineering at IIT, Bombay in March, 1995 and he has also pursued two post-doctoral stints at the University of Antwerp, Belgium and the University of South Australia, Adelaide. He has served in the capacity of Team Leader in the World Bank sponsored projects as well as National Highways Authority of India and various state Public Works Departments (PWD) for the conduct of Road Safety Audits spanning over 4500 kms which include Urban Expressways, National Highways, State Highways, Major District Roads and Urban Roads. He has published more than 100 technical papers in international and national journals and presented several research papers in various scientific forums. He is the recipient of Outstanding Scientist Medal in March, 2015 instituted by the erstwhile Planning Commission, Government of India along with Construction Development Council (CIDC). Presently he is serving as the Co-Convener of Road Design and Safety Committee i.e. H-7 Committee of Indian Roads Congress (IRC). He was instrumental in the formulation / revision of 15 IRC publications during the last 12 years.

Dr. Ashish Verma obtained his Ph.D. from IIT Bombay and is currently serving as Associate Professor of Transportation Engineering at Dept. of Civil Engineering, Centre for infrastructure, Sustainable Transportation, and Urban Planning (CiSTUP), and Robert Bosch Centre for Cyber Physical Systems (RBCCPS) at Indian Institute of Science (IISc), Bangalore, India. He was a Visiting Professor at ITMO University, Saint Petersberg, Russia during 2016. His research interests include sustainable transportation planning, integrated public transport planning and management, modelling and optimization of transportation systems, travel behaviour, driver behaviour and road safety, intelligent transportation system (ITS), traffic management. He has authored more than 120 research publications in the area of sustainable transportation and road safety. He has also authored a book on "Public Transport Planning and Management in Developing Countries" published by CRC Press and another book on "Integrated Public Transportation System – Planning and Modelling". He is the founding and current President of the society Transportation Research Group of India (TRG). He is presently serving as Vice Chair (Conference) of Scientific Committee, and Steering Committee Member of World Conference on Transport Research Society (WCTRS) based in the University of Leeds, UK.

Advances in Traffic Engineering

Traffic Flow Modelling for Congested Urban Road Links of Ahmedabad City

A. A. Amaliyar, B. S. Patel, and H. R. Varia

Abstract Knowledge of fundamental traffic flow characteristics and vehicle behaviour are necessary for appropriate operation of system. In developing countries like India, the urban road traffic, in particular, is highly heterogeneous. It comprises of vehicles of widely varying static and dynamic characteristics. Heterogeneous traffic flow creates enormous delay, wastage of fuel, air and noise pollution, accidents and interruption to emergency vehicles. The fundamental parameters of road traffic flow are speed, flow and density. If these parameters are measured properly on congested urban road links, then mathematical models can be developed. These models are basic need for quantifying capacity and consequently determining level of service of the road. Models for the free flow conditions are available for the congested Indian urban roads, but very few attempts have been made for developing model for the forced flow conditions. It is difficult to capture stop-and-go condition for developing the speed–flow relationship from the field observations. Hence, this study aimed to develop an appropriate methodology to collect the data for the stop-and-go condition particularly. The congested road links of Kalupur area of Ahmedabad city have been selected, where heterogeneous traffic is flowing creating enormous delay about more the one hour in the evening peak period. The data is collected by videography and then analysed for small time interval of 20–30 s. The speed–flow–density curves are plotted for free flow as well as for forced flow conditions, and different equations are obtained.

Keywords Heterogeneous traffic · Speed–flow–density · Congestion · Level of service

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1 Introduction

In developing country like India, road traffic on urban roads is highly heterogeneous which includes vehicles of extensively varying static and dynamic nature. The vehicles are using the same road space without segregation. Basic science of traffic flow tendency like speed, traffic volume and density under heterogeneous conditions is necessary to tackle the traffic operation problems. The traffic volume is very useful for planning, designing and operation. The roads of India are a perfect example of heterogeneous traffic like two-wheelers, three-wheelers, bus, minibus, truck, car, tractor, bicycle, non-motorized can be seen using the same road space. The traffic situation of Ahmedabad is highly heterogeneous type and vehicles are not following traffic rules and regulations, which makes it challenging to study and figure out traffic flow characteristics. To understand the traffic flow characteristics, the relationships have been established between fundamental traffic flow parameters like speed, flow and density on congested urban road links.

1.1 Problems with Heterogeneous Traffic

Heterogeneous traffic flow includes fast-moving and slow-moving vehicles or motorized and non-motorized vehicles. The vehicles are different in size, control and static and dynamic nature. Traffic is not separated by vehicle type, and therefore, all vehicles travel in the same right of way. Smaller size vehicles move in a haphazard manner through any available gap between large size vehicle. In heterogeneous flow, problems are as follows:

- Wasting time of passenger and vehicle for all activities.
- Delays, which may result in late commencement of different activities like meetings, office works, education, business, emergency services or any other type of work.
- Incompetency to forecast of travel time accurately.
- Wastage of fuel so increase in air pollution and CO_2 emissions.
- Increase of stress and frustration to vehicle drivers and reduce their health.
- In emergency case, blocked traffic may interfere with the passage of emergency vehicles.

1.2 Aim of Study

The aim of the study is to develop traffic flow models (speed–flow–density relationship) for heterogeneous traffic condition on selected urban road stretches.

1.3 Objectives of the Study

1. To quantify vehicle volume count on selected stretches.
2. To conduct the spot speed study on selected stretches.
3. To obtain the speed–flow–density relationship for the free flow as well as for forced flow (including stop-and-go) condition.

1.4 Scope of the Study

This study is limited to selected stretches of Ahmadabad city. In future, this study of speed–flow–density can be used for similar type of other road links and measured volume count, spot speed study analysis and density can be used as a reference for the similar type of studies. This developed relationship may be useful for traffic assignment procedure on the similar types of links. This study is also useful for comparing pre- and post-traffic improvement conditions of the selected stretches.

2 Review of Literature

The various important technical terminologies related to traffic flow are as follows:

2.1 Flow (q)

Flow is defined as the number of vehicles that a pass a point on a highway or a given lane or direction of a highway during a specific time interval. It is defined as 'q' and expressed as 'vehicle/h'.

2.2 Speed (v)

Speed is the rate of movement of traffic or of specified components of traffic and is expressed in distance/time (km/h). It is defining as 'v'.

2.3 Density (k)

Density is defined as the number of vehicles present in a stated length of road at instant. It is defining as 'k' and expressed as 'no. of vehicles/distance (vehicle/km)'.

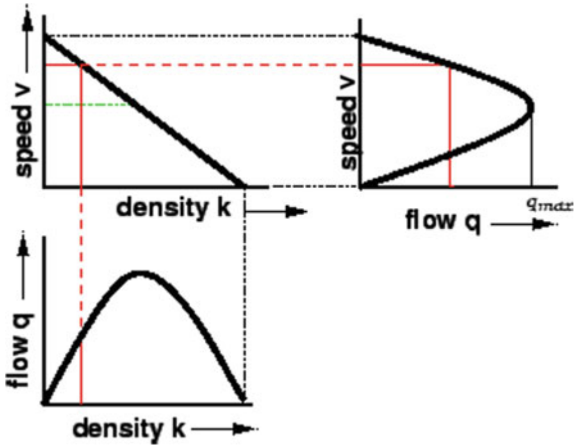


Fig. 1 Speed–flow–density relationship

The general relation between fundamental traffic parameters (q, k and v) is

$$q = k * v \tag{1}$$

where q = flow in number of vehicle/time. k = density in no. of vehicle/distance. v = speed in distance/h.

2.4 Fundamental Diagram and Models of Road Traffic

The fundamental relationship between speed, flow and density is shown in (Fig. 1). They are referred to as the fundamental diagrams of traffic flow.

2.5 Review of Past Studies

Rao and Rao (2014) have developed models for estimating free flow speed of urban arterials in Delhi. They found that factors like total vehicles, friction points, access points, number of intersection, number of flyovers and access points have significant contribution on free speed. Patel and Gor (2013) have studied speed–flow–density relationship for State Bank of India to bus station and bus station to State Bank of India road link in Modasa. They concluded that reduction in capacity is due to encroachment, on-street parking and pedestrian flow. Patel and Kumawat (2014) have studied on speed flow modelling equation on four-lane NH-8. They found that the maximum and moderate speeds of various types of vehicles are too high than city area.

Dhapudkar (2014) has studied for heterogeneous traffic of Nagpur City. He observed that present equations of traffic stream are not suitable for this type of heterogeneous traffic. Bainsa et al. (2012) have studied on modelling of traffic flow on Indian expressways in Rajasthan, India. They found that for all categories of vehicles, the PCU of a given vehicle category decreases with increase in v/c ratio. Joshi and Patel (2014) have studied on six-lane divided urban arterial road, and they concluded that the capacity of the urban arterial road greatly affected by effect of lane width, presence of non-motorize vehicles and effect of side friction. Doshi (2015) has quantified the influence of slow-moving vehicle (SMV) on density along the stretch and quantified the increase in travel time due to effect of slow-moving vehicle and developed regression model for forecasting travel speed based on vehicular composition (slow-moving, fast-moving). He found that there is sudden decrease in speed with increase in SMV composition above 40–50% and beyond.

3 Methodology

The flowchart of steps involved in this study is shown in Fig. 2.

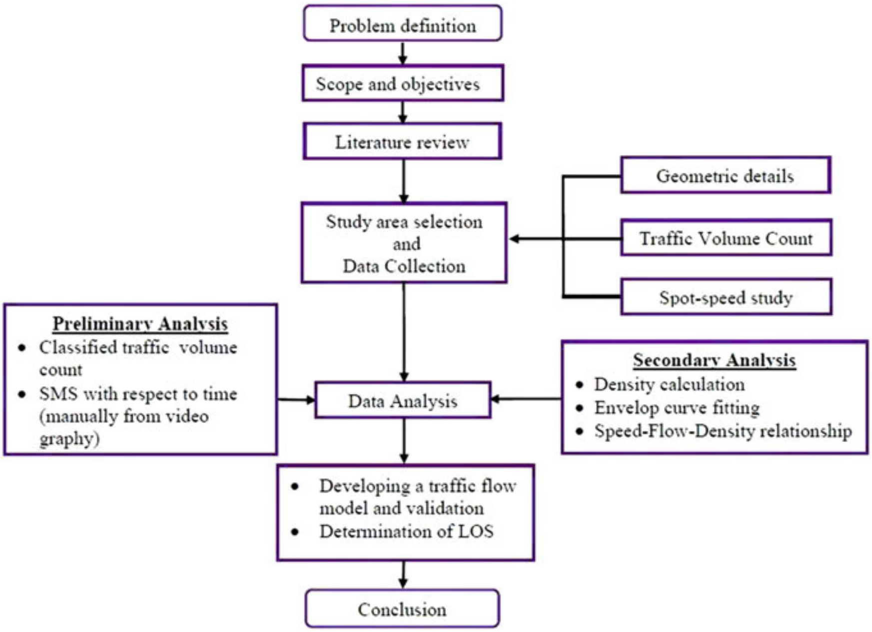


Fig. 2 Flowchart of methodology

Table 1 Geometric details of all stretches

S. No.	Name of road	No. of lanes	Divided/undivided	Type of flow	Length (m)	Width (m)
1	Sakar Bazaar to Railway Station Kalupur	3	Undivided	One way	400	12
2	Railway Station Kalupur to Sakar Bazaar	3	Undivided	One way	400	12
3	Kalupur Police Station to Gangaram Tower	2	Undivided	Two way	400	9.50
4	Vijlighar to Multistorey Parking Relief Road	2	Undivided	Two way	140	9.50

1. Sakar bazaar to Kalupur Railway Station road (3 lanes one way) having length 400 m situated in Pratapnagar colony between Pratap and company to Hotel Excel.
2. Kalupur Railway Station to Sakar bazaar (3 lanes one Way) having length 400 m between Labh Guest House-Moti bakery-Kapasiya Police Chowky.
3. Kalupur Police Station to Gangaram Tower (2 lane 2 way undivided) having stretch length about 400 m.
4. DCB Bank Relief Road to Vijlighar (2 lane 2 way undivided) having stretch length about 140 m. Table 1 shows geometric details of all stretches.

4 Study Area and Data Collection

The maps of Kalupur area selected stretches are shown in Figs. 3 and 4.

4.1 Field Survey Method

Field survey of all stretches has been carried out to collect the geometry of all selected stretches. First the permission of videography is obtained from Police Inspector of Kalupur area. After careful consideration and field observations, suitable positions of video-camera have been decided and obtained the required permissions from different buildings/shop owners. After that, the survey of the selected stretches is

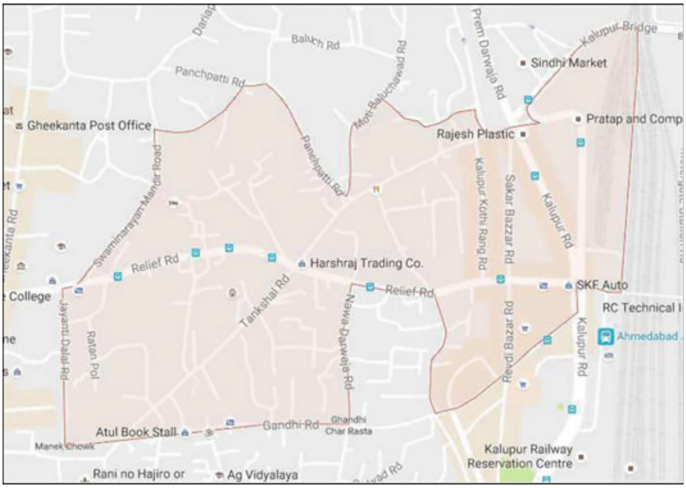


Fig. 3 Kalupur area of Ahmedabad

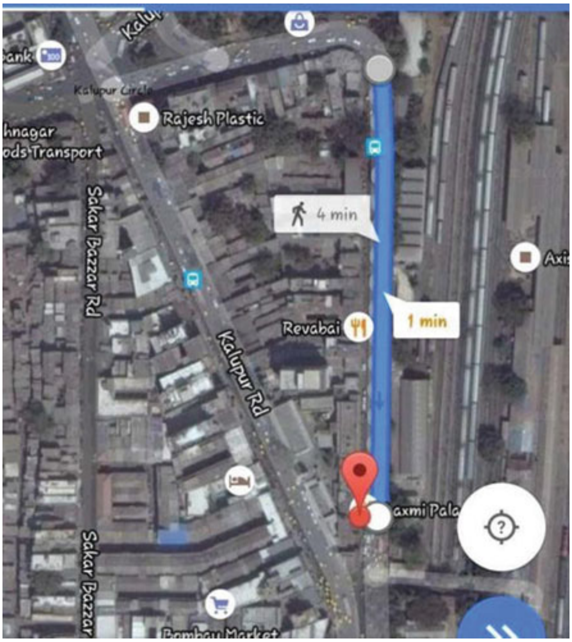


Fig. 4 Sakar bazaar to Kalupur Railway Station road

scheduled on five working days. Data was collected at 8:15:00 a.m. to 7:00:00 p.m. hours by videography.

4.2 Traffic Volume Count Survey

In this method at pre-determined location of selected stretches, the numbers of vehicles counted and recorded in excel sheet forms for the desired small time intervals (20–30 s) from the display of video recording. As per the existing average time interval of stop-and-go condition on the stretches, 20–30 s time interval is adopted. For the same time interval, spot speed observations are also made. Traffic volume count is converted in passenger car unit as per IRC-106:1990. PCU values for 2w-0.5, 3w-1.2, Car-1, Bus-2.2, LCV-1.4, NM-2 are taken.

4.3 Spot Speed Study

This survey is carried out to collect and analyse the data using video recording display marking (video counting) method. The speed of the vehicle is obtained from the time taken to cross the 12 m spacing on a selected stretches (as the maximum length is 12 m of bus). It consists of a series of observation of the individual speed at which vehicles are passing a point at a selected midblock location. These observations are used to estimate the speed distribution of the entire traffic stream at that location under the condition prevailing at the time of study. An appropriate methodology is adopted to capture stop-and-go condition particularly at selected road links.

5 Data Analysis and Results

5.1 Total Traffic Flow at Different Stretches

Traffic flow at different stretches is measured for the small time interval (20–30 s) by video recording display on laptop/computer/projector screen. Total Classified traffic Volume Count (CVC) in vehicles for the different stretches is given in Table 2.

Table 2 Total classified traffic volume for different stretches

Road name	2 W	3 W	CAR	BUS/TRUCK	L.C.V	N.M.
Sakar Bazaar to Kalupur Railway Station	22,723	17,729	4092	1363	707	1601
Railway Station Kalupur to Sakar Bazaar	21,493	14,192	4328	1280	564	1461
Kalupur Police Station to Gangaram Tower	20,080	8212	517	2	18	1000
Gangaram Tower to Kalupur Police Station	18,961	7854	540	6	14	983
Vijlighar to DCB Bank (Multi-storey Parking)	15,908	6261	426	1	15	737
DCB Bank (Multi-storey Parking) to Vijlighar Relief Road	16,775	6918	514	5	14	956

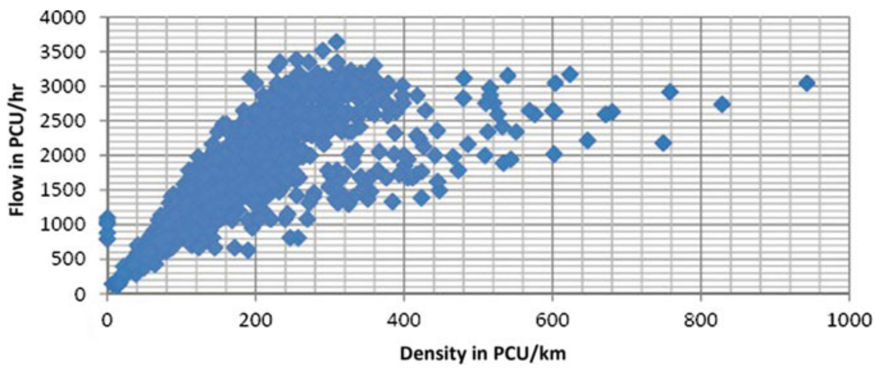


Fig. 5 Flow–density relationship at Kalupur Police Station to Gangaram Tower

5.2 *Speed–Flow–Density Graphs*

After calculation of traffic flow, space mean speed and density, the scatter graphs are generated in Microsoft Excel for all stretches. Flow–density, speed–flow, and speed–density scatter graphs are shown in Figs. 5, 6 and 7 respectively for the stretch of Kalupur Police Station to Gangaram Tower.

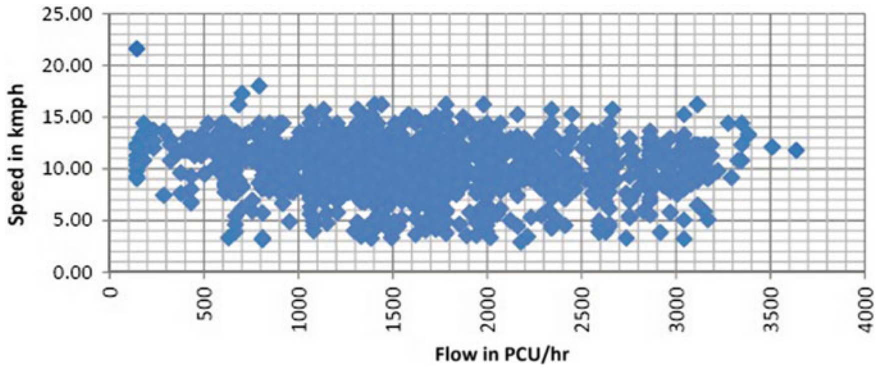


Fig. 6 Speed–Flow relationship at Kalupur Police Station to Gangaram Tower

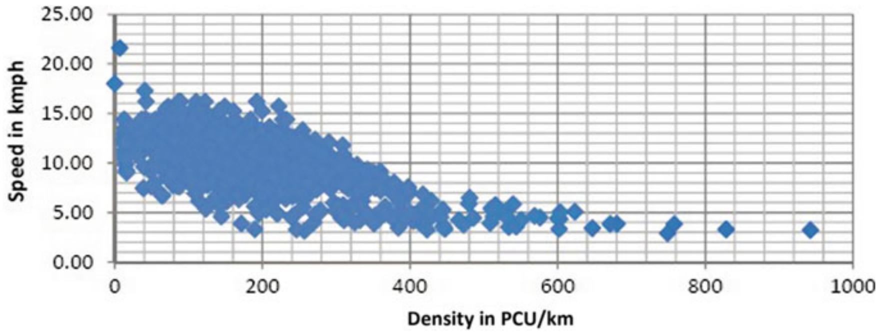
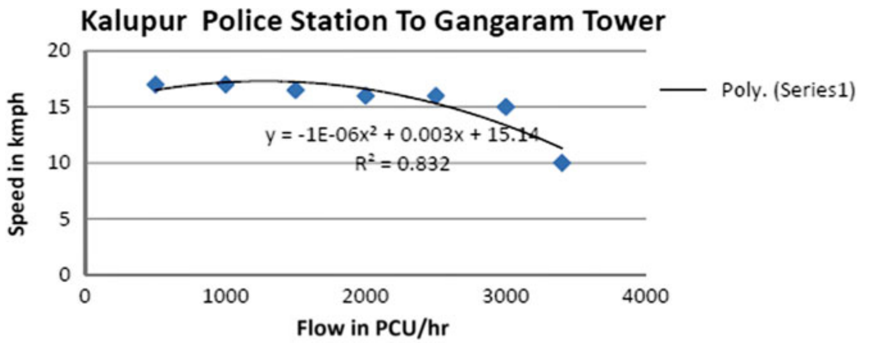


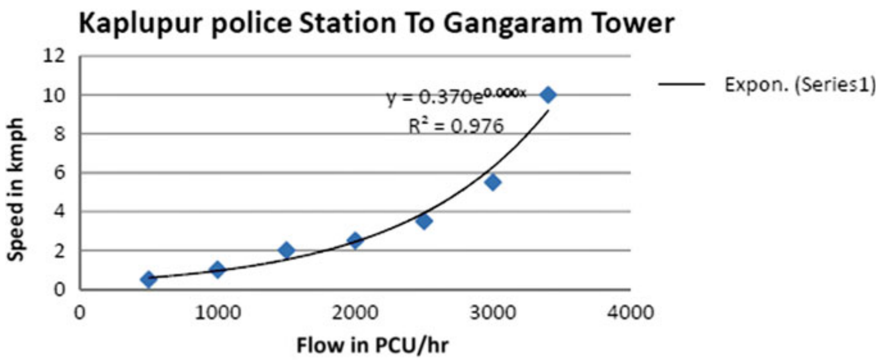
Fig. 7 Speed-Density relationship at Kalupur Police Station to Gangaram Tower

5.3 *Speed–Flow–Density Modelling*

From the speed–flow scatter graphs, curves are plotted to cover the outer points on the upper graph portion (free flow and unstable flow condition) and lower graph portion (unstable flow and forced flow condition) for different stretches and obtained speed–flow, flow–density and speed–density models using trend line function of MS Excel. For free flow and forced flow condition for the Kalupur Police Station to Gangaram Tower stretche, graphs are plotted and shown in Figs. 8, 9 and 10 with best fit trend line equations. Table 3 summarizes the developed models for all the selected stretches.



(a) Uncongested



(b) Congested

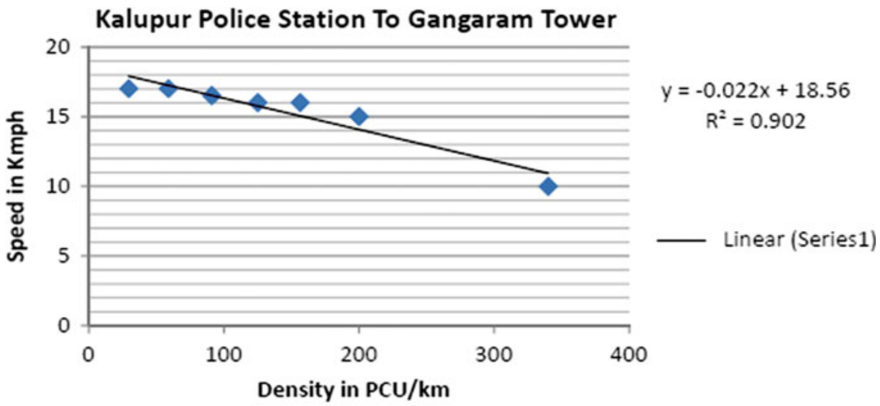
Fig. 8 **a (uncongested)** and **b (congested)** Speed–Flow model at Kalupur Police Station to Gangaram Tower

6 Volume Capacity Ratio

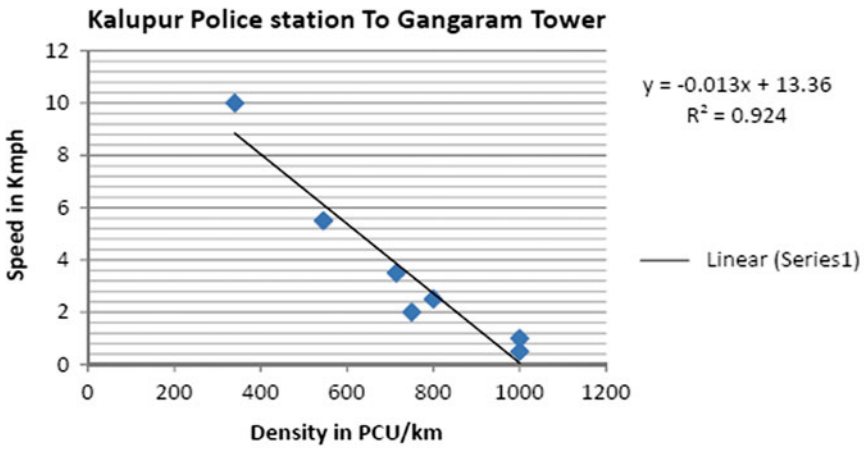
The volume by capacity (V/C) ratio is one of the most used indices to assess traffic situation, in which V is the total number of vehicles passing a point in one hour and C is the capacity of the facility. It is an indicator of the quality of the operations at an intersection.

$$V/C = \text{rate of flow/capacity}$$

V/C ratio that is greater than 1.0 shows that the facility will fail, because it is unable to discharge the demand arriving at the section. Normally, V/C value between 0.85 and 0.95 is considered for design purposes. The capacity of given stretch is considered equal to the observed maximum flow value. Volume capacity ratios of different stretches for different flow values are obtained (Tables 4, 5).



(a) Uncongested



(b) Congested

Fig. 9 **a (uncongested)** and **b (congested)** Speed–density model at Kalupur Police Station to Gangaram Tower

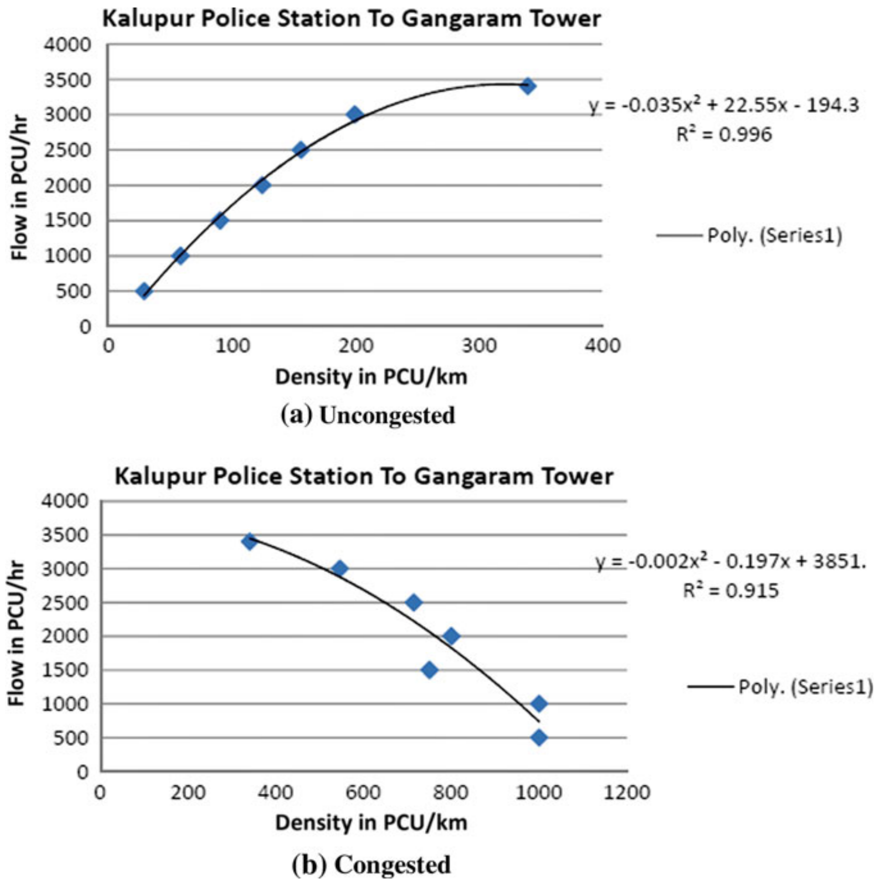


Fig. 10 **a (uncongested)** and **b (congested)** Flow–density model at Kalupur Police Station to Gangaram Tower

6.1 Level of Service (LOS)

LOS is a qualitative measure used to relate the quality of traffic service. LOS is used to analyse highways by categorizing traffic flow and assigning quality levels of traffic based on performance measure like speed, density, etc. Level of service can be found out by equal five parts (A, B, C, D and E) of maximum flow (capacity) (Fig. 11). Tables 6 and 7 show the LOS of Kalupur Police Station to Gangaram Tower and Gangaram Tower to Kalupur Police Station stretches respectively.

Table 3 Flow–density–speed models for all stretches

S. No.	Road name	Speed (y)–flow (x) model	Co-efficient of determination R^2	Speed (y)–Density (x) model	Co-efficient of Determination R^2	Flow (y)–Density (x) Model	Co-efficient of Determination R^2
1	Sakar Bazaar to Railway Station Kalupur	$y = -3E{-}07x^2 + 0.002x + 13.22$	0.926	$y = -0.008x + 19.14$	0.906	$y = -0.013x^2 + 23.65x - 678.4$	0.999
		$y = 1.222e0.000307x$	0.989	$y = -0.008x + 16.33$	0.797	$y = 0.001x^2 - 11.52x + 19{,}101$	0.756
2	Railway Station Kalupur to Sakar Bazaar	$y = -2E{-}07x^2 + 0.001x + 13.93$	0.949	$y = -0.008x + 18.96$	0.937	$y = -0.012x^2 + 22.58x - 538.9$	0.999
		$y = 0.574041306e0.000286454x$	0.986	$y = -0.007x + 17.35$	0.96	$y = -0.007x^2 + 15.96x + 923.0$	0.783
3	Kalupur Police Station to Gangaram Tower	$y = -1E{-}06x^2 + 0.003x + 15.14$	0.832	$y = -0.022x + 18.56$	0.902	$y = -0.035x^2 + 22.55x - 194.3$	0.996
		$y = 0.370e^{0.0001x}$	0.976	$y = -0.013x + 13.36$	0.924	$y = -0.002x^2 - 0.197x + 3851$	0.915
4	Gangaram Tower to Kalupur Police Station	$y = -1E{-}06x^2 + 0.004x + 13.57$	0.833	$y = -0.018x + 17.74$	0.805	$y = -0.034x^2 + 22.73x - 253.1$	0.996
		$y = 0.363e^{0.0002x}$	0.98	$y = -0.012x + 12.57$	0.883	$y = -0.0005x^{2-3}.298x + 4761$	0.888

(continued)

Table 3 (continued)

S. No.	Road name	Speed (y)–flow (x) model	Co-efficient of determination R^2	Speed (y)–Density (x) model	Co-efficient of Determination R^2	Flow (y)–Density (x) Model	Co-efficient of Determination R^2
5	Vijlighar to DCB Bank Relief Road	$y = -2E{-}06x^2 + 0.005x + 14.07$	0.87	$y = -0.023x + 18.82$	0.944	$y = -0.030x^2 + 21.67x - 149.7$	0.999
		$y = 0.3100e^{0.0009x}$	0.97	$y = -0.006x + 8.565$	0.706	$y = 0.006x^2 - 15.19x + 9911.$	0.747
6	DCB Bank Relief Road to Vijlighar	$y = -2E{-}06x^2 + 0.004x + 14.71$	0.91	$y = -0.012x + 17.53$	0.956	$y = -0.015x^2 + 18.11x - 16.64$	0.99
		$y = 0.430e^{0.0004x}$	0.946	$y = -0.007x + 7.861$	0.911	$y = -0.001x^2 - 1.274x + 3448$	0.844

Table 4 V/C ratio for Kalupur Police Station to Gangaram Tower

Flow	V/C Ratio
700	0.2
1400	0.4
2100	0.6
2800	0.8
3500	1

Table 5 V/C ratio for Gangaram Tower to Kalupur Police Station

Flow	V/C ratio
700	0.2
1400	0.4
2100	0.6
2800	0.8
3500	1

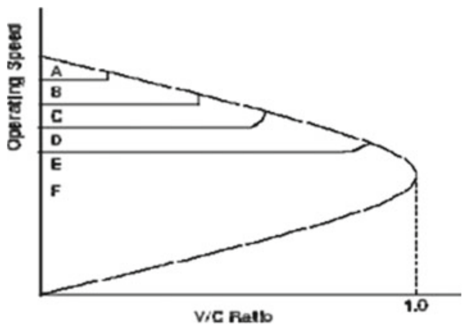


Fig. 11 Level of service

Table 6 Level of service for Kalupur Police Station to Gangaram Tower

Flow	LOS	V/C ratio
0–700	A	0–0.2
700–1400	B	0.2–0.4
1400–2100	C	0.4–0.6
2100–2800	D	0.6–0.8
2800–3500	E	0.8–1.0

Table 7 Level of service for Gangaram Tower to Kalupur Police Station

Flow	LOS	V/C ratio
0–700	A	0–0.2
700–1400	B	0.2–0.4
1400–2100	C	0.4–0.6
2100–2800	D	0.6–0.8
2800–3500	E	0.8–1.0

6.2 Level of Service as Per HCM (2000)

Level of service as per HCM (2000) is shown in Figs. 12 and 13 for stretch number 3 of Table 1 and similarly obtained for the other stretches. Table 8 shows the speed–flow–density values observed at capacity of different links.

7 Conclusion

The present study is carried out to determine the speed–flow–density relationship of selected two stretches of three-lane one-way undivided and four stretches of two-lane two-way undivided roads of Kalupur area of Ahmedabad City. Field data is collected by videography. Major findings of the study are briefed as follows:

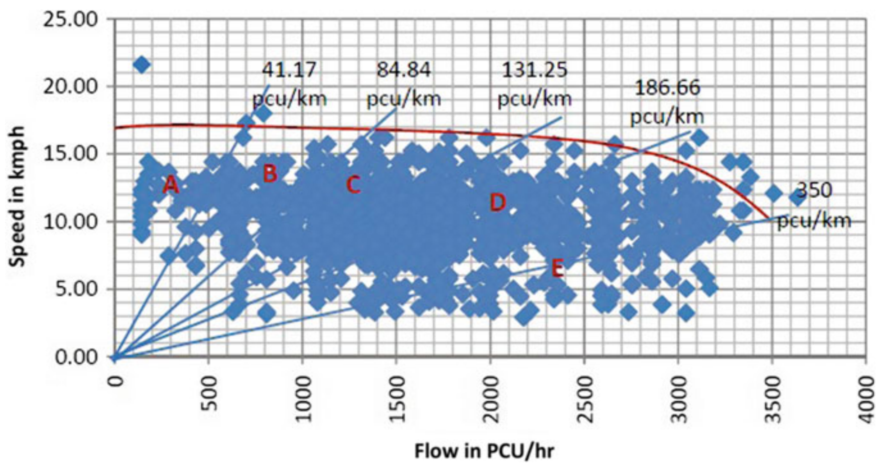


Fig. 12 LOS as per HCM 2000 for Kalupur Police Station to Gangaram Tower

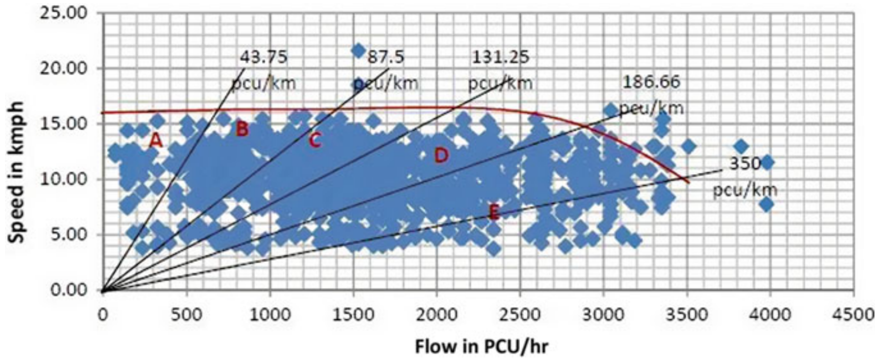


Fig. 13 LOS as per HCM 2000 for Gangaram Tower to Kalupur Police Station

Table 8 Flow–density–speed values at capacity of different links

Road name	No of lan	Width m	Max. flow (Capacity) PCU/hr	Speed @ max flow kmph	Density @ max flow PCU/km	Vmax kmph	Kjam PCU/km
Railway Station Kalupur to Sakar Bazaar	3 lane one way undivided	12	9800	10	980	17	2041
Sakar Bazaar to Railway Station Kalupur		12	9600	10	960	17	2370
Kalupur Station to Gangaram Tower	2 lane 2 way undivided	9.5	3500	10	350	17	844
Gangaram Tower to Kalupur Police Station			3500	10	438	16	985
Vijlighar to DCB Bank Relief Road		9.5	3500	8	438	17	818
Multistorey Parking Relief Road to Vijlighar			3500	8	438	17	1460

7.1 Traffic Composition

- Mix and composite traffic observed during study.
- On three-lane one-way undivided road, it is found that composition of two-wheelers is between 45 and 55%. On two-lane two-way undivided roads, it is found that composition of two-wheelers is quite high between 60 and 70%.
- On three-lane one-way undivided road, it is found that composition of three-wheelers varies between 30 and 40% and on two-lane, two-way, undivided road, it is found between 20 and 30%.
- On three-lane one-way undivided roads, it is found that composition car is between 5 and 15%, and on two-lane two-way undivided roads, it is lesser about 0–5%.
- On three-lane one-way undivided roads, it is found that light commercial vehicle and buses are more compared to two-lane two-way undivided roads. On all selected stretches, non-motorized vehicles are observed on an average 3.4%.
- For the three-lane one-way undivided road section, maximum flow rate 9800 pcu/h, maximum speed 17 kmph and jam density of 2041 pcu/km are observed.
- For the two-lane two-way undivided road section, maximum flow rate 3500 pcu/h, maximum speed 17 kmph and jam density of 1460 pcu/km are observed.
- The forced flow condition, i.e. stop-and-go condition, can be captured by collecting data for small time intervals of 20–30 s for 12 m block length.
- All the selected stretches are running at 'F' level of service during morning and evening peak hour.

7.2 Comparison of Model

- In the model of Railway Station Kalupur to Sakar bazaar, upper and lower graph error is less compared to other models. So, this model can be adopted for three-lane one-way undivided road.
- In the model of Gangaram Tower to Kalupur Police Station, upper and lower graph error is less compared to other models. So, this model can be adopted for two-lane two-way undivided road.

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Evaluation of Capacity and Level of Service for Selected Urban Arterial Roads—A Case Study of Rajkot City

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Abstract Rapid increase of transportation demand in urban roads causes delay, congestion, raise in vehicle operating cost, and safety and environmental problems. In this research, effort is made to evaluate capacity and level of service in divided and undivided mid-block sections of urban roads in peak and off-peak hours of the day. Speed and traffic volume data were collected by a video record method for 12 h. As the traffic in Indian cities is heterogeneous and has various static and dynamic characteristics, it needs to be converted into homogeneous traffic, so that we can evaluate capacity and level of service; therefore, it has been done by three methods, homogenization coefficient, PCU value as per IRC, and Chandra's method, and the result with the capacity and level of service is compared. Various speed–flow, flow–density, and speed–density relationships were made based on field data. Speed–density relationship was developed by Greenshield, Greenberg, and Underwood models, and the R^2 values were more fitted into the Underwood's model; hence, the capacity was estimated based on this model. The level of service was analyzed by DPCU/C and PCU/C for the morning and evening peak hours, and the observed level of service of the most peak hour periods of the road sections was obtained C, D, and E. Also, friction model has been developed by SPSS software, in which the input variables were BIC, V/C ratio, PSV, PEDC, and PEDSW and the output variable was the speed. It was observed that friction has a substantial effect on the speed of vehicles.

Keywords PCU · DPCU · Speed · Capacity · Level of service · Friction

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1 Introduction

Road traffic in India suffered from limited integrative roadway infrastructures and lack of operational and management experience. In urban area, population and urbanization have increased dynamically and have an impact on various infrastructures among that the road infrastructures are the most affected due to growth of vehicle and mixed traffic stream, caused traffic congestion, delay, road safety, noise and air pollution, and transport efficiency. Knowledge of road capacity is crucially important to understand traffic characteristics correctly. Roadway capacity can represent a benchmark for the utilization of existing roads, which can in turn be used to determine the current demand and to predict future road improvements based on traffic volume. Road designer and planner must consider the amount and type of vehicles and also the prediction of traffic in the future during planning and design stage. Road capacity is defined as the maximum hourly rate at which a vehicle or a person can reasonably anticipate to pass through a point or unified road or lane section during the certain period of time under the prevailing road, traffic, and control conditions, typically capacity measured by persons per hour or vehicles per hour. Weather condition, road condition, traffic control condition, and on-street parking are the most effected factors on road capacity. A design service volume is the maximum number of vehicles that can be accommodated at a specific level of service. For selection of design service volume, three main factors should be considered such as road safety, cost, and travel time; it is not advisable to plan and design the roadway width with the level of service E or F which indicates the maximum hourly flow equal or farther than roadway capacity, and the speed is very low. The DSV is designated to take into consideration an equilibrium between safety and economy. The levels of service, C and D, are usually considered for the planning and design of urban roads.

2 Objectives of the Study

The main objectives of this study are to evaluate roadway capacity and level of service in a divided and undivided two-lane and four-lane carriageway under mix traffic condition.

3 Literature Review

Patel and Joshi (2012) Traffic volume and average speed data were collected by manual and videography techniques in a 16-h period of access control six-lane divided urban road. Unobserved data were simulated by ANN model. The capacity of six-lane divided carriageway was estimated by Greenshield model, and the obtained capacity was 7450 vehicles and 2480 vehicles per lane which was reasonably realistic as compared to similar studies in India. Joseph and Nagakumar (2014) a field survey was conducted to determine and collect an average journey time, speed, and delay data, during peak and off-peak hours in six mid-block road sections in Bangalore city. The level of service was found by *v/c* ratio, and the obtained level of service during peak hour exceeds 1. The average journey time was observed in a 3.8-km stretch 17 min, and the average travel speed was observed 13 kmphs. Semeida (2013) ATS and traffic volume data were collected by a manual method on a multi-lane highway in Egypt. The capacity and level of service model are developed by artificial neural network (ANN). The input variables were taken as SA, HV, PW, LW, MW, and LC, and output variable was a density. It is analyzed for the desert road when pavement width increased, density will be decreased. For the agriculture, road density was reduced with the increase of lane width. It indicated that ANN model is more accurate than regression model for forecasting of capacity and density with R^2 values 0.992 and 0.997 for desert and agriculture road. Nokandeh et al. (2016) Speed and traffic volume data were collected by videography technique in different parts of India. Heterogeneous traffic flow is converted into an equivalent number of passenger car unit (PCU). Greenshield, Underwood, and Greenberg models were developed for the relationship of operating speed and density. The capacity of the road sections was calculated from the speed–volume curves. A polynomial equation was established between operating speed and capacity at each section. It was observed that with increase in operating speed, capacity is also increasing; with an increase in operating speed every 2 km/h, increased capacity was observed to be up to 75 PCU/h. Patel and Gor (2013) In this research, traffic flow and speed data were collected by the video record method from SBI to the bus stand and bus stand to SBI Road connected with Modasa city. The traffic flow was converted into PCU value (PCU/h). The capacity is estimated from the speed–flow relationship, which was observed as 1975 PCU/h from Malpur Cross Road to bus station and 2025 PCU/h from bus station to Malpur Cross Road. The drop of the capacity which was observed might be the effect of the encroachment and on-street parking. Pedestrian behavior might have an effect on the capacity of the road. Chand et al. (2014) Traffic flow, speed, and traffic composition data were collected by videography technique in seven sections of six-lane divided urban arterial roads in New Delhi. Mixed traffic flow converts into the equivalent number of passenger cars or PCU values. The capacity of the road was found out by Greenshield model, and the average directional capacity of the road was estimated at 6314 PCU/h. But the estimated capacity under the effect of bus stops was observed at 5745 PCU/h. It is indicated that the capacity of urban road decreased at a range of 8–13% with a presence of curbside bus stops.

4 Selection of Study Area and Traffic Data Collection

This study is carried out in four-lane and two-lane divided and undivided carriageway in Raiya and Sadhu Vaswani roads of Rajkot city. Traffic data were collected by videography in the mid-block sections of normal working days of the 12-h period to cover the wide range of traffic flow behavior and traffic condition. Moreover, morning and evening peak hours and off-peak hour period and also the vehicle composition are found. The selected road sections had uniform width, free from curvature, gradient, and bus stop.

4.1 Traffic Volume Count

The traffic volume survey was carried out by video record method during 12-h period from 8 a.m. up to 8 p.m. Traffic data were extracted from videography by 5-min interval by simple finger digital counters, and various types of vehicles were tabulated in separate columns. By analyzing 12-h flow, the morning peak hour period was 9 a.m. to 12 p.m. and the evening peak hour period was 5 p.m.to 8 p.m., and also, off-peak hour period was 12 p.m. to 5 p.m. in selected stretches. Vehicle composition in selected roads was more than 95% of the vehicles stream consisting of 2-wheeler, 3-wheeler, and passenger car. Following figures show the vehicle composition and variation during the day (Figs. 1, 2, 3 and 4).

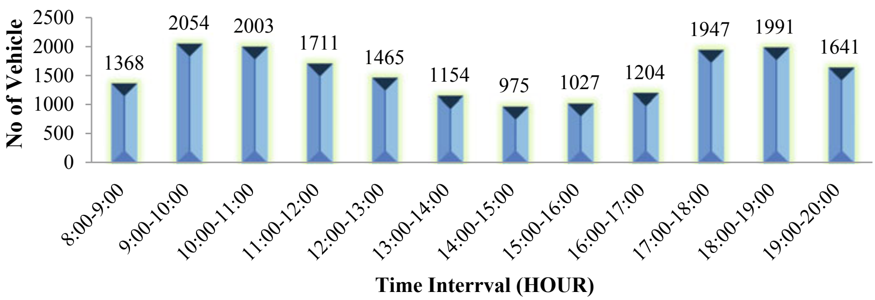


Fig. 1 Hourly variation of Sadhu Vaswani Road

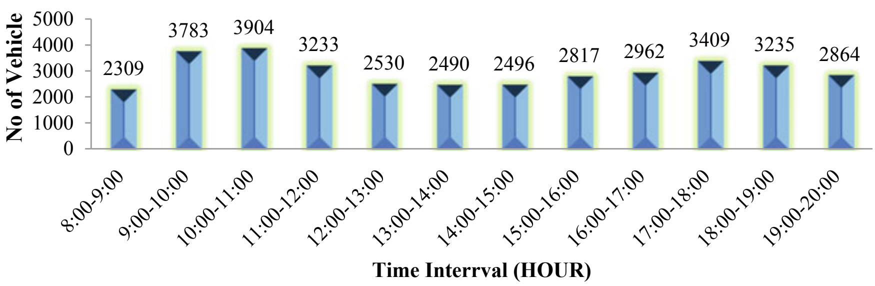


Fig. 2 Hourly variation of Raiya Road

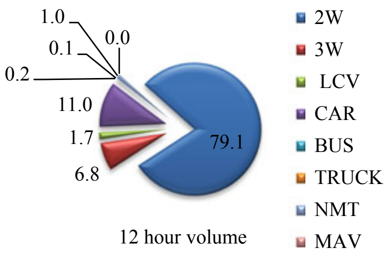


Fig. 3 Percentage of vehicle composition of Sadhu Vaswani Road

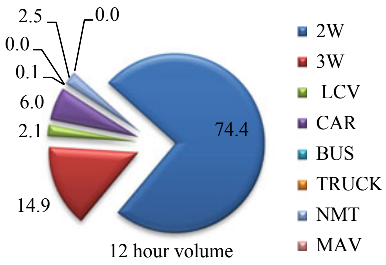


Fig. 4 Percentage of vehicle composition of Raiya Road

4.2 *Vehicle Speed Measurement*

Spot speed data were measured by direct timing technique. The trap length of 30-m road pavement was marked by white cement between two reference points. Space mean speed was analyzed by 5-min interval for various types of vehicles in the morning and evening peak hours (Figs. 5, 6, 7 and 8).

5 Estimation of Road Capacity

Traffic flow and speed data were extracted from video record method in 5-min interval, and the traffic volume converted into an equivalent number of passenger cars exercises the concept of passenger car unit or PCU and DPCU values. This

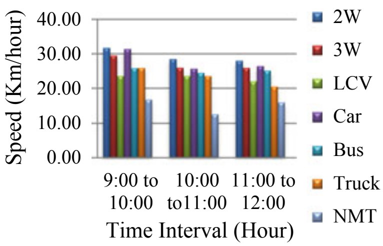


Fig. 5 Average speed (kmph) of various vehicles in the Raiya Road of morning peak hour period

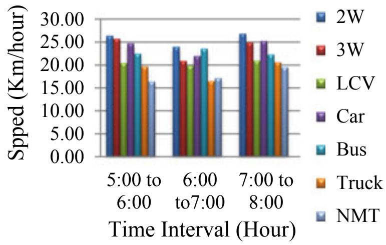


Fig. 6 Average speed (kmph) of various vehicles in the evening peak hour period of Raiya Road

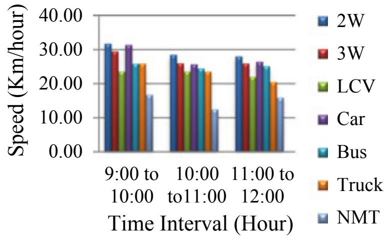


Fig. 7 Average speed (kmph) of various vehicles in the S. V. Road of morning peak hour period

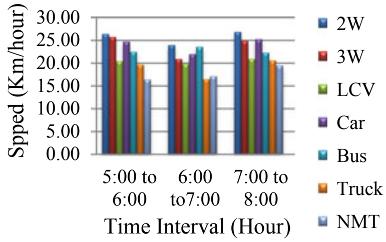


Fig. 8 Average speed (kmph) of various vehicles in the evening peak hour period of S. V. Road

homogenization was carried out by three methods. The speed–density relationship is developed by Greenshield, Greenberg, and Underwood models.

5.1 PCU Value as Per IRC-106-1990

Traffic flow is directly converted by PCU value which is suggested by IRC (Table 1).

5.2 Homogenization Coefficient Method

This method is practiced in developed countries where homogeneous traffic conditions persist and lane discipline is followed. The PCU value is calculated by the following equation.

Table 1 PCU value recommended by IRC-106-1990

S. No.	Vehicle type	5% traffic composition	10% and above traffic composition
Fast vehicles			
1	Two-wheeler motorcycle or scooter	0.5	0.75
2	Passenger car and pickup van	1	1
3	Auto rickshaw	1.2	2
4	LCV	1.4	2
5	Truck or bus	2.2	3.7
6	Agriculture tractor trailer	4	5
Slow vehicles			
7	Cycle	0.4	0.5
8	Cycle rickshaw	1.5	2

$$\text{PCU} = \frac{L_\text{i}/V_\text{i}}{L_\text{c}/V_\text{c}} \tag{1}$$

where L and V are the length and speed of a vehicle; suffix i indicates a vehicle type, and c indicates the car.

5.3 Chandra's Method

Chandra's method is the correction or adjustment of the homogenization coefficient method as a substitute length of the vehicle is replaced by the projected area of the vehicle. The PCU value is calculated by the following equation:

$$\text{PCU} = \frac{V_\text{c}/V_\text{i}}{A_\text{c}/A_\text{i}} \tag{2}$$

where V_c is the speed of passenger car and V_i is the speed of vehicle i, and A_c and A_i are their estimated rectangular areas (Tables 2 and 3).

6 Traffic Speed–Flow Model

Greenshield, Greenberg, and Underwood models were developed in the relationship of speed–flow data. The Underwood exponential model was more useful with the speed–density data with an R^2 value more than 0.7, Figs. 9 and 11 describing these relationships. Based on the derived equation of this model, speed–flow relationship

Table 2 Various vehicles standard width, length, and estimated area

Vehicle types	Width (m)	Length (m)	Area (m^2)
Small car	1.44	3.72	5.36
3-W	1.4	3.2	4.48
2-W	0.64	1.87	1.2
Big car	1.8	5	9
Minibus	2.2	6	13.2
Bus	2.6	12	31.2
LCV	2	5.17	10.34
Truck (single or dual rear axle)	2.5	7.5	18.75
MAV (more than dual rear axle)	2.5	12	30
Bicycle	0.45	1.9	0.86
Cycle rickshaw	1.22	2.84	3.46

Table 3 Comparison of PCU and DPCU values by IRC, homogenization coefficient, and Chandra's method

No	Vehicle types	DPCU[1] value based on homogenization coefficient		DPCU value based on Chandra's method		PCU[2] value of IRC-106-1990	
		Maximum	Minimum	Maximum	Minimum	5% traffic composition	10% above traffic composition
Raiya Road							
1	Two-wheeler	0.66	0.34	0.29	0.15	0.5	0.75
2	Three-wheeler	1.17	0.72	1.13	0.69	1.2	2
3	Car	1	1	1	1	1	1
4	LCV	2.55	1.22	3.1	1.81	1.4	2
5	Bus	5.15	2.93	8.96	6.81	2.2	3.7
6	Cycle	1.06	0.76	0.34	0.27	0.4	0.5
Sadhu Vaswani Road							
1	Two-wheeler	0.61	0.36	0.27	0.16	0.5	0.75
2	Three-wheeler	1.2	0.79	1.16	0.77	1.2	2
3	Car	1	1	1	1	1	1
4	LCV	2.69	1.16	3.74	1.96	1.4	2
5	Bus	3.69	3.41	6.65	6.15	2.2	3.7
6	Truck	2.85	2.16	4.95	3.75	2.2	3.7
7	Cycle	1.08	0.46	0.34	0.15	0.4	0.5

[a]DPCU indicates dynamic passenger car unit

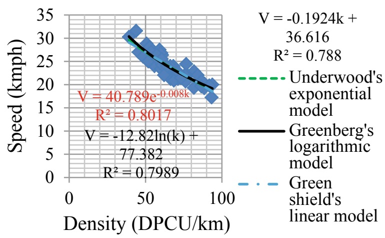


Fig. 9 Speed–density relationship of S. V. Road

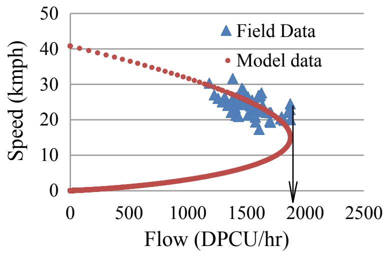


Fig. 10 Speed–flow graph of S. V. Road

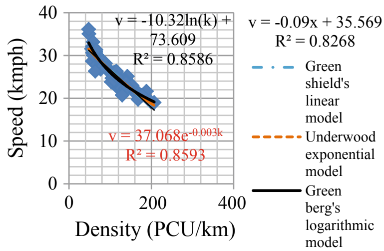


Fig. 11 Speed–density relationship of Raiya Road

is developed; in consequence, road capacity was obtained. In this study, capacity of two-lane, four-lane divided and undivided carriageway was estimated by different PCU and DPCU values in the same road sections, and the comparison result is given in Table 4 and Figs. 10 and 12.

7 Normalization of Capacity

The selected road width of four-lane and two-lane divided and undivided stretches is 20.40 and 10.40 m, respectively. There is a necessity to convert the road width into a standard width (7.5 m) and to calculate normal capacity. This normalization is done by following equation and then compared with the DSV recommended by IRC in Table 5.

Table 4 Capacity of roads with various PCU values

No.	Road name	Types of road	Direction of road	Carriageway width (m)	Capacity of the road (PCU/hr)		
					As per PCU value of IRC-106-1990	Based on Chandra's method	Based on homogenization coefficient method
1	Sadhu Vaswani Road	Two-lane undivided road	Both directions	10.4	3702	1876	2968
2	Raiya Road	Four-lane divided road	Toward 150-ft Ring Road	20.4	3514	1615	2850
			Toward Race Course Road		4892	2634	4388

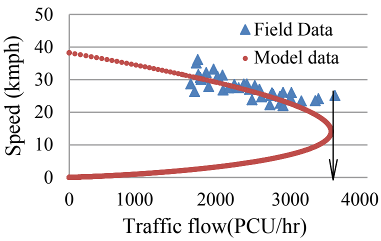


Fig. 12 Speed–flow graph of Raiya Road

$$NC = \frac{TC * SW}{CW} \tag{3}$$

where

NC normalized capacity.
TC total capacity.
CW roadway width of the road section (m).
SW standard width of the road (7.5 m).

8 Level of Service (LOS)

Level of service (LOS) is used to qualitatively define the traffic operational in a road and describe the quality of road facilities base for the road user. It described qualitatively rather than quantitatively in a common sense. The factors which have to be considered for LOS are travel time, speed, delay, convenience, maneuverability, and safety. Level of service is classified by a letter, A to F; A is representing free

Table 5 Comparison of capacity based on PCU value of IRC, homogenization coefficient, and Chandra's methods with DSV of IRC and Indo-HCM

Road name	Direction	Roadway width (m)		Capacity of the road (PCU/h)		DSV[a] (PCU/h) IRC	DSV (PCU/h) Indo-HCM
		Field (m)	Normal (m)	Observed capacity	Normalized capacity		
Based on PCU value of IRC							
Sadhu Vaswani Road	Both directions	10.4	7.5	3702	2670	1500	1705
Raiya Road	150-ft Ring Road	10.2	7.5	3514	2584	3600	2032
	Race Course Road	10.2	7.5	4892	3597	3600	2032
Based on homogeneous coefficient method							
Sadhu Vaswani Road	Both directions	10.4	7.5	2968	2140	1500	1705
Raiya Road	150-ft Ring Road	10.2	7.5	2850	2096	3600	2032
	Race Course Road	10.2	7.5	4388	3227	3600	2032
Based on Chandra's method							
Sadhu Vaswani Road	Both directions	10.4	7.5	1876	1353	1500	1705
Raiya Road	150-ft Ring Road	10.2	7.5	1615	1187	3600	2032
	Race Course Road	10.2	7.5	2634	1936	3600	2032

[a]DSV indicates design service volume

flow or the greatest operating condition, and F is the force flow or poorest operating condition. The carriageway width is designed and planned with consideration of a balance between safety and economy of design service volume; mostly for the urban road, levels of service, C and D, are considered. It is not advisable or allowable to plan or design the roadway width with the level of service E or F, which describes the equal or more than carriageway capacity with very low speed. In this research, attempt is made to calculate LOS based on volume to capacity (v/c) ratio and the obtained

Table 6 Comparison of LOS by different methods and with HCM and Indo-HCM

Peak hours period	LOS based on PCU value of IRC		LOS based on Chandra's DPCU value		LOS based on homogeneous coefficient DPCU value	
	HCM	Indo-HCM	HCM	Indo-HCM	HCM	Indo-HCM
Raiya Road toward 150-ft Ring Road						
9:00–10:00	C	C	D	D	C	C
10:00–11:00	D	D	D	D	C	C
11:00–12:00	D	E	D	E	C	C
17:00–18:00	D	E	D	D	C	C
18:00–19:00	E	E	E	E	D	E
19:00–20:00	E	E	D	E	D	D
Raiya Road toward Race Course Road						
9:00–10:00	D	D	C	C	C	C
10:00–11:00	D	D	C	C	C	C
11:00–12:00	C	C	C	C	C	C
17:00–18:00	C	C	C	C	C	C
18:00–19:00	C	C	C	C	C	C
19:00–20:00	C	C	C	C	C	B
Sadhu Vaswani Road						
9:00–10:00	D	E	D	E	D	D
10:00–11:00	D	E	D	E	D	D
11:00–12:00	D	D	D	D	C	C
17:00–18:00	D	E	D	E	D	D
18:00–19:00	E	E	D	E	D	E
19:00–20:00	D	E	D	D	D	D

ratio is compared with Indo-HCM-2017and highway capacity manual TRB-2000, which is suggested criteria for urban arterial road. Finally, the result is compared with various methods in the following tables and the level of service of morning and evening peak hour periods is defined (Table 6).

9 Roadside Friction

Roadside friction is demarcated as a combined variable that defines the degree of interaction between traffic flow and lateral activity, sometimes across or within the mode of travel. The activities disturb the traffic flow like pedestrian, bus stop, NMT, on-street parking, and roadside activities. The present study only considered these parameters as a roadside friction like bicycle, parking and stopping vehicle, pedestrian side walking, and pedestrian crossing. The friction data were collected by videography technique during the morning and evening peak hours at 5-min interval in selected road sections of 30-m interval. Based on the field data, multi-regression model has been developed between dependent and independent variables by SPSS software. The dependent variable was the speed, and independent variables were bicycles, parking and stopping vehicles, pedestrian side walking, and pedestrian crossing. The following equation indicates regression model.

$$\text{Speed }(V) = A + B_1(\text{BIC}) + B_2(V/C) + B_3(\text{PSV}) + B_4(\text{PEDSW}) + B_5(\text{PEDC}) \tag{4}$$

where

V	speed of the observed vehicle.
A	regression model constant which points out FFS.
B_1, B_2, B_3, and B_4	regression coefficients of the predictive variables.

This regression model is developed by SPSS-25 version software. In the result, R^2 value, regression coefficient, and β coefficient were attained. The β coefficient is a normalization or standardized coefficient, which is summed and weighted by using the following equation to determine the unit measurement of friction, and it is shown in Table 7 by these calculations.

$$\text{Friction model} = A(\text{PSV}) + B(\text{BIC}) + C(\text{PEDSW}) + D(\text{PEDC}) \tag{5}$$

where PSV, BIC, PEDSW, and PEDC are independent variables (No/30 m/h).

A, B, C, and D = weighted homogeneous or standardized coefficients.

9.1 Regression Models (With Variable and Without Variable)

In the present research, the relationship between speed and PSV, V/C ratio, BIC, PEDSW, PEDC with an explanatory power of the model R^2 value was first developed by the regression model. The regression calculation is then performed by excluding each of the variables, and the R^2 value is attained. The difference between the R^2 values with variable and without variable indicates the effect of factors that have an impact on the speed of the vehicle. These impacts are described in Table 8 (Figs. 13 and 14).

Table 7 Weighted coefficients and determination of friction

R^2 value	Independent variable	Regression coefficients	Beta coefficients	Weighted coefficients	Friction equation
Raiya Road toward 150-ft Ring Road					
0.425	V/C ratio	−8.8	−0.305	–	Friction = 1PSV + 0.043BIC + 0.186PEDSW + 0.06PEDC
	BIC	−0.019	−0.018	0.043	
	PSV	−0.299	−0.415	1	
	PEDSW	−0.088	−0.077	0.186	
	PEDC	−0.092	−0.025	0.060	
Raiya Road toward Race Course Road					
0.391	V/C ratio	−9.2	−0.35	–	Friction = + 1PSV + 0.03PEDC
	BIC	0.209	0.418	–	
	PSV	−0.235	−0.415	1	
	PEDSW	0.026	0.046	– –	
	PEDC	−0.024	−0.012	0.03	
Sadhu Vaswani Road					
0.464	V/C ratio	−7.704	−0.247	–	Friction = 0.21BIC + 1PSV
	BIC	−0.093	−0.11	0.210	
	PSV	-0.224	−0.517	1	
	PEDSW	0.065	0.073	–	
	PEDC	0.275	0.145	–	

9.2 Speed and Friction Relationship

In Table 7, the equations which were obtained by the weighted beta standardized coefficient were used for the field data for relationship between speed and friction; following figures show the speed–friction relationship (Figs. 15, 16 and 17).

Note: In the above graphs the (units/hr/30 m), it means the unit of friction in an hour of the trap length of 30 m of the pavement, and the decline line specifies that with increase in friction speed decrease.

10 Conclusion

This research is carried out on the two major stretches of the Rajkot city. These stretches are connected with major residential, industrial, and business centers, and the selected roads were two-lane, four-lane divided and undivided urban arterial roads. The required traffic data were collected by manual and videography survey in a 12-hour period and collected traffic flow converted by PCU and DPCU values of homogenization coefficient, Chandra’s method, and PCU value suggested by IRC.

Table 8 With and without variables and difference between R^2 values

Regression model	Friction model R^2 value	With and without variables			Difference of R^2	Difference by (%)
		Name of variable	Model	R^2		
Toward 150-ft Ring Road						
Speed = 36.6–8.77 V/C ratio − 0.019BIC − 0.299PSV − 0.088PEDSW − 0.92PEDC	0.425	BIC	S = 34.56 − 8.5 V/C ratio − 0.226PSV − 0.029PEDSW − 0.2PEDC	0.421	0.004	0.4
	0.425	PSV	S = 34 − 12.87 V/C ratio − 0.071BIC − 0.029PEDSW − 0.274PEDC	0.336	0.089	8.9
	0.425	PEDSW	S = 34.4 − 0.0631BIC − 8.17 V/C ratio − 0.223PSV − 0.179PEDC	0.425	0	0
	0.425	PEDC	S = 34.8 − 10.2 ratio − 0.021BIC − 0.285PSV − 0.039PEDSW	0.345	0.08	8
Toward Race Course Road						
Speed = 34.9 − 9.2 V/C ratio − 0.235PSV + 0.2BIC + 0.026PEDSW − 0.024PEDC	0.391	BIC	Speed = 35.6 − 0.287PSV − 6.4 V/C ratio + 0.015PEDSW − 0.08PEDC	0.241	0.15	15
	0.391	PSV	Speed = 31.432 − 0.25BIC − 7.2 V/C ratio − 0.075PEDSW + 0.024PEDC	0.274	0.117	11.7
	0.391	PEDSW	Speed = 34.9 − 9 V/C ratio + 0.2BIC − 0.024PEDC − 0.235PSV	0.389	0.002	0.2

(continued)

Table 8 (continued)

Regression model	Friction model R^2 value	With and without variables			Difference of R^2	Difference by (%)
		Name of variable	Model	R^2		
	0.391	PEDC	Speed = 35 − 9.5 V/C ratio + 0.2BIC − 0.231PSV + 0.026PEDSW	0.376	0.015	1.5
Sadhu Vaswani Road						
Speed = 34.4 − 0.093BIC − 7.7 V/C ratio − 0.224PSV + 0.065PEDSW + 0.275PEDC	0.464	BIC	S = 35.9 − 9.8 V/C ratio − 0.193PSV + 0.05PEDSW + 0.2PEDC	0.444	0.02	2
	0.464	PSV	S = 40.79 − 18 V/C ratio − 0.035BIC + 0.003PEDSW + 0.137PEDC	0.327	0.137	13.7
	0.464	PEDSW	S = 34.5 − 7.5 V/C ratio − 0.093BIC − 0.224PSV + 0.3PEDC	0.461	0.003	0.3
	0.464	PEDC	S = 34.3 − 7.2 V/C ratio − 0.023BIC − 0.214PSV + 0.06PEDSW	0.437	0.027	2.7

Dependent variable: speed Independent variables: BIC, PSV, V/C ratio, PEDSW, PEDC

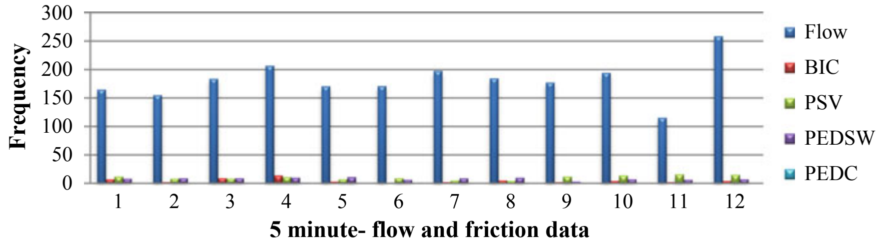


Fig. 13 5-min interval flow and friction data of the Raiya Road

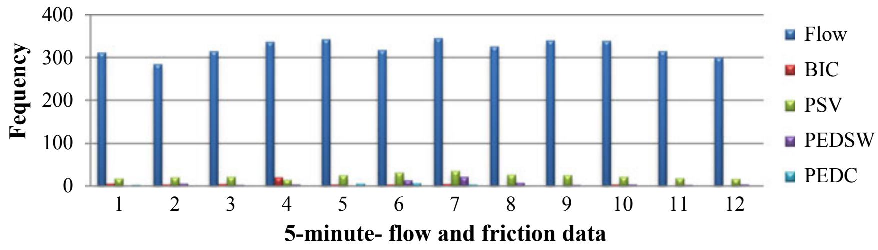


Fig. 14 5-min interval flow and friction data of the Sadhu Vaswani Road

Fig. 15 Speed–friction graphs in Raiya Road

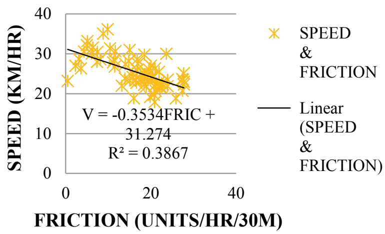
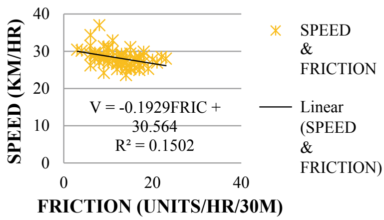


Fig. 16 Speed–friction graphs in Raiya Road



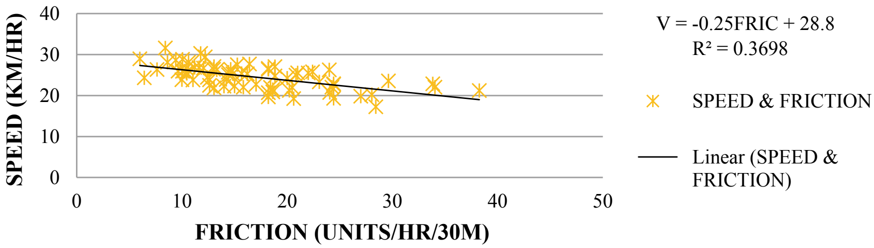


Fig. 17 Speed and friction graphs in Sadhu Vaswani Road

Greenshield, Greenberg, and Underwood models were used to develop relationship between speed and density, the Underwood model gives good results with an R^2 value more than 0.7, and the capacity of the road was estimated by the relationship of the speed–flow models. The width of the selected roads was more than a standard width (7.5) m; therefore, it needs to convert the observed capacity into normal capacity, which is carried out by Eq. 3. The level of service is determined by *v*/*c* ratio, and the ratio was compared with HCM and Indo-HCM recommendations.

- During the traffic flow survey, most of the traffic streams consisted of cars, 2-W and 3-W, and the LCV, bus and truck which were up to 5 percentage.
- The suggested PCU value as per IRC for 2-W and 3-W gives higher traffic flow, and the value is also more than the calculated DPCU value of homogeneous coefficient, and Chandra's methods; hence, the obtained capacity is also more.
- The average calculated DPCU value of Raiya and Sadhu Vaswani roads based on the homogenization coefficient method for 2-W and 3-W is (0.5, 0.94) and (0.48, 0.99), respectively, and the acquired capacity is less than IRC and more than Chandra's method. This method is mostly used in developed countries where they have good lane discipline.
- The average calculated DPCU value of Raiya and Sadhu Vaswani roads based on Chandra's method for 2-W and 3-W is (0.22, 0.91) and (0.21, 0.96), respectively. The average DPCU value of 2-W is very less than other methods, so the traffic flow and capacity are also lower than above two methods, but the obtained DPCU value of bus, LCV, and truck is more than IRC and homogeneous coefficient method, and the composition of these vehicles is up to 5%. Chandra's method due to consideration of static and dynamic characteristics gives good result of the Indian traffic situation.
- The level of service during the peak hour periods based on the HCM and PCU value of IRC method of Sadhu Vaswani and Raiya roads was observed (D, E) and (C, D and E). And also by Indo-HCM, it was observed E and D.
- The level of service during the peak hour periods based on the homogenization coefficient and Chandra's method of HCM was observed for Sadhu Vaswani roads C and D, and for Raiya Road, it was C, D and E, and by Indo-HCM, it was obtained B, C and D.

In the select road sections, a friction model was also developed by SPSS software by the 5-min interval collected friction data, the data which we considered in this study are PSV, BIC, PEDC, PEDSW, and *v*/*c* ratio, so the analyzed result is concluded by the following points.

- On the four-lane divided urban arterial road, the impact of PEDC, PEDSW, BIC, and PSV on the speed of the vehicle was observed (1.5, 0.2, 15, and 11.7)%. It specified that for pedestrians who are walking in the pedestrian walkway, there is less effect on the speed of the vehicle, but the PEDC, BIC, and PSV have an impact on the speed of vehicle; finally, the capacity of the roadway goes down due to the roadside friction.
- In the urban arterial two-lane undivided Sadhu Vaswani Road, the friction factors or variables impact was observed separately, in which the most influential factors were PSV and roadside parking, and these effects were (13.7, 2, 0.3, and 2.7)% of PSV, BIC, PEDSW, and PEDC, respectively.
- Multi-regression model is developed by SPSS software based on the roadside friction data; as a result, β coefficient, R^2, and regression coefficient are obtained. The

β coefficient is summed and weighted to find the unit measure of friction model. The collected data were practiced in the friction model and attained roadside friction, and the scatter graph between speed and friction is drawn. Eventually, it was observed that friction has a substantial effect on the speed of the vehicle, by increasing roadside friction, speed of the vehicle is reduced, and as a result, the capacity will be going down.

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Estimation of Equivalency Units of Vehicle Types for Road Geometry

N. Sai Kumar, V. M. Naidu, and C. S. R. K. Prasad

Abstract Capacity plays an important role while planning and designing of any roadway. The features of geometric of a roadway like the grade and curve radius will govern the capacity of a roadway. Passenger car unit (PCU) is used to estimate the capacity of the roadway. The passenger car unit values of a vehicle type alter concerning to speed. The speed of the vehicle is governed by geometric features of a roadway. This work objective is to learn the effect of geometric of the roadway such as grade, curve, and straight sections on PCU values of heterogeneous traffic conditions on a two-way four-lane national highway. Geometric and traffic data collected at 7 sections on NH-16. PCU value's estimation of the vehicle types of mixed traffic is difficult as compared with the homogeneous traffic conditions. PCU values are estimated by using speed–area ratio (dynamic PCU) method. Dynamic PCU (speed–area ratio) approach considers the vehicle average speed. The outputs had revealed that the capacity of the roadway declines as the percentage of downgrade increases. With an increase in the percentage of upgrade, the capacity of the road increases. The capacity of the roadway increased at the quick curve in contrast with mild curve and straight roads.

Keywords Passenger car unit · Geometry · Capacity · Grade · Curved · Dynamic PCU

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1 Introduction

Road network of India is around 5.60 million km. Indian road network is the next leading road system after the USA in the world. The USA having the street system length around 6.59 million km. India's total road network 0.15 million out of 5.60 million km is national highways, i.e., 2.7% of entire road system, and they carrying the 40% of whole road traffic, out of which only up to 26,000 km length of national highways are widened to 4 lane roads.

As developing countries like India, the traffic pattern is different, compared with developed countries. The traffic conditions are heterogeneous in India and vehicles do not move in lanes. In heterogeneous traffic conditions, various categories of vehicles travel on the identical road without any physical partition connecting them. But in the developed countries, the traffic pattern is homogeneous and lane-based, the same category of vehicles travels strictly in lanes at the similar speed.

Many researchers develop different techniques and models to analyze the heterogeneous traffic flow. Passenger car unit (PCU) is used to homogenize the heterogeneous traffic. The concept of PCU was first introduced in the highway capacity manual (HCM) in 1965 to convert a different class of vehicles into equivalent car units. Capacity has major significance while developing and new roadway design. The designer should have the profound facts about the planning of any new roadway, and that will influence the road design, preliminary investment and future repairing cost. The road geometric features like the upgrade, downgrade, and radius of the curve will influence the road capacity. On upgrades, heavy vehicles like buses, multi-axle vehicles and heavy commercial vehicles result in noteworthy and decrease in speed in contrast with the light vehicles. Reduction in the difference in the speed of the vehicle depends upon the dynamic features and vehicle weight. The discrepancy in the speed of the vehicle will result in the alteration of vehicle PCU values, at curved segment; the vehicle speed alters with respect to curve radius. At curves with smaller radius is experienced the deceleration as compared with mild curves, i.e., the vehicle speed alters considerably with respect to the change in radius of the curve.

The present study is undertaken to identify the influence of road geometry features like the grade of the road and the radius of the curve on PCU values. The traffic data were collected at 7 sections of four-lane national highway NH-16, and geometric details of the sections also collected. These data were analyzed, PCU values for each vehicle type at each section was estimated.

2 Literature Review

HCM provided PCU values are suitable for homogeneous traffic conditions. Researchers developed many methods for estimation of equivalency values in mixed traffic situation, such as modified density technique, the technique based on relative delay, speed–area ratio technique (Chandra and Sikdar 2000), and headway method.

PCU of vehicles is varied with respect to climate and environmental conditions of the area, roadway characteristics, etc.

Shalkamy et al. (2015) are calculated the curved and tangent area's effect on equivalency values, and they concluded that the PCU factors of vehicle types are increased linearly with the increase of radius of horizontal curve and road width. The PCU values are increased in the case of big vehicles as compared with small vehicles. Srikanth and Mehar (2017) develop speed models by using multiple nonlinear regression (MNLR) approach for estimation of PCU values in mixed traffic situations, and this work concluded that the calculated equivalency values of vehicle categories by using this process are slightly more, logical and realistic under mixed traffic situations compared with dynamic PCU.

Metkar et al. (2012) evaluated the existed techniques for assessment of equivalency factors and their aptness to mixed traffic situations and authors spotted some gaps in the current work for future study from the Indian perspective. Chandra et al. (2000) developed the dynamic PCU model for assessment of PCU factors in mixed traffic condition by considering the mean speed and rectangular projected area of each category of vehicles. Chandra and Kumar (2003) worked on the effect of width of the road on PCU factors on two-lane roads by using speed–volume relationship and dynamic PCU method, and it is reported that the equivalency values of a type of vehicle raises with respect to raise in width of pavement. The capacity of double-lane roadway rise with increase in total width of roadway and given an expression for adjustment factors for width of lane.

Chandra and Goyal (2001), revealed the road grade effect on equivalency values, and information were gathered from 8 different road segments and reported that every % of downward slope raises the capacity by 3.09%, and for every percent of upward slope, the capacity decreases by 2.61%. Chandra (2004) formulated the method to estimate the capacity of double-lane carriageway under mixed traffic condition. Information were gathered from around 40 selected sections all over India to examine the consequences of parameters that are influencing like traffic composition, lane width, gradient, slow-moving vehicles, shoulder width, directional split, and pavement surface conditions. Yagar and Aerde (1983) examined the effect of the slope of the road on functional speed of the vehicle and reported that approximately 1.8 km/h operating speed of the vehicle decrease for every 1% of the raise in grade of the road.

Arkatkar and Arasan (2010), a microscopic simulation technique was used for estimation of PCU values, for microscopic simulation authors developed HETEROSIM traffic simulation model to examine the effect of gradient of the road and its vehicle length performance in heterogeneous traffic and reported that equivalency values of the vehicle types are considerably varied with the traffic volume, size of the gradient, and its length. Arkatkar (2011) HETEROSIM microscopic simulation model was used to examine the effect of road geometry on equivalency values of vehicle types. PCU of vehicle types changes significantly with volume of traffic, upgrade and its length, roadway width.

3 Objective of Study

The objective of this paper is to assess the equivalency values of vehicle categories for road geometry like grade, curve, and straight sections by using dynamic PCU method on NH-16.

- To assess values of PCU of vehicle types at four graded sections as +4.2%, + 3.3%, −4.2%, and −3.3%.
- To assess values of PCU of vehicle for two curved sections with radius of 265 m and 85 m.
- To assess values of PCU of vehicle a straight segment.
- To compare PCU values of various geometries.

4 Collection of Data

Study includes data collection from 7 different sections on NH-16 near Visakhapatnam, two graded sections data collected near the zoo park: one curved section is near Venkojipalem bus stop, and another curved section is at Endada in Visakhapatnam and straight road section is at Ongole. All the segments are four-lane carriageway with two-way traffic with the carriageway width of 7.5 and 0.6 m of width of the shoulder. The grades of the sections were collected using the auto level and leveling staff. The radius of the curved sections is founded by using Google Earth Pro software and paint3D software. Video filming technique was used for traffic data collection.

4.1 Methodology for Traffic Data Collection

A stretch of 50 m was measured and that was marked the starting and ending by using plaster at each selected section. Camera was fixed at an appropriate place to include the section of 50 m. Traffic data for 2 h were gathered from at every stretch by video camera. The recorded videotape was projected on a large monitor, and travel time of every vehicle to cover the entire section was extracted with the help of video software, i.e., MPC-HC. MPC-HC is a video software that provides the time values in the order of 0.001 s. From travel time of vehicles, the vehicle's speeds were calculated.

4.2 Geometric Details of the Sections

See Tables 1, 2 and 3.

4.3 Volume of Traffic at Each Section

Total nine vehicle types are considered for this study. The type of vehicles is: A stands for Auto, CS stands for standard car, TW stands for two-wheeler, CB stands for big car, LCV stands for light commercial vehicle, HCV stands for heavy commercial vehicle, MAV stands for multi-axle vehicle, etc. (Table 4 and 5).

Table 1 Graded sections

Section no.	Carriageway width (m)	Grade of the section
Section 1	8.1	+4.2% upgrade
Section 2	8.1	-4.2% downgrade
Section 3	8.1	+3.3% upgrade
Section 4	8.1	-3.3% downgrade

Table 2 Curved sections

Section no.	Carriageway width (m)	Curve Radius (m)
Section 5	8.1	85
Section 6	8.1	265

Table 3 For straight section

Section no.	Width of Carriageway (m)
Section 7	8.1

Table 4 Volume of vehicle types at each section

Section no.	Duration (h)	TW	A	CS	CB	LCV	VAN	HCV	MAV	BUS
Section 1	2	2036	572	920	351	79	6	96	48	144
Section 2	2	726	359	700	284	49	‘20	78	20	116
Section 3	2	1791	500	611	638	124	22	29	114	158
Section 4	2	1018	320	432	487	98	20	19	133	147
Section 5	2	666	420	224	483	68	25	25	90	143
Section 6	2	1650	478	482	596	88	40	26	144	176
Section 7	2	407	109	104	110	34	14	49	190	62

Table 5 Dimensions and projected areas of vehicle types

Vehicle type	Length (m)	Width (m)	Area (m^2)
Two-wheeler (TW)	1.97	0.74	1.46
Auto (A)	3.20	1.30	4.16
Standard car (CS)	3.72	1.44	5.36
Big car (CB)	4.58	1.77	8.11
Light commercial vehicles (LCV)	4.30	1.56	6.71
Van (VAN)	5.24	1.95	10.21
High commercial vehicles (HCV)	6.70	2.30	15.41
Multi-axle vehicles (MAV)	11.50	2.42	27.83
Bus (BUS)	10.60	2.40	25.44

5 Assessment of PCU Values

The accuracy of the PCU value of types of vehicles is important in the estimation of any road capacity. The PCU values of vehicle types are used to alter a particular type of vehicle into PCUs. Number of scientists formulated various techniques to estimate PCU values in mixed traffic. In this work, speed–area (dynamic PCU) technique was used for calculation of the passenger car unit factors. Chandra and Sikdar (2000), developed dynamic PCU method for calculation of the PCU factors in mixed traffic situations.

Advantages of dynamic PCU method are given below:

1. It considers the average vehicle speed, because the factor of PCU of the vehicle types alters with respect to vehicle speed. This involves that the capacity of the roadway is fully the effect of change of speed (Arkatkar 2011)
2. It considers the projected rectangular area of the vehicle type to assess the equivalency values of the vehicle.

Limitation of the dynamic PCU method is given below:

1. This technique does not use the traffic composition in the estimation of the PCU values of vehicle types.

The equation for estimation of PCU values by using speed–area method is as follows:

$$\mathrm{PCU_i} = \frac{V_\mathrm{c}/V_\mathrm{i}}{A_\mathrm{c}/A_\mathrm{i}} \tag{1}$$

where

$\mathrm{PCU_i}$ Equivalent passenger car unit of the *i*th vehicle.
V_c The average speed of the passenger car (km/h).
V_i The average speed of type *i*th vehicle (km/h).

Table 6 Average speeds of each vehicle type at each section

Section no.	Average speed o vehicle in km/h (2 h)								
	TW	A	CS	CB	LCV	VAN	HCV	MAV	BUS
Section 1	56	38	50	44	37	50	31	28	30
Section 2	81	54	90	83	59	60	57	57	59
Section 3	60	50	65	63	58	57	56	54	58
Section 4	55	45	62	63	58	59	57	54	58
Section 5	38	32	34	33	32	35	30	30	36
Section 6	55	45	56	56	53	57	52	51	54
Section 7	55	47	94	95	63	64	53	51	61

A_c passenger car rectangular projected area (m^2).
A_i Type of *i*th vehicle rectangular projected area (m^2).

The mean speeds of each vehicle type at each section were calculated by using the travel time data; it is given in Table 6.

Substitute the mean speeds of the standard car (CS) and i_{th} vehicle, areas of the standard car (CS) and i_{th} vehicle in Eq. (1). The PCU factor of each vehicle type at each section was calculated by using dynamic PCU (speed–area) method. The PCU values of vehicle types at each section are given in Table 7.

The speeds of vehicle types were increased at downgrade sections as compared with upgrade sections, the physical separation between vehicles also increases at downgrade sections because of the speed of vehicle increases as compared with upgrade sections. The speeds of vehicle types were decreased at the sharper curve (85 m curve) as compared to the larger curve (265 m curve). Because vehicles were forced to decelerate speed due to lesser freedom to move, the PCU factor of each type of vehicle at each section was given in graphs. In the graphs, the x-axis represents the geometric feature and the y-axis represents the PCU values range (Figs. 1, 2, 3, 4, 5, 6, 7 and 8).

Table 7 Values of PCU of different types of vehicle in every section

Section no.	PCU values								
	TW	A	CS	CB	LCV	VAN	HCV HCV	MAV	BUS
Section 1	0.24	1	1	1.7	1.68	1.9	4.62	9.08	7.99
Section 2	0.3	1.29	1	1.64	1.94	2.87	4.56	8.7	7.28
Section 3	0.29	1.01	1	1.54	1.41	2.17	3.33	6.05	5.32
Section 4	0.3	1.05	1	1.48	1.35	2	3.14	5.99	5.07
Section 5	0.24	0.83	1	1.55	1.35	1.88	3.23	5.95	4.58
Section 6	0.27	0.96	1	1.51	1.34	1.89	3.13	5.73	5
Section 7	0.46	1.53	1	1.48	1.88	2.78	5.09	9.53	7.31

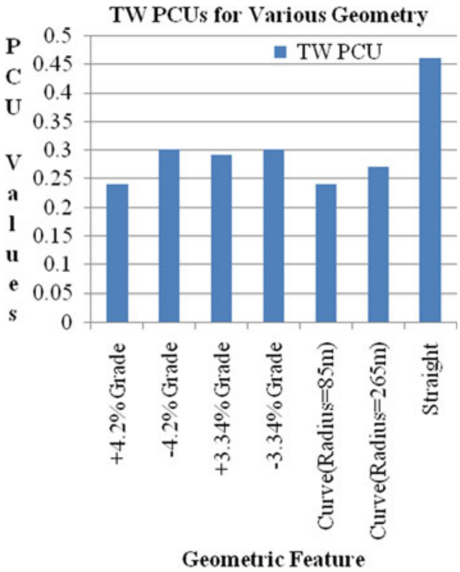

Fig. 1 PCU values of two-wheelers (TW)

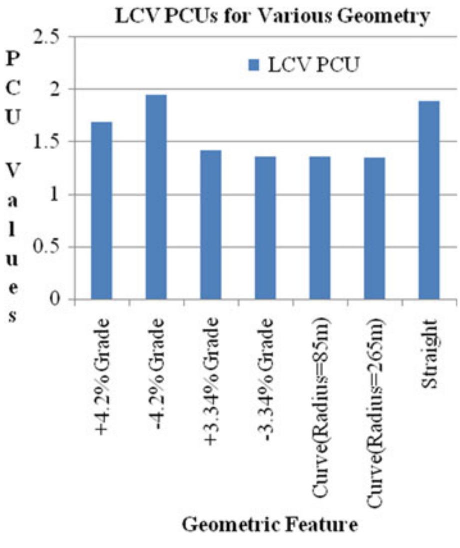

Fig. 2 PCU values of light commercial vehicles (LCV)

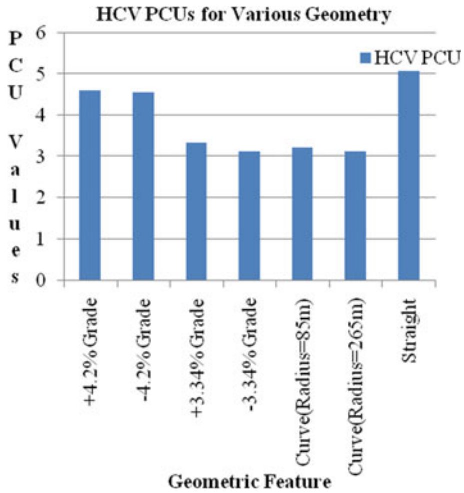

Fig. 3 PCU values of heavy commercial vehicles (HCV)

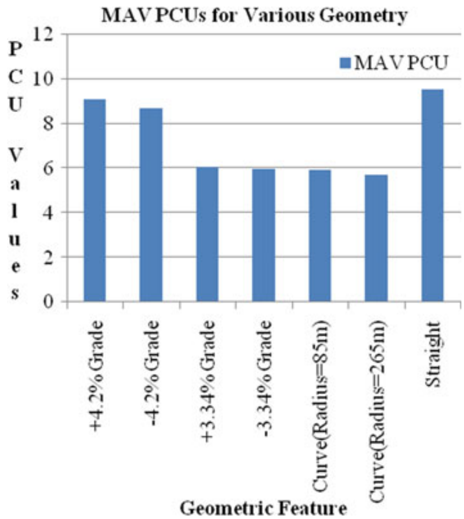

Fig. 4 PCU values of multi axle vehicles (MAV)

6 Summary and Conclusions

This paper presented the estimation of PCU values for road geometry. Dynamic PCU (speed–area) method was used to estimate PCU factors of 8 vehicle types by taking passenger car as a standard unit. Video filming technique was used for data collection at 7 sections and contains 4 graded sections, 2 curved sections, and one

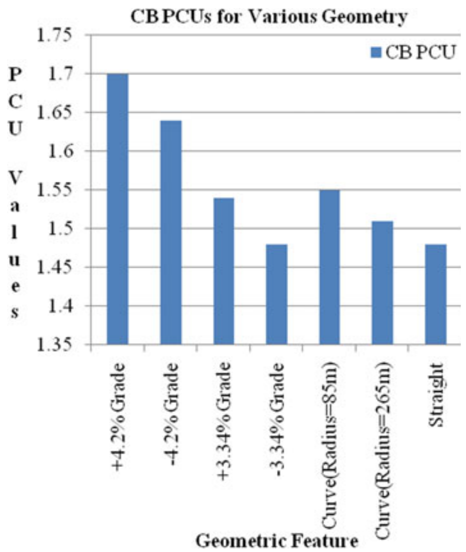

Fig. 5 PCU values of big car (CB)

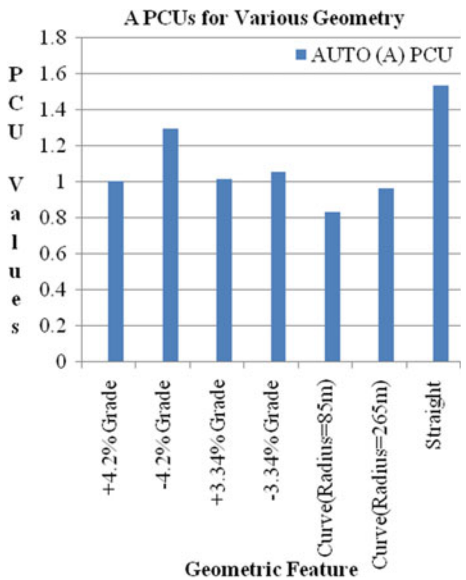

Fig. 6 PCU values of auto (A)

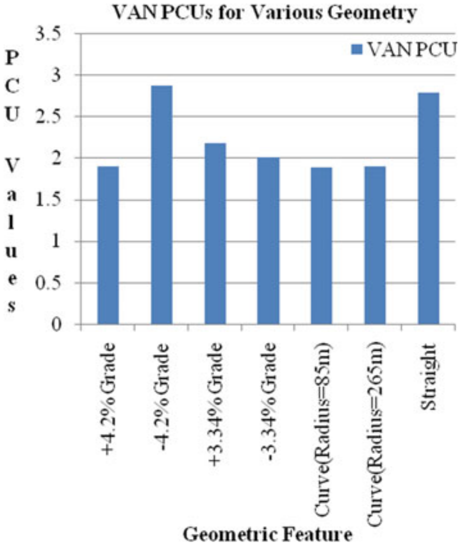

Fig. 7 PCU values of VAN

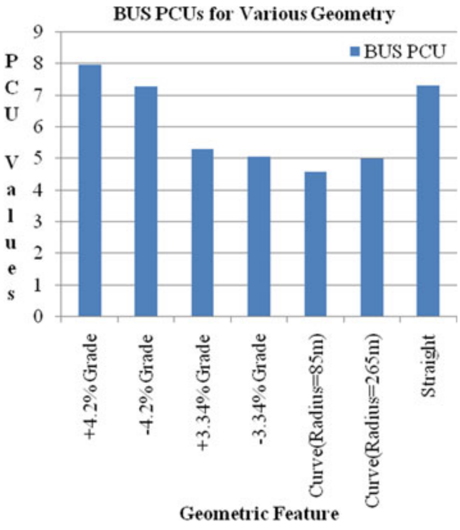

Fig. 8 PCU values of BUS

straight section. The speed of vehicles was calculated at each section by using the travel time extracted from video film. By using speed and projected rectangular areas of vehicles, the PCU factors of vehicle types were calculated by using dynamic PCU method. The conclusions arrived are:

1. The equivalency factors of various types of vehicles are higher at stretches + 4.2% grade, −4.2% grade, and straight stretches, because of the higher speed variation between the vehicles types and standard car.
2. The PCU factors of the smaller vehicles increased with respect to raise in curve radius, due to the higher freedom of the vehicle to move.
3. Heavy vehicles got little variation in their equivalency factors at two curved sections.
4. The equivalency factors of vehicles are increase with respect to rise in the upgrade, due to the drop in upgrade speed.
5. The equivalency factors of vehicles are decreased with the raise in the downgrade, due to the raise in downgrade speed.

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Capacity Estimation of Indian Multilane Highway by Calibrating Driver's Behavior Parameters

Tanumoy Ghosh, Sudip Kumar Roy, and Subhamay Gangopadhyay

Abstract The heterogeneous traffic conditions with non-lane-based traffic flow can be best evaluated by simulation of traffic. The present study uses VISSIM to simulate the mixed traffic of Indian multilane highways as it is a time increment-based multimodal simulation software. The psycho-physical behavior of drivers is considered in the present study because the driver performs an action when a threshold is reached to its boundary and is expressed as the function of speed differences and distances between the vehicles. The calibration by several dimensions is done on car-following model that has been modified by Wiedemann in VISSIM. The driver of a vehicle in VISSIM considers the leading vehicles and vehicles on adjoining lanes. Thus, hysteresis plots of relative speed against relative distance are made for aggregated leader and follower vehicles based on follower vehicular category to get the calibrated coefficient of correlation parameters (CC) used in VISSIM. A new simulation model with calibrated CC parameters is made to get a more realistic capacity estimate of multilane highway in Indian conditions.

Keywords Heterogeneous · Simulation · VISSIM · Hysteresis · Capacity

1 Introduction

The road network is considered to be very much crucial in the socio-economic expansion of any nation, particularly for developing nation like India. India along with many

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other developing countries is extensively involved in expanding their road network for building a better transportation and logistics system. India has one of the prevalent road setup in the world (National highway development …), spread across in diverse conditions of roadway, traffic, and weather. The rate of traffic flow growth is considered six to eight percent per annum and it is further expected to increase in near future, while the increase in road length is less than two percent resulting in rigorous obstruction taking place in every essential highway within the nation. The traffic flow demand is growing rapidly and soon it would be in the form of overcrowding of motorized and non-motorized vehicles on roadway networks. Significant increase in vehicular traffic volume on these roads is alarming to adopt special measures and to set priorities for improving current status of roadway system. India has been involved in the roadway capacity amplification through planning and implementing new multilane highway projects for providing better accessibility, safety, and service to the native people. The traffic flow in India is of mixed nature as it comprises of widespread vehicle variety that can take any lateral and longitudinal positions on a roadway without following lane discipline. It is complicated to evaluate the mixed traffic condition by the tactics developed for identical traffic since they are likely to illustrate incredibly minute consequences of the heterogeneity on highway capacity. The most critical issue in highway planning and management is to determine the roadway capacity of any inter-city highway or urban expressway. The formation of extra roads and highway is done to bring stability in increasing highway capacity and demand due to growing urban culture, superior expertise in transportation, and mounting financial system of the nation. The contemplation of prevailing road condition along with the traffic flow, including driver's vehicle following behavior is very much important for estimation of convenient capacity of highways. This situation urges the importance of modeling of traffic flow particularly by microsimulation system with consideration of every individual article in the structure.

2 Literature Background

The speed, flow, and density of a traffic stream comprising the fundamental relationships of traffic engineering assist in evaluating the highway capacity of a region carrying mixed traffic flow. The safety in vehicular movement and capacity of a roadway are affected by individual vehicle factors like individual speed, braking efficiency, and driver's behavior along with exterior influencing factors. In this circumstance, the following behavior of any vehicle (rather driver) to the vehicle in front of it becomes important as the heterogeneous traffic flow has no lane discipline to pursue.

In this regard, Gipps (1981) developed a behavioral-based car-following model to be used in computer simulation. The model generated the response of the following vehicle on the basis of limits on braking and acceleration rates set by a driver. The characteristic of real flow of traffic was produced by the model on assigning realistic values to various simulation parameters. Ramanayya (1988) tested a model based on simulation for mixed traffic movement under varied volume and percentage of vehicular mix. He then developed speed flow models and service volumes of three level of services based on the results of simulation run, that lead to the generation of passenger car unit relative to a design vehicle unit. Wiedemann and Reiter (1992) demarcated the dependence of two vehicles on the basis of relative speed between them by defining the response of an ensuing vehicle behind a vehicle moving in front. They developed different thresholds of perception and driving regimes that control the activities of the driver in a succeeding vehicle within a traffic stream. On the basis on this conception, Wiedemann 74 model for urban roads (Wiedemann 1974; Wiedemann 1974; Wiedemann 1991; Wiedemann 1991) and Wiedemann 99 model for freeways (Aghabayk et al. 2013; Aghabayk et al. 2013) were developed. Fellendorf and Vortisch (2001) worked on diverse ways of authentication of microscopic simulation of traffic model in VISSIM software under different situations of the real world. The desired speed distribution of vehicles on highways of Germany was used to calibrate the models used for simulation purpose. Arasan and Koshy (2005) evaluated heterogeneous traffic flow by simulation on the basis of pipes model of vehicle following characteristics. They validated their model on the basis of distribution of headway and speed of vehicles of different classes. Mallikarjuna and Rao (2006) estimated the passenger car equivalency standards by developing a model on simulation on the basis of cellular automata for heterogeneous traffic characteristics. They also examined the speed and area of diverse vehicles for analyzing the area occupancy concept of vehicles. Menneni (2008) has done microscopic illustration of traffic in his thesis work by using data of leading and following pair movement of vehicles. He calibrated coefficient of correlation parameters of VISSIM for identifying new patterns of vehicle movement to do precise measurements of diversity in complex traffic relationships. Mehar et al. (2012) worked on capacity estimation of the multilane highway having mixed traffic by using VISSIM. The speed–flow curve developed by both field and simulated data was compared to found that speed and capacity of the traffic were overvalued by VISSIM, and hence, it required calibration for heterogeneous traffic characteristics. Puvvala et al. (2013) made an attempt to estimate the capacity for urban expressway with no stringent traffic lane regulation and found the suitability of VISSIM for simulation of the diverse flow of traffic with statistical significance. Arkatkar et al. (2015) collected traffic data on expressway to build up simulating methodology for diverse traffic on Indian expressway. After calibration of the data, they validated the model on simulation to build the essential relations of traffic at different flow levels by using VISSIM software.

3 Methodology

After going through detailed literature reviews, it can be concluded that very few researchers have done simulation of heterogeneous traffic with appropriate methodology for calibration of vehicle following model. The present study has been carried out by calibration of psychological car-following model based on some individual hysteresis plots of different vehicle categories. The overall methodology of the present study initiates with the selection of the study section and data collection of free flow mixed traffic by videographic techniques. Then, the collected data has been analyzed extensively to get the traffic composition of the study section, along with volume, speed, flow, and density values by using some basic relations of traffic engineering. In the present study, a section each of four and six-lane separated highway has been selected for evaluation, of which the traffic data for one direction is collected. From the recorded data, the time headway and space headway have been calculated for individual vehicles to get the value of relative distance. The relative speed for individual vehicles has also been calculated for each of the lanes with one-way traffic separately. Hysteresis plot has been done with relative speed (X-axis) against relative distance (Y-axis) for each vehicle category moving in the selected sections considered for the current work. The values of relative velocity and relative distance are taken for hysteresis plots particularly for the vehicles maintaining time headway within 8 s. From the hysteresis plots, the coefficient of correlation (CC) parameters used in the microsimulation software VISSIM has been calculated (as discussed later). With the changed CC parameters than the default values, the simulation has been run and the result for both the calibrated parameter values and default values has been analyzed. The significant difference in the traffic parameters like speed, capacity, etc., of calibrated model values with the default parameter model justifies the importance and need of the use of calibrated simulation model for heterogeneous traffic conditions.

4 Study Section

The speed–volume data has been mainly collected for a trap length of 75 and 100 m for the National Highway number 45 and National Highway number 8, physically marked with the help of white paint and traffic cones. A video camera was then installed with the help of tripod stand at a height of about 6 m with 1.5 m cantilever. The camera was adjusted in such a way that the entire segment span was clearly visible to enable decoding of traffic volumes and space mean speed of different vehicle types. The section of National Highway 45 with chainage of 98 km 400 m near Acharapakkam (Chennai–Villupuram Highway, India), is a four-lane divided National Highway with each 3.5 m wide lane having straight and level geometry with 1.5 m paved shoulder along with 1 m earthen shoulder, while the section of National Highway 8 with chainage of 165 km toward Ahmadabad city in India is a six-lane divided National Highway with 12.35 m width of carriageway on either side having a straight and level geometry and 1 m paved shoulder along with 2 m morrum shoulder. The prevalent weather condition for data extraction was sunny without any rainfall.

5 Traffic Composition with Speed Characteristics

The traffic volume, composition, and the speed characteristics of vehicles in a traffic stream are very important components for estimation of vehicular capacity in a road network by direct empirical methods. The traffic volume data is used to calculate the traffic flow by using the passenger car unit values, which is divided by vehicle speed to get traffic density. Thus, the speed–flow relation is obtained from speed–density relation to get the capacity of the traffic stream. The speed of individual type of vehicles is measured by calculating the time of a vehicle to traverse a longitudinal length with accuracy of milliseconds. Table 1 represents the particulars of traffic volume, composition with space mean speed consideration on different highways.

6 Microscopic Simulation Model

In a microscopic simulation model, every unit of reality can be simulated individually. As a microsimulation model contains multiple parameters, it can simulate the individual interactions among vehicles along with complex driver's behavior in a mixed traffic flow particularly for Indian multilane highways. In the current work, simulation software VISSIM has been used to simulate heterogeneous traffic conditions in Indian multilane highways as it is a multimodal time increment, and behavioral activity-based tool for simulation to model diverse traffic (PTV 2013). VISSIM follows two modified models modified by Wiedemann on the basis of car-following theories and psycho-physical behavior of drivers as a driver can be in different manner of driving like diving without any hindrance, impending manner, following conditions, and braking behavior. The Wiedemann74 and Wiedemann99 car-following models are applicable in VISSIM, so the present study deals with calibrating the vehicle-following behavior of drivers by using these two models. The different threshold of the following behavior of a vehicle according to Wiedemann is represented in Fig. 1.

Some of the notations used in the figure according to Wiedemann (1974) are as follows: AX signifies the minimum distance headway in a standstill condition, that is the preferred space between the front portion of two consecutive vehicles waiting in a line of traffic, ABX corresponds to the minimum preferred vehicle following space, SDX represents the maximum desired following distance with the brink of perception to represent the maximum distance of vehicle following characteristics, SDV corresponds to the threshold at which a recognition by driver is made while approaching a slower vehicle, and OPDV and CLDV are the boundary conditions for variation in speed in opening and closing procedure, respectively, during a vehicle following situation.

The Wiedemann 74 model (Wiedemann 1974) in VISSIM is suggested for use in urban traffic. The driver behavior modeling in car following is based on perception thresholds. The formulation can be best explained using a relative velocity against

Table 1 Traffic volume, composition and space mean speed parameters on four- and six-lane divided highways

Highway type	Traffic volume (vehicle/h)	Vehicle type	Traffic composition (%)	Speed parameters (km/h)				
				Maximum	Minimum	Average	15th percentile	85th percentile
NH-45 Four lane	824 (9–10 a.m.), 1081 (10–11 a.m.), 1369 (1–2 p.m.), 2258 (4–5 p.m.), 2457 (5–6 p.m.)	CS	25	134.89	21.69	70.99	49.8	86.6
		BC	26.5	149.88	21.39	74.15	53	91
		TW	29.24	117.39	18.71	49.76	32	64.2
		3 W	1.3	55.28	23.48	37	22.94	46.4
		LCV	4.84	94.99	26.2	47.65	26	59.6
		HCV	3.3	87.1	25.14	39.25	27.88	50.2
		MAV	1.42	83.26	28.07	35.02	26.1	47
		BUS	8.4	112.97	30.82	53.57	22	64.2
NH-8 Six lane	1019 (9−10 a.m.), 1087 (10–11 a.m.), 793 (1–2 p.m.), 874 (4–5 p.m.), 1128 (5–6 p.m.)	CS	23.28	138.73	24.21	75.17	55.23	86.74
		BC	23.34	143.85	28.98	80.24	60.78	91.23
		TW	12.18	109.91	20.62	52.85	37.82	59.67
		3 W	3.86	69.95	21.98	40.96	26.4	48.86
		LCV	13.4	91.56	22.58	57.21	38.21	66.93
		HCV	12.76	96.33	21.62	47.13	30.1	51.37
		MAV	8.28	71.94	19.60	41.36	29.3	48.69
		BUS	2.9	101.91	26.67	62.51	39.76	69.92

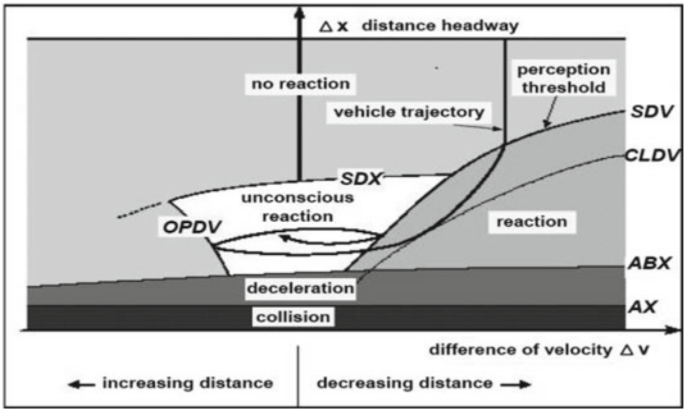


Fig. 1 Threshold of Wiedemann car-following model. *Source* Menneni (2008)

relative distance graphs. The following formula of the model is implemented in the present study for determining the coefficient of correlation parameters from the hysteresis plots as explained later.

$$\mathrm{ABX} = \mathrm{AX} + (\mathrm{bx\_add} + \mathrm{bx\_\ mult} * N[0.15, 0.5]) * \sqrt{\mathrm{Vslow}} \quad (1)$$

The Wiedemann 99 model is also very much effective for following behavior of vehicles and is efficiently applied during the present study to find out different parameters related to drivers' car-following behavior. The coefficient of correlation (CC) parameters as per Wiedemann 99 model are denoted from CC0 to CC9. The parameter CC0 having no deviation characterizes the preferred bumper-to-bumper (front) distance amid congested vehicles.

$$\mathrm{AX} = \mathrm{CC0} \quad (2)$$

CC1 characterizes the time that a driver in a following condition desires to maintain.

$$\mathrm{ABX} = \text{Lead Vehicle Length} + \mathrm{CC0} + \mathrm{CC1} * \text{V slower} \quad (3)$$

CC2 controls the longitudinal alternation throughout the vehicle following state. So, it identifies the additional distance more than the preferred safety distance prior to the driver purposely moves closer to another vehicle.

$$\mathrm{SDX} = \mathrm{ABX} + \mathrm{CC2} \quad (4)$$

CC3 characterizes the commencement of deceleration procedure as soon as a driver begins to slow down by identifying a slower vehicle in front.

$$\text{SDV} = \text{CC3} \quad (5)$$

CC4 and CC5 characterize the difference in speed throughout the course of vehicle following situation. CC4 and CC5 control variation in speed in closing and opening procedure, respectively, during a vehicle following situation.

CC6 delineates the effect of distance on speed alternation throughout the following state.

CC7 delineates real acceleration during alternation in a following situation.

CC8 delineates preferred acceleration as a vehicle starts from a motionless state.

CC9 delineates the preferred acceleration of a vehicle at 80 km/h speed. Though, it is restricted by the utmost acceleration in favor of the type of vehicle.

7 Calibration of Model Parameters

The sensitivity of the driver's following behavior parameters is investigated by taking simulated capacity as a measure of effectiveness. In the present study, the parameter calibration of Wiedemann 99 model is as follows: CC0 is considered as the standstill distance between the vehicles, which is taken from calibrated Wiedemann 74 model. After that, with the help of Solver Tool in Microsoft Excel, the optimized value of CC1 is calculated. Then, from the relative distances as obtained from the field data, cumulative percentage plots are developed and the 25th percentile value is taken as the value of CC2. The parameter CC3 is defined as the perception threshold when leader starts decelerating, but as it is very difficult to calculate this limit, it is assumed as a straight line and the slope of the line is taken as CC3. The parameters CC4 and CC5 are negative plus positive following threshold, respectively, of speeds. These parameters control the speed differences between the vehicles during the car-following process. So, the parameters CC4 and CC5 are calculated from the hysteresis plots as 25th percentile value of cumulative percentage plots of negative and positive values, respectively, of relative velocity. As there is no proper verification of explanation for CC6, the default value is taken in the study. Due to the absence of data for acceleration at oscillation, standstill, and at 80 kmph speed, defaulting standards of CC7, CC8, and CC9 are taken for the current work. The changed values of the CC parameters depending on the field data are shown Table 2 for different vehicle categories moving in two different road sections and hysteresis plots of relative velocity against relative distance for standard cars of both the road sections of NH-45 and NH-8 are shown in Figs. 2 and 3.

Table 2 Wiedemann 99 model parameters listed with changed values in VISSIM

Highway type	Parameter	Default value	BC	SC	LCV	HCV	MAV	BUS	TW	3 W
NH-45 four lane	CC0	1.5	1.24	1.18	1.32	1.64	1.69	1.41	0.78	0.65
	CC1	0.9	0.33	0.38	0.43	0.64	0.57	0.49	0.41	0.71
	CC2	4	7.85	7.74	7.26	7.83	8.26	9.78	7.88	8.14
	CC3	−8	−11.6	−9.91	−8.99	−8.73	−10.8	−7.74	−6.59	−9.16
	CC4	−0.35	−1.52	−1.64	−1.2	−1.24	1.14	−1.17	−1.34	−1.06
	CC5	0.35	2.04	1.14	1.28	1.22	1.27	1.58	2.03	1.19
NH-8 six lane	CC0	1.5	1.39	1.36	1.25	1.74	1.79	1.73	0.86	0.91
	CC1	0.9	0.57	0.6	0.71	0.8	1.21	0.79	0.75	0.81
	CC2	4	6.96	7.41	7.92	8.03	7.57	7.64	7.16	9.39
	CC3	−8	−7.32	−4.57	−7.27	−6.66	−6.19	−7.11	−7.09	−5.87
	CC4	−0.35	−1.34	−1.17	−1.18	−0.95	−1.09	−1.02	−1.14	−1.1
	CC5	0.35	1.42	1.36	1.26	1.72	1.01	1.19	1.04	1.22

Fig. 2 Hysteresis plot of standard cars for a section of NH-45

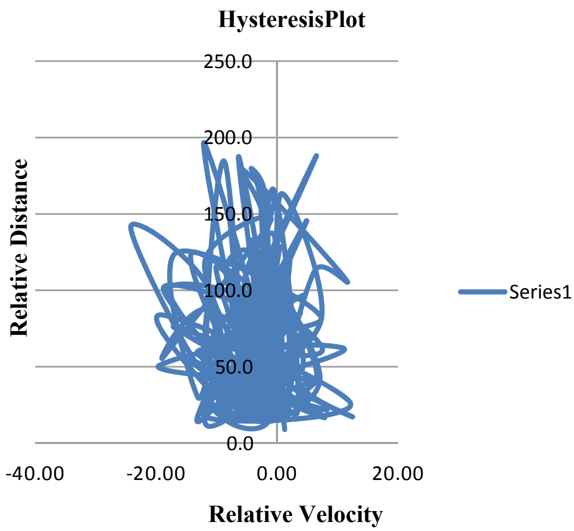
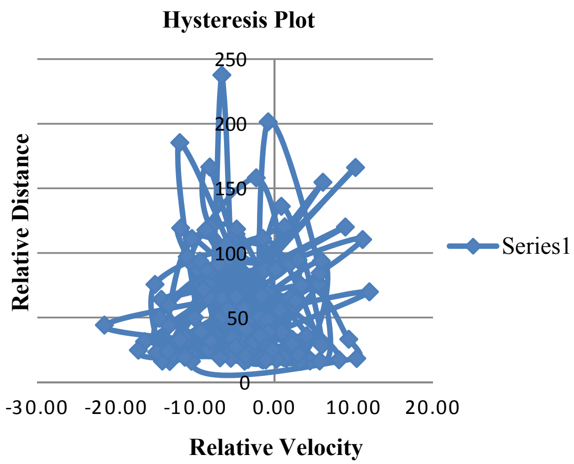


Fig. 3 Hysteresis plot of standard cars for a section of NH-8



8 Capacity Estimation Method

In this study, the capacity of both four- and six-lane separated highway has been calculated by using traditional (mainly Greenshield's model) and simulation techniques. From the traffic data observed, two-hour peak traffic both in morning and evening and one-hour data in the afternoon has been considered. Passenger car units (PCU) of diverse vehicle variety is utilized to get the flow value of traffic in PCU/h by converting the heterogeneous traffic volume measured (at every five minutes interval) to homogeneous volume (Chandra et al. 1995). Average value of stream speed is calculated by determining speed of individual vehicles in five-minute volume count.

The speed–volume data is converted to speed–density data by making use of the fundamental relationship between speeds, flow, and density. Various forms of equations including Greenshield's model (which became predominant with greater R^2 value) are tried to fit speed–density data. The best fit speed–density curve has been used to develop the complete shape of speed–flow curve for estimating the capacity of roadway. Capacity is estimated by simulation method in the same process except that the simulation is run with the changed coefficient of correlation parameters as obtained from the field data. The results of simulation give changed volume and speed values of the same traffic stream. Thus, with the changed values, the capacity estimated gives a realistic value than the value estimated with default coefficient of correlation parameters.

9 Validation of Model Parameters

The model calibration and validation are necessary to make a realistic simulation model. The sensitivity of the coefficient of correlation (CC) parameters in VISSIM has been done by two-way analysis of variance that is by ANOVA: Two factors without replication, to see whether any change in parameter value from its default value has effect on simulated capacity at 5% level of significance. It is seen from the test on default values and estimated values of CC parameters that the *p*-value for every vehicle category is less than the critical *p*-value of 0.05 as shown in Table 3. Thus, it can be inferred that the estimated CC parameters (CC0, CC1, CC2, CC3, CC4, CC5) are significantly different from the default CC parameters, that can ultimately have effect on the simulated capacity of the highways.

For verifying the sequential justification of simulation model, a comparison of observed and simulation flow value has been done at 5 min interval of traffic flow. The bar charts showing sequential justification of standard traffic flow at each 5 min intermission for two road stretch has been represented in Figs. 4 and 5.

A statistical validation has been done through the paired t-test, where the anticipated t-statistic, *p*-value, and t-critical values acquired for 5% level of significance, alongside relevant degrees of freedom. The results inferred the anticipated t-statistic value (−1.617) for NH-45 and t-statistic (−1.906) for NH-8 is lesser than the t-critical (2.0009 for NH-45 and 2 for NH-8) value for t-test and the p-value (0.111 for NH-45 and 0.061 for NH-8) is more than the 5% significance level. Thus, it is accomplished that there is no statistically noteworthy disparity between the observed and simulated flow values of mixed traffic in favor of a significance level of 5%.

Table 3 Results of the two-way ANNOVA test

Road Name	Criteria	BC	SC	LCV	HCV	MAV	BUS	TW	3 W
NH-45	*p*-value	0.003875	0.001781	0.001078	0.001115	0.002842	0.004861	0.00235	0.001848
NH-8	*p*-value	0.000543	0.003814	0.001259	0.001215	0.000905	0.000756	0.000716	0.005728

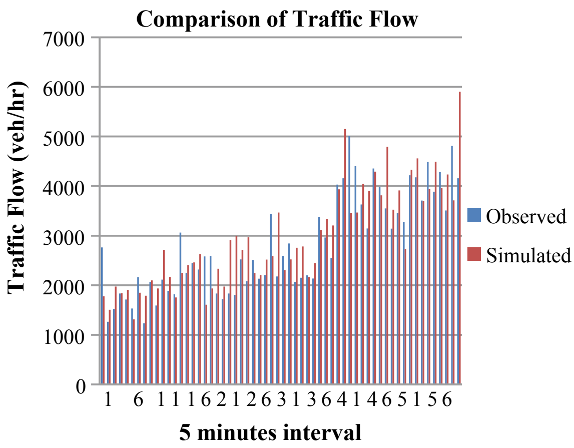


Fig. 4 Comparison of observed and simulation flow of traffic at a section on NH-45

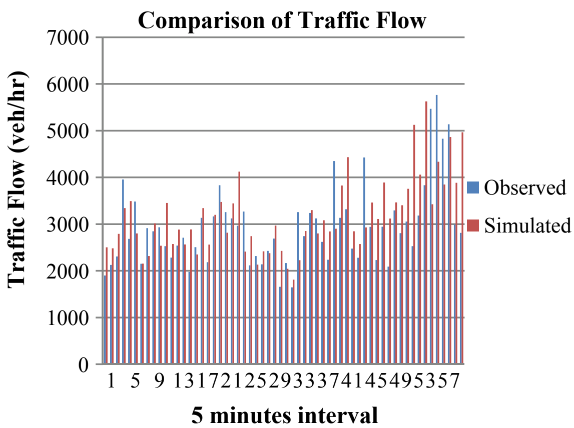


Fig. 5 Comparison of observed and simulation flow of traffic at a section on NH-8

10 Conclusion

The simulated capacity estimated for the four- and six-lane separated National Highways has been found to be 5296 PCU/h/direction and 6593 PCU/h/direction, respectively, with average lane capacity of 2648 PCU/h/lane and 2198 PCU/h/lane, respectively, while the capacity estimated by traditional method for the same has been found to be 5445 PCU/h/direction and 6469 PCU/h/direction, respectively, with average lane capacity of 2723 PCU/h/lane and 2156 PCU/h/lane, respectively. Thus, the simulated capacity varies with the traditional capacity values for four- and six-lane separated highway sections by 2.73% and 1.91%, respectively, that lies well within a variation of 5% value as the level of significance considered for data validation is also 5%. It can be inferred from the result that the simulation model produces realistic capacity estimate with changed coefficient of correlation parameters.

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Performance Enhancement of an Un-signalized Intersection Under Heterogeneous Traffic Conditions Using Microscopic Simulation: A Case Study of Bhumkar Chowk Intersection

Pankaj More, Ashish Methekar, Kasturi Kesarkar, Monali Nerkar, Shantanu Karhadkar, and Siddharth Gaikwad

Abstract The emergence of IT parks in Pune city has led to an increase in the flow of traffic during peak hours at Bhumkar Chowk, which is a four-legged un-signalized intersection with a vehicular underpass. This paper attempts to enhance the performance of Bhumkar Chowk intersection which is a typical example of a heterogeneous traffic scenario seen in India. The work is also carried out to resolve the traffic congestion by reducing the travel time, queue length, delay and traffic volume. Signals are designed for the intersection as well as new vehicular underpass adjacent to the current underpass is proposed.

Keywords Heterogeneous traffic · Microscopic simulation · Travel time · Delay · Queue length

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1 Introduction

Traffic congestion is a major problem in all metropolitan cities. Pune being a major developing city is also facing such a crisis. Bhumkar Chowk is a four-legged intersection with a vehicular underpass situated below NH 48, connecting Dange Chowk to Hinjewadi IT Park, PCMC. It is located on the western side of Pimpri-Chinchwad. The Rajiv Gandhi Infotech Park of Hinjewadi houses hundreds of companies of different sizes in a 2800 acre area. The vehicle categories flowing towards the IT Park through the intersection areas follows Cars, Two-wheelers, Three-wheelers, Buses, Heavy Good Vehicles (HGVs) and Light Commercial Vehicles (LCVs). Most of the commuters use their privately owned vehicles daily. Also, there is a lack of public transit system. This leads to an increase in traffic congestion at Bhumkar Chowk intersection and creates undesirable scenarios for commuters in terms of delay, long queue length, lesser stream speed, etc. The signals are manually operated by the traffic police which leads to chaos and irregularity in traffic flow.

Due to heavy traffic volume at the intersection, videographic survey was adopted to identify the cause of traffic congestion. Manual survey was carried out at turning movements. Further, classified traffic volume count was obtained by manually extracting the data from the recorded videos for peak and non-peak hours. The variation in traffic volume for different movements about the intersection was known.

Microscopic simulation model is developed using VISSIM 10.05 for the entire area of extent. The area of extent of the study is shown in Fig. 1. Validation is an important step in simulation and necessary to be done so that few important parameters should be in agreement with the actual field observations. The parameters considered for validation of simulation model are Travel time, Delay and Queue length. After the validation of the base model is complete, it is compared with the updated models comprising the proposed solutions. A Before–After analysis is performed to compare the simulation models and give a spotlight on the changes in the traffic flow after the proposed solutions are implemented.

2 Literature Review

This study aims to reduce traffic congestion at Bhumkar Chowk, Wakad. Thus in order to examine this topic, a review of literature was carried out. Saha et al. (2013) analyzed traffic congestion and suggested remedial measures at the traffic intersection in Pabna City, Bangladesh. They concluded that the existing city road network was inadequate for increasing traffic volume and managing the traffic flow was the only way to reduce the traffic volume.

Patel (2014) studied the traffic conditions at Thaltej intersection, Gujarat wherein accidents were a major problem. Various alternatives like redesigned round-about, traffic signal, flyover, underpass and uplifted round-about were proposed and checked for their sustainability. Thakur et al. (2017) analyzed the present condition at Gunjan

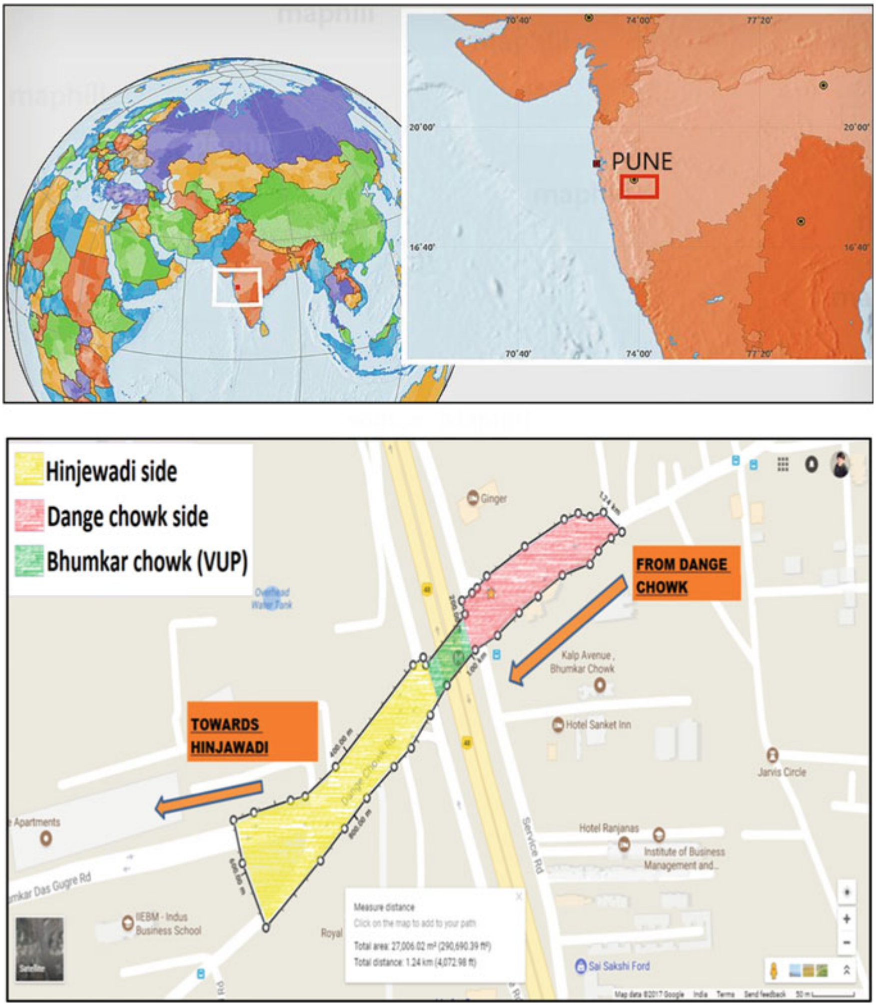

Fig. 1 Location map of study area.*Source* Maphill. *Source* Google Earth

Chowk, Pune by carrying out a manual count survey and vehicle moving survey on an hourly and daily basis. The present intersection was insufficient to carry the traffic towards Pune station as well as Nagar road and hence a suitable solution was suggested to handle the current heavy traffic flow. Jabeena et al. (2014) studied the traffic flow characteristics and traffic stream modelling on Mahatma Gandhi Expressway. They developed a model in VISSIM to study the flow characteristics in depth. The results obtained from this study are useful for estimating the stream characteristics of the expressway and provide a basis for planning, designing the expressway in the future. Chepuri et al. (2015) suggested feasible traffic management measures, which may result in the reduction of delay and travel time to both BRTS buses and private traffic. Hendricks (2015) analyzed the process of validation on PTV VISSIM 7. He analyzed the simulation effects on average standstill distance between two vehicles. Charypar et al. (2015) studied the possibilities of reducing the cost by using a queue-based model. Siddharth and Ramadurai (2013) represented the sensitivity analysis and automatic calibration for Indian heterogeneous condition. Maji et al. (2015) studied the traffic operations under a flyover and a modified traffic island layout was proposed for the effective channelization of traffic.

3 Study Area

Pimpri-Chinchwad is located in Pune District of Maharashtra state. It has a rapidly developing industrial area. Pimpri-Chinchwad is at latitude 18.6298° N & longitude 73.7997° E and covers an approximate area of 171.5 km^2. PCMC has experienced rapid growth in the IT sector located at Hinjewadi in recent years. This has led to an increase in the vehicular volume through Bhumkar Chowk corridor. Maximum commuters using this intersection are the employees of the IT sector. The traffic composition in the area of extent shows a predominance of two-wheelers which are 60% of total vehicles while cars comprise 33% and the remaining composition constitutes of trucks and buses.

The total length of the study area is 959.79 m. The study area consists of four carriageways out of which two main carriageways are centrally located and two service roads on either side of the main carriageway. A reconnaissance survey was carried out in which area of extent, conflicts and types of surveys to be carried out has been decided. The study area was subdivided into three parts viz, Dange Chowk side, Bhumkar Chowk (VUP), Hinjewadi side for ease in surveying work and is indicated in Fig. 1.

4 Data Collection

Various traffic surveys were carried out for the collection of data on-road characteristics, traffic volume count, accident count and signal timings. The road dimensions are necessary for setting up road networks in VISSIM for simulation of models. Traffic volume count is required for knowing the volume at different legs of intersection and thereby prioritizing the lanes in the software. Accident data helps in identifying whether the intersection is accident-prone and signal timings are required to know the maximum and minimum cycle time at the intersection.

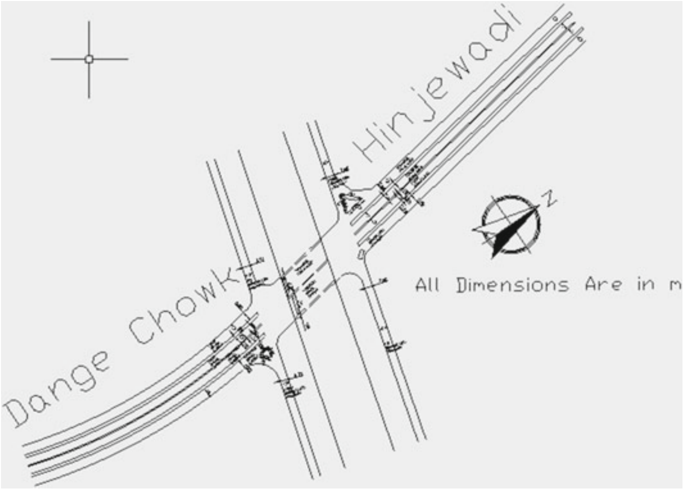

Fig. 2 AutoCAD drawing of Bhumkar Chowk

4.1 Carriageway Width Survey

Carriageway width survey of the study area is carried out to obtain characteristics and various geometric features of the road. The main road approaching the vehicular underpass is having 16.12 m wide two-lane dual carriageway, 8.06 m on each side. The median dividing the carriageway is 0.8 m wide. The width of the service road is 7.04 m on each side. The width of the vehicular underpass is 7.68 on each side. The width of service roads advancing towards Bengaluru and Mumbai are 6.71 and 7.62 m, respectively. The following figure shows the widths of carriageways in the study area drawn in AutoCAD (Fig. 2).

4.2 Accident Data Collection

Accident data gives information about all types of accidents occurring at a particular intersection or road stretch. Such data serves to identify the basic causes of accidents and suggest means for overcoming the deficiencies that lead to such accidents. Accident data was collected from the authorized police station. These data states the number of accidents, mishaps occurred at Bhumkar Chowk for the time span of 3 years (January 2014–August 2017) (Table 1).

This data was necessary to understand whether the said intersection was accident-prone area or not. From the given results, it is found that Bhumkar Chowk is not accident-prone area; also the accidents do not contribute to the traffic congestion.

Table 1 Number of accidents per year

Year	Accidents	Minor accident	Fatal accident
2014	5	4	1
2015	2	–	2
2016	5	1	4
2017	2	2	–

Table 2 Survey Details

S. No.	Traffic survey direction	Type of survey	Date and day of survey
1	Dange Chowk–Hinjewadi	Videographic Survey	14.09.2017 Thursday
2	Hinjewadi–Dange Chowk	Videographic Survey	14.09.2017
3	Hinjewadi–Mumbai	Manual survey	02.02.2018 Friday
4	Hinjewadi–Pune	Videographic survey	14.09.2017
5	Mumbai–Hinjewadi	Videographic survey	14.09.2017
6	Mumbai–Dange Chowk	Manual survey	19.01.2018 Friday
7	Dange Chowk–Mumbai	Manual survey	09.02.2018 Friday
8	Dange Chowk–Pune	Videographic survey	14.09.2017
9	Pune–Dange Chowk	Manual survey	16.02.2018 Friday
10	Pune–Hinjewadi	Manual survey	16.02.2018

4.3 Traffic Volume Count Survey

The traffic volume survey was carried out for peak hours (8:00 a.m. to 11:00 a.m.) and for non-peak hours (2:00 p.m. to 5:00 p.m.) to verify variations in traffic volume count, which was done using videographic and manual survey. The vehicular count has been taken for all the legs of the intersection. The camera was setup at Dange Chowk side since the vehicular volume entering and leaving the intersection is prohibitive. The location of the camera was chosen such that the traffic from both the directions in the main carriageway was visible. In order to save time, manual survey was carried out for the legs having low vehicular volume (Table 2).

4.4 Signal Timing Survey

Existing signal designed for the intersection has failed to regulate the current traffic flow. Therefore, the traffic is managed manually by the traffic police. The traffic on main roads (Dange Chowk and Hinjewadi) is given priority and the traffic on service roads is merged along with it. Average signal cycle time was obtained by observing the manually operated signals for one hour (Table 3).

Table 3 Detailed Virtual Signal Timings

S. No.	Direction	Signal timings (minutes: seconds)		
		Maximum	Minimum	Average
1	Dange Chowk–Hinjewadi (Green Time)	4:45.82	1:16.66	2:41.41
2	Dange Chowk–Hinjewadi (Red Time)	3:38.63	1:25.79	2:9.52
3	Hinjewadi–Dange Chowk (Green Time)	3:38.63	1:25.79	2:9.52
4	Hinjewadi–Dange Chowk (Red Time)	4:45.82	1:16.66	2:41.41

5 Data Extraction

5.1 *Traffic Volume Count Survey*

Videos collected from videographic and manual survey were extracted manually and categorized into twelve different vehicular classes such as Two-Wheeler, Three-Wheeler, Car, Utility vehicle (Jeep, Van), Mini Bus, Standard Bus, LCV-Passenger, LCV-Goods, 2-Axle Trucks, 3-Axle Trucks, Multi Axle Trucks (Semi Articulated), Multi Axle (Articulated), Tractors, Tractors with Trailers, Cycle and Cycle Rickshaws. This data was entered direction wise in MS Excel Sheet. The data used as input in the simulation model consists of five vehicular categorizes clubbing similar types of vehicle classes as stated below (Table 4; Fig. 3).

Table 4 Vehicle types and classes

S. no	Vehicle types	Vehicle classes
1	Two-Wheeler	Two-Wheeler
2	Three-Wheeler, LCV-Passenger, LCV-Goods	Three-Wheeler
3	Car, Utility Vehicle,	Car
4	Mini Bus, Standard Bus	Bus
5	2-Axle, 3-Axle, Multi Axle Semi Articulated, Multi Axle Articulated	HGV

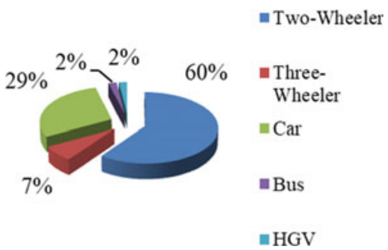

Fig. 3 Vehicle composition at Bhumkar Chowk

Table 5 Results obtained from speed and delay survey

S. No.	Field delay (s)	Travel time (s)
1	95.545	145.3182

5.2 *Speed and Delay Survey*

Speed and delay survey was carried out to find out the delay and travel time required in different legs of the intersection. The data was collected manually and is represented in tabular form below. This data shows the total field delay and total travel time required in all the legs of the intersection (Table 5).

6 Microscopic Simulation of Bhumkar Chowk

6.1 *Introduction*

VISSIM is a microscopic simulation model that is used to analyze and optimize traffic flow (Fig. 4). This software provides a variety of applications for urban roads and expressways for complex heterogeneous traffic flow. The simulation model is worked out in VISSIM in the form of the following flowchart as shown in Fig. 5. The model is developed for the existing field conditions which is termed as base model. Its development can be summed up as developing a base network, defining model parameters, model calibration and model validation (Fig. 5).

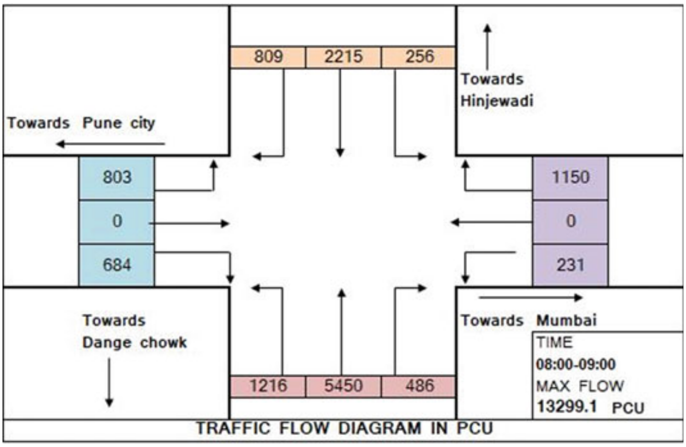

Fig. 4 Direction wise volume of traffic

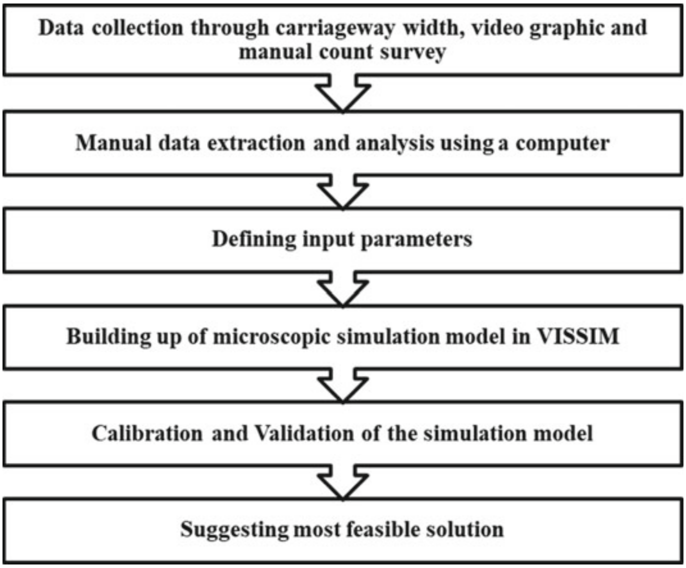

Fig. 5 Procedure followed for development of base model

6.2 Preparation of Base Model

The base model was created using VISSIM 10.05 for a stretch of 1.2 km including the Bhumkar Chowk intersection.

The links are created and the directions are specified as per the field conditions using the data extracted from the carriageway width survey for various road widths. The connectors are used to link the crossroads with the main roads. The behaviour of the links is selected as urban motorized as the survey area falls in urban zone. Shifting lanes is adopted at the site for reducing and controlling the queue length. Such conditions cannot be created in the software due to its limitations. Thus, to match the actual field conditions the width of the intersection is doubled to get a similar flow at the site. Similarly, a model for signalized intersection having new signal design and an updated signalized having adjacent vehicular with newly designed signals are developed in VISSIM 10.05 (Fig. 6).

6.3 Input Parameters

As VISSIM 10.05 (student version) is used for simulation so, the simulation is run for a comparatively lesser duration of 600 s. The buffer time is taken as the first 120 s out of the total run time. Total volume of different categories of vehicle is taken for one hour survey period from 9:00 a.m. to 10:00 a.m. as relative flow in six directions

Fig. 6 Overview of base model in VISSIM 10.05

in VISSIM model by extracting data from the vehicular volume count (Table 6). The connectors are designed according to observations taken on the field. Various vehicle classes considered are Two-wheelers, Three-wheelers, Cars, Buses and HGV, and volume of each class is defined for different legs. The desired speed of each vehicular category is decided by specifying minimum and maximum speed in heterogeneous traffic scenarios.

Priority rules are used for modelling of un-signalized intersection at Bhumkar Chowk. Firstly, major roads are given high priority which leads to long queue formation on minor roads. An attempt is made to reduce the queue length by setting undetermined priorities. Various conflict points are observed at the site therefore, priorities for conflicts are set as passive to reduce the queue length in the links.

The intersection is an un-signalized intersection, which is operated manually. Thus, the manually operated signal timings are recorded. A ninety-fifth percentile value of the observed readings is taken and signal cycle timings are designed for a two-phase signal by Webster method matching present condition on the field. Evaluation parameters like provision of vehicle travel time counters, queue counters and delay time counters are added to carry out the evaluation of the base model. This is an important step for validating the model.

Table 6 Traffic flow (in terms of number of vehicles per hour)

S. No.	Direction		Vehicle classes				
	From	To	Two wheeler	Three wheeler	Car	Bus	HGV
1	Hinjewadi	Dange Chowk	1392	153	374	96	76
2	Hinjewadi	Mumbai	185	47	59	11	15
3	Dange Chowk	Hinjewadi	4124	175	1439	90	47
4	Dange Chowk	Bengaluru	812	87	385	27	37
5	Pune	Dange Chowk	520	44	331	16	60
6	Mumbai	Hinjewadi	743	61	299	10	50

6.4 Driving Behaviour

VISSIM is developed in Germany and is calibrated according to the European driving conditions. To match the Indian driving conditions for heterogeneous traffic, it is necessary to add the Indian driving behaviour in VISSIM. This study uses Weidmann 74 model which is one of the two cars following models used for designing base networks in VISSIM. Driving behaviour is adopted to suit the Indian traffic conditions. Some of the parameters which differentiate the Indian traffic conditions from European countries are average standstill distance, additive part of safety distance and multiplicative part of safety distance.

6.5 Model Calibration

Calibration adjusts the various parameters of the simulation model till the model precisely depicts field conditions. The data extracted from different surveys is used as an input for base model. Simulation runs are performed to carry out the estimation of output. Validation checks the behaviour of the model with respect to traffic performance, matching the actual field conditions at microscopic levels. Calibration process includes default vehicular geometrical parameters such as length, width, weight, axle spacing and wheel base of the different vehicular categories considered (Arkatkar et al. 2011). Calibration of the model is based on trial and error method.

The calibration parameters considered are as follows: Simulation resolution (10 Time steps/sim. s), simulation speed (10 sim. s/s), vehicle composition (2 W-60%, 3 W-7%, Car-29%, Bus/HGV-4%), look ahead distance minimum and maximum (40 and 250), look back distance minimum and maximum (30 and 150), average standstill distance (0.30–0.50), free lane selection, overtake on the same lane from left and right, minimum lateral distance (Distance standing 0.20 at 2 km/h and Distance driving 0.50 at 50 km/h), minimum longitudinal speed (1 km/h), maximum speed difference of cooperative lane change (3 km/h), maximum collision time of cooperative lane change (10 s), desired speed of vehicles (25–100), total simulation time. The total simulation time is 600 s out of which the initial 120 s is the buffer time.

6.6 Model Validation

Base model is validated for four parameters which include delay time, travel time, queue length and volume. Speed and delay survey was carried out for peak hour (9:00 a.m. to 10:00 a.m.) using car as well as two-wheeler in all six directions of the intersection. It helped in obtaining the delay and travel time required by the vehicle. Another survey was carried out to find the queue length, where the queue length was calculated manually using measurement tape with the help of markings on four major

Table 7 Average travel time (s) for validated model

S. No.	Direction	Distance (m)	Average travel time (s)		Error (MAPE, %)
			Field values	Simulated values	
1	Hinjewadi to Dange Chowk	394.95	166	151.965	8.454
2	Hinjewadi to Mumbai	350.15	105	115.043	8.729
3	Dange Chowk to Hinjewadi	392.11	184.5	171.282	7.164
4	Dange Chowk to Bengaluru	379.19	109	123.095	12.931
5	Pune to Dange Chowk	450.66	185	162.012	12.425
6	Mumbai to Hinjewadi	391.42	217	192.822	11.141

roads identified at the intersection. The trial and error method is continued until the MAPE (Mean Absolute Percentage Error) value falls below 25%. It is observed as 10.14% in our case on the random seed no. 12, 16 and 24 (Table 7).

7 Data Analysis

The analysis aims at channelizing the traffic flow to reduce the congestion at the intersection. The width of the vehicular underpass is inadequate to handle the present traffic volume whereas the roads approaching the intersection are comparatively wider. This creates a bottleneck formation at the intersection which causes delay and reduces the stream speed. Providing the un-signalized intersection with signal-based system is also an effective way to manage the traffic. A comparative analysis for travel time and delay for base model as well as the two modified models helps in finding the technical feasibility of the best solution to be implemented. The travel time and delay in all six directions are significantly reduced after the application of improved signal system but, a provision of new adjacent underpass along with updated signals helps in optimizing them to a greater extent.

The queue length at the four directions as mentioned in Table 8 is comparatively higher than other directions and therefore, these four directions are considered for validation of queue length. A survey was carried out for one hour from 9:00 a.m. to 10:00 a.m. to obtain queue lengths after each signal cycle and average value of these values is used for comparing the queue length of base model as well as the modified models. The queue length at service roads on both sides of the intersection is decreased considerably but it is increased on the main roads of the intersection to some extent (Tables 9 and 10).

Table 8 Queue length (m) obtained for different cases

S. No.	Link name	Field data	Base model	Signalized intersection	Updated signalized intersection
1	Hinjewadi Service Road	20	27.674	4.482	2.009
2	Hinjewadi Main Road	79	68.508	95.986	84.268
3	Dange Chowk Service Road	108	76.36	4.288	3.095
4	Dange Chowk Main Road	131	92.527	76.577	88.542

Table 9 Total delay (s) for different cases

S. No.	Field delay	Base model	Signalized intersection	Updated signalized intersection
1	95.545	76.484	31.819	45.878

Table 10 Travel time (s) for different cases

S. no.	Travel time counters	Travel time (s) derived in VISSIM		
		Base model	Signalized intersection	Updated signalized intersection
1	Hinjewadi to Dange	151.965	57.753	42.126
2	Hinjewadi to Mumbai	115.043	108.689	98.49
3	Dange to Hinjewadi	101.282	55.066	56.835
4	Dange to Bengaluru	123.095	44.496	37.529
5	Pune to Dange	122.012	80.798	33.68
6	Mumbai to Hinjewadi	92.822	41.013	41.498

From the comparative analysis, it can be seen that the updated signalized intersection is technically more feasible than the signalized intersection.

8 Sustainability of the Solution

The sustainability of the recommended updated signalized intersection is to be checked, as it would help in finding its suitability in the long run. The growth in traffic is forecasted for the next decade, the forecasted values are derived from the past traffic volume data provided by the Regional Transport Office (RTO), Pimpri-Chinchwad. The average annual growth rate of traffic is calculated as 11.1% for the decade. Simulation runs are carried out for finding the travel time, delay and queue length. These results are compared with the values of current field condition and updated signalized intersection given in Tables 11, 12 and 13.

From the comparative study of the above three tables, it is seen that despite the increase in queue length, the travel time and total delay are still less than the current field conditions. The increase in traffic volume over the years justifies the increased length of the queue but the overall delay and travel time still remains comparatively lesser than the present condition. Thus the proposed model can effectively work for a period of ten years (Tables 12 and 13).

Table 11 Travel time (s) for period of ten years

S. no.	Link name	Travel time (s) derived in VISSIM			
		Field value	Updated signalized intersection	After 5 years	After 10 years
1	Hinjewadi to Dange	166	42.126	87.671	100.364
2	Hinjewadi to Mumbai	105	78.49	95.679	131.458
3	Dange to Hinjewadi	184.5	56.835	69.975	75.692
4	Dange to Bengaluru	109	37.529	47.021	54.317
5	Pune to Dange	185	33.68	72.227	91.899
6	Mumbai to Hinjewadi	217	41.498	59.735	71.399

Table 12 Total delay (s) for period of ten years

S. No.	Field delay	Updated signalized intersection	After 5 years	After 10 years
1	95.545	45.878	49.122	54.748

Table 13 Queue length (m) obtained period of ten years

S. No.	Link name	Field data	Updated signalized intersection	After 5 years	After 10 years
1	Hinjewadi Service Road	20	2.009	6.916	12.400
2	Hinjewadi Main Road	79	84.268	127.324	133.922
3	Dange Chowk Service Road	108	3.095	6.917	47.180
4	Dange Chowk Main Road	131	88.542	110.035	118.800

9 Conclusion and Recommendations

The traffic volume at Bhumkar Chowk intersection is increasing day by day due to an increase in the number of employees in the IT sector. Also, there is an increase in the traffic density at Bhumkar Chowk intersection due to bottleneck formation at vehicular underpass. From the results of traffic volume study, it is seen that the density of traffic increases considerably during peak hours to 5020 PCU which exceeds the urban road permissible PCU value of 2500.

The carriageway width is sufficient to handle such heavy traffic but, the width of underpass is inadequate. The following recommendations are given to improve the existing field conditions.

1. Improved signal timings:

The existing signals are operated manually; the cycle time of existing signal is 443 s whereas the cycle time of the improved signal is 86 s. The signal is designed as a two-phase signal with the Hinjewadi side as the first phase and Dange Chowk side as the second phase of the signal. The red, amber and green time for phase 1 are 59.4 and 23 s, respectively, and that for phase 2 are 30.4 and 52, respectively (Fig. 7).

2. Provision of adjacent vehicular underpass with updated signals:

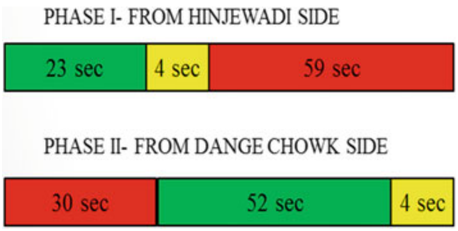

Fig. 7 Newly designed signal timings

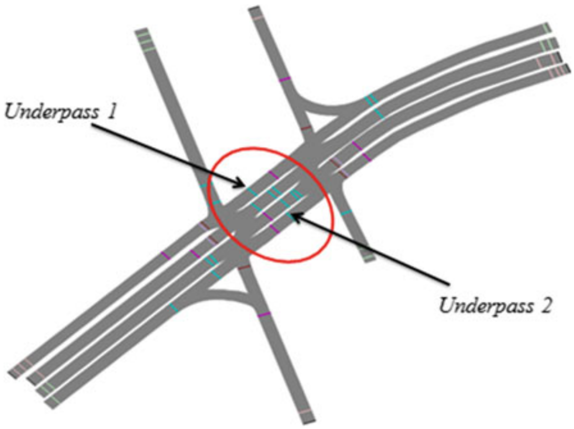

Fig. 8 Provision of adjacent vehicular underpass

The provision of the new vehicular underpass along with the updated signals is recommended as it is technically more feasible. The width of box culvert is 8.1 m on each side as it is provided on either side of the existing vehicular underpass.

The base model developed for the project can be adopted for further research on Bhumkar Chowk. This study has been carried out to check the technical feasibility of solutions, a check on the economic feasibility of the proposed solution can be carried out further before the execution of the solution (Fig. 8).

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The Evaluation of Traffic Congestion Analysis for the Srinagar City Under Mixed Traffic Conditions

Adinarayana Badveeti, Mohammad Shafi Mir, and Kasinayana Badweeti

Abstract On urban roads in India, there has been a rapid increase in Motor vehicles (MV) and also an increase in a large number of non-motorized road users since last few decades on urban roads in Srinagar metropolitan area (SMA) under heterogeneous traffic conditions. Traffic congestion in the city will cause a large number of problems. The current situation of traffic jam condition on urban-transport networks are very high and that occurs as use rapidly growing and is distinguished by longer trip times, smaller speeds, and fast-growing vehicular queuing. The present methodology aims to study traffic congestion indicators such as level of service (LOS) of roadway, Travel Time Index (TTI), and their variants. The data were collected at different locations in the CBD area of the city where high traffic congestion flows were observed. At these selected locations, vehicular volume count, spot speeds (m/s) were observed during peak and non-peak hours. Motor vehicles were categorized based on different vehicle groups. The final results obtained from the calibration and validation of models were discussed and the obtained level of service comes under F. It was concluded that Rainawari was found to have the major traffic congestion different in peak hours and the concerned authorities like Srinagar Development Authority (SDA) should take proper remedial measures to control traffic congestion issues on the road link between NIT Srinagar and Dal gate especially Rainawari area.

Keywords Traffic congestion · Congestion modeling · Vehicular volume · Travel time index

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1 Introduction

In developing countries like India, in Srinagar, the metropolitan area is rapidly increasing the population and vehicular traffic volume since the last few decades. So there is a big congestion problem in the city due to the lack of improper widening road networks on urban roads under mixed traffic conditions. Particular preventive measures have been taken in order to check like car-pooling, road pricing policy, and enhanced traffic management and the span of new road construction can for short-term relieve congestion; in the prolonged-term, it directly encourages additionally a growth in car traffic through travel time increased and a move away from the public transport (PT). The comprehensive paper report on which this concept is based aims to issue congestion policy maker and technical staff with real-time data from the field measurement.

The present situation of Srinagar city is unplanned road networks available on urban roads under mixed traffic conditions. In the future, there is scope of widening the roads, implementation of proper congestion preventive measures, and traffic congestion management policies which can be implemented on heterogeneous traffic environment on urban arterial roads.

2 Research Objectives

The proposed project work aims at identifying the major congestion zones on the various link roads connecting NIT Srinagar Campus to Dal gate area and study of the flow characteristics like volume and spot speed. The specific objectives of the research work are as follows: (i) To conduct spot speed studies and volume count related studies in various identified segments joining the NIT Srinagar Campus and Dal gate area. (ii) To analyze the data obtained from the segments for statistical and traffic model related studies. (iii) Determine the level of service of the various link roads joining the two locations. (iv) Identify the major congestion zones between NIT Srinagar Campus and the Dal gate area.

3 Literature Review

To examine the current review of literature based on the Travel Time Index (TTI), level of service (LOS), preventive congestion measures, etc. and we studied about different road sections like homogeneous and heterogeneous traffic on urban roads of various countries like India, China, and the U.S.A.

Year	Author name and Country	Subject, method, and study parameters	Remarks
2004	Downs (2004)	They were proposed to describe traffic congestion on roadways in urban areas	Preventive measures of traffic congestion not taken his research work
2001	Varaiya (2001) (U.S.A) California (Department of Transportation)	They have identified the congestion problems and to handle the congestion problems on urban roads (He considered only weekdays.)	He has defined congestion as occurring on a freeway when the average speed drops below 35 mph for 15 min or more on a typical weekday

Year	Author name	Subject or parameter	Remarks
2006	Bertini (2006) (Minnesota)	They have defined freeway congestion traffic flowing below 45 km/h for a different length of the direction They have defined freeway congestion in terms of LOS F, when the volume/capacity ratio is greater than or equal to one	He considered only weekdays (b/w 6:00 a.m. and 9:00 a.m. or 2:00 p.m. and 7:00 p.m. on a weekday) Level of service (LOS) is F (very poor facilities on urban roads.)
2007	Choi et al. (2007)	1. They have conducted a study by applying the Travel Time Index (TTI) to show the level of traffic congestion 2. They have observed that Travel Time Index reports that the traffic congestion in both space and time with a minimum of data collection efforts	They determined only Travel Time Index and they won't determine other parameters related to the traffic congestion
1996	Parbat and Tare (1996) (Indore city, India Mixed traffic conditions)	They determined the Travel Time Index by using test car technique and studied the level of congestions through traffic indices related to travel time	Only Travel Time Index was considered and using text car method only
2002	Stathopoulos and Karlaftis (2002)	They have studied the estimation of the time duration of congestion on the given road section with respect to the following time periods	They estimated travel time along the road section with respect to the different time periods

(continued)

(continued)

Year	Author name	Subject or parameter	Remarks
2011	Denver Regional Council of Governments (2011) (DRCOG) in U.S.A	It examines the traffic congestion parameter like freeways, vehicle speed (mph) Travel Time Index (TTI), etc.	Keys: the organization has taken congestion preventive measurements and mitigation strategies
2012	Mohan Rao and Ramachandra Rao (2012) (CSIR New Delhi IIT Delhi, India.)	They studied on measuring urban traffic congestion—a review	They considered only two factors (1.) microlevel factors and (2.) macrolevel factors related to the traffic congestions on urban roads mixed traffic conditions
2015	Kukadapwar and Parbat (2015)	Evaluation of Traffic Congestion on Links of Major Road Network: A Case Study for Nagpur City 1. They have identified TTI and LOS for urban roads	They suggested suitable congestion measures need for Indian urban roads under heterogeneous environmental conditions
2014	Wen et al. (2014)	Study on Traffic Congestion Patterns of Large City in China Taking Beijing as an Example 1. They have studied TPI, TTI, and macroscopic analysis	1. They were studied on weekdays, holiday, and weather conditions 2. Congestion demand management policies were studied

4 Study Methodology

4.1 Study Area

The potential congestion zones between NIT Srinagar Campus and Dal gate were chosen as study sites. Study sites were selected in such a way that major changes in traffic can be effectively tracked. The selected sites are in the form of study stretch lengths (Fig. 1).

4.2 NIT–Dal Gate Road

After a reconnaissance survey of the selected study area, sites were chosen at which traffic conditions were supposed to change. Mid-block sections were chosen in such a way that they represented the traffic condition for the entire road segment. Five sites

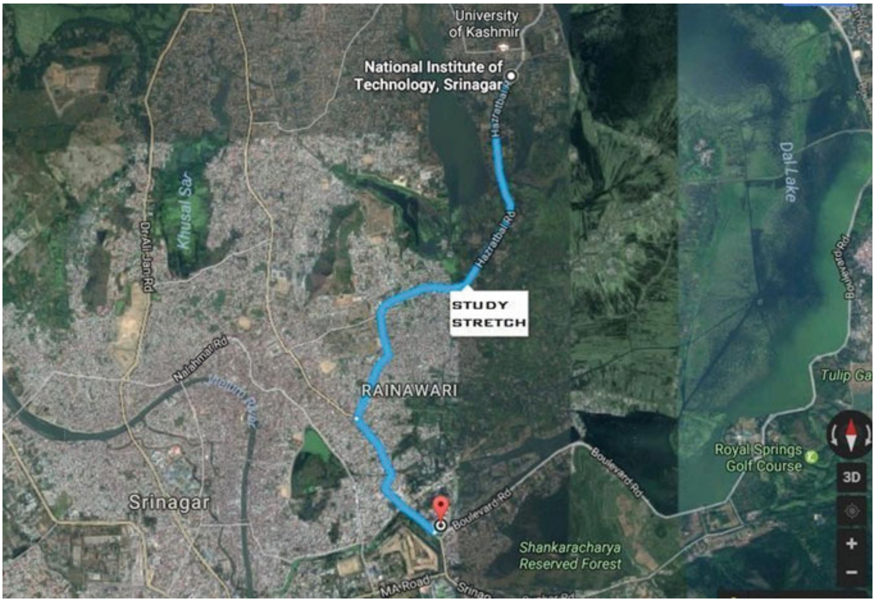


Fig. 1 Study stretch (*Source* Google earth)

were chosen between NIT campus (Hazratbal) and Dal gate. The latitude, longitude, and the distance of each site from NIT are shown in the below (Fig. 2)

4.3 Site Selection

The study area consists of five major link roads connecting NIT Srinagar campus to Dal gate, namely Nigeen, SaidaKadal, Rainawari, Khanyar, and Dal gate. These roads are located within Srinagar city of Jammu and Kashmir, India. Numbers of famous tourist places, heritage sites are located on these roads so a good amount of traffic is playing on these roads. The selected sites are chosen in such a way so that there is considerable variation in traffic data. The selected sites are zones of heavy congestion.

5 Data Collection

The data collected includes volume counts of 17 h, the rest 7 h are extrapolated as per instructions. Spot speed studies are performed for congestion hours as congestion usually occurs in these hours.

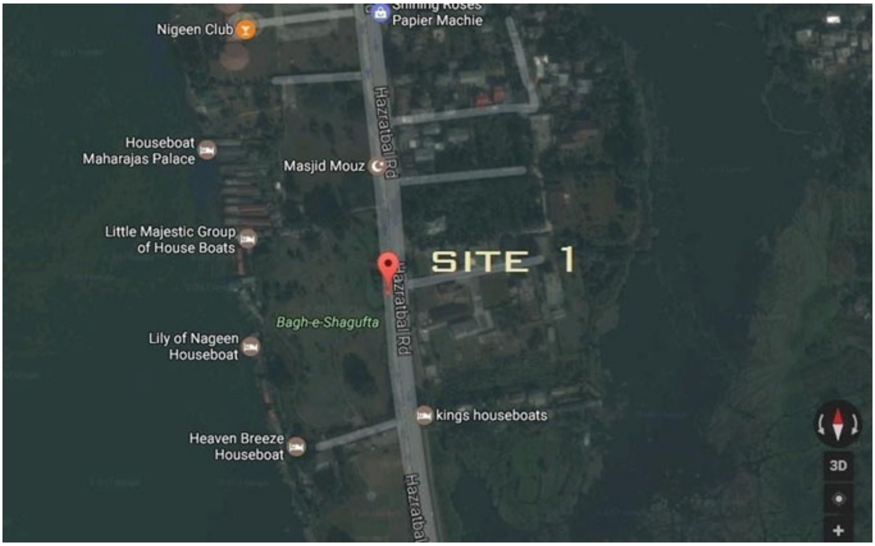


Fig. 2 Site description (*Source* Google earth)

Volume: Hourly and sub-hourly volume were collected at the selected stations for a time period of 17 h. It was done by manual counting method.

Speed: Spot speed at the selected stations was collected for one hour in both directions using the stop-watch method. In this method, a thirty-meter stretch on road was selected and time taken by a vehicle to travel this stretch was recorded. From known values of speed and time, the speed of the vehicle was estimated.

The data for Volume Studies and Spot speed studies were conducted on five sites located on the road connecting NIT Srinagar and Dal gate. The Volume Data counts were performed for 17 long hours on each site, rest 7 h data was extrapolated. Requisite PCU factors for each type of vehicle were used to convert the count into equivalent PCU factors. Standard Vehicle classification categories were taken as shown in Appendix. The spot speed studies were conducted for congested hours. The table represents the data for two peak hours and the respective space mean speed and time speed. The density, time headway, and space headway were thereof calculated from the data obtained.

Location: Dal Gate (Table 1, 2 and 3)

Table 1 Details of vehicular volume data (PCU's) and time period

Time	Total vehicle in PCU's
12 a.m. to 1 a.m.	1269
1 a.m. to 2 a.m.	1514
2 a.m. to 3 a.m.	1437
3 a.m. to 4 a.m.	869
4 a.m. to 5 a.m.	736
5 a.m. to 6 a.m.	602
6 a.m. to 7 a.m.	3103
7 a.m. to 8 a.m.	1264
8 a.m. to 9 a.m.	2235
9 a.m. to 10 a.m.	2802
10 a.m. to 11 a.m.	3118
11 a.m. to 12 p.m.	2938
12 p.m. to 1 p.m.	3169
1 p.m. to 2 p.m.	3097
2 p.m. to 3 p.m.	2872
3 p.m. to 4 p.m.	3317
4 p.m. to 5 p.m.	3330
5 p.m. to 6 p.m.	2981
6 p.m. to 7 p.m.	2614
7 p.m. to 8 p.m.	1607
8 p.m. to 9 p.m.	1064
9 p.m. to 10 p.m.	1284
10 p.m. to 11 p.m.	631
11 p.m. to 12 a.m.	2056
Total	49,909

6 Data Analysis

Hourly volume has been obtained for 17 h for 5 sites. Rest 7 h were extrapolated. (Appropriate PCU factors were used as per recommended by IRC.) Average spot speeds at every site for each type of vehicle was obtained from collected speed data.

1. Time mean speed and space mean speed were calculated accordingly.
2. Average space and Average time headways are calculated.
3. Frequency distribution of speed was plotted.
4. Graphs between speed versus volume and volume versus density have been used. As data points correspond to a lesser range therefore linear variation was assumed wherever required.

Table 2 Details of speed data (2 peak hours)

		1st hour	2nd hour	Time mean speed	Space mean speed
Trucks	Up	23.9	21.2	22.55	22.46
	Down	24.1	21.8	22.66	22.6
Full bus	Up	19.5	34.1	27.07	25.06
	Down	19.5	34.8	27.78	25.59
Mini bus	Up	21.3	34.2	30.4	29.02
	Down	21.4	33.2	26.67	25.44
Car/Jeep/Taxi/Van	Up	23.9	28.6	26.26	26.05
	Down	24.6	24.3	24.43	24.43
3-Wheeler	Up	21.9	27.6	24.82	24.49
	Down	20.8	27.5	24.61	24.14
2-Wheeler	Up	28.4	34.2	30.98	30.72
	Down	29.8	32.6	31.01	30.95
Average speed	Up	23.15	29.98	27.01	26.3
	Down	23.37	29.03	26.19	25.52

Table 3 Details of density, time headway, space headway (Congestion hours)

Speed	Volume	Density	Time headway	Space headway
Kmph	Appl	Vpkpl	S	M
23.15	1689	72.3	2.13	13.83
27.4	1724	59.4	2.088	16.83
32.4	1468	46.9	2.45	21.32
23.7	1641	70.9	2.19	14.1
24.3	1593	53.1	2.25	18.83
31.3	1542	51.2	2.33	19.52

6.1 Modeling Traffic Stream Parameters

The traffic volume count for complete 24 h was plotted on a day-time scale for all the sites. Statistical studies on spot speed were done to obtain the 99th, 85th, and, 15th percentile speed and median speed. Similarly mean deviation, standard deviation, and mode were obtained. The plot for speed versus volume, volume versus density was determined to figure out the relationship between the parameters. Consequently, the volume to capacity ratio was computed to determine the level of service for each zone.

7 Results and Data Analysis

The data for traffic volume Studies and Spot speed studies were conducted on five sites located on the road connecting NIT Srinagar and Dal gate. The traffic volume Data counts were performed for 17 long hours on each site, rest 7 h data was extrapolated. Requisite PCU factors for each type of vehicle were used to convert the count into equivalent PCU factors. Standard Vehicle classification categories were taken as shown in Appendix. The spot speed studies were conducted for congested hours. The table represents the data for two peak hours and the respective space mean speed and time speed. The density, time headway, and space headway were thereof calculated from the data obtained.

Site: Dal gate: The pie chart represents volume composition for different vehicles for urban roads (Figs. 3 and 4)

The graph represents the volume count for different vehicles and different timings of a given location (Fig. 5).

The graph represents speed v/s percentage of the cumulative frequency of a given location (Fig. 6).

85th percentile speed	28.03003
99th percentile speed	29.48545
50th percentile speed	24.39148
15th percentile speed	20.75293

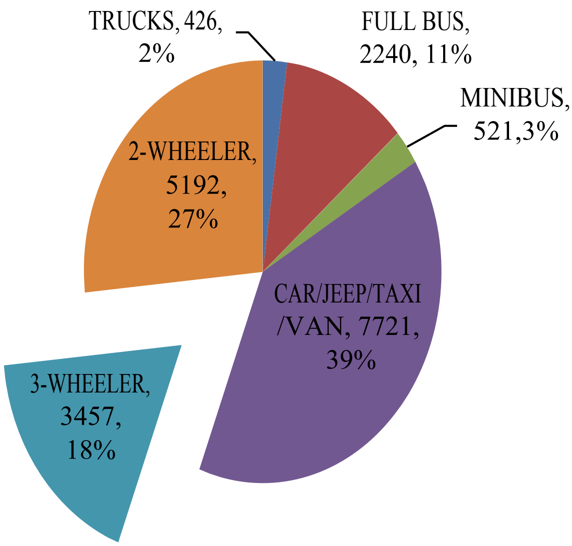
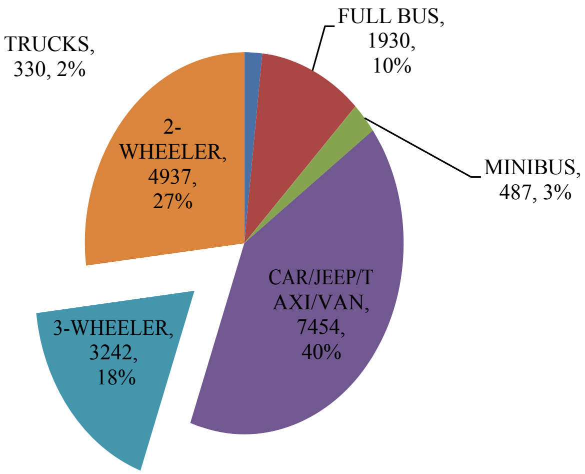


Fig. 3 Volume Composition [Nit To Dal gate]

Fig. 4 Volume composition (Dal gate to NIT)



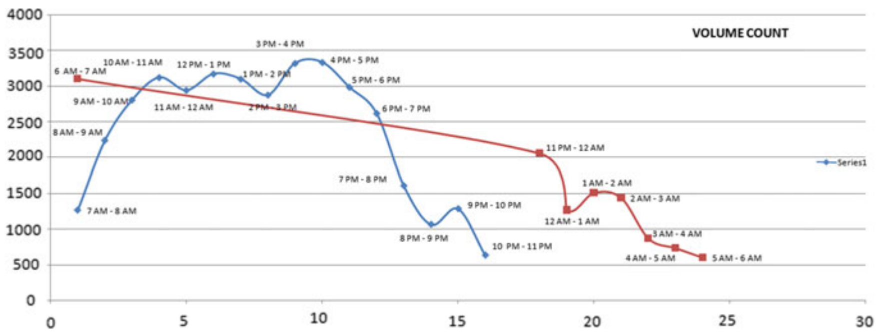


Fig. 5 Volume count graph (Extrapolated data points are shown by a different trendline)

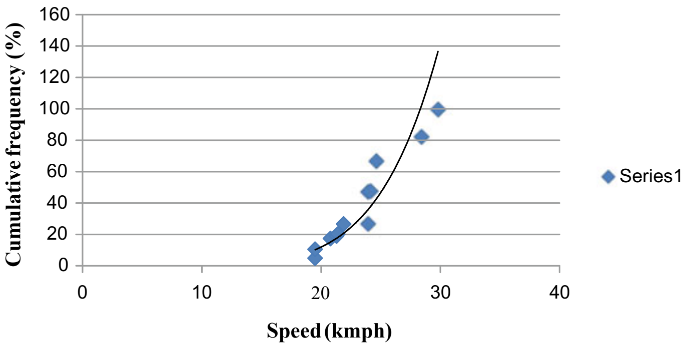


Fig. 6 Speed distribution curve

8 Model Results on Urban Roads

The following details of speed and volume for Dal gate area in the urban road under mixed traffic conditions.

Site: Dal gate (Fig. 7)

Note: Although the relationship between speed versus volume is parabolic, we have assumed linear variation is assumed in this case as data points correspond to a lesser range

$$\text{Equation}: \ y = -0.035\,x + 75.88$$
$$\text{Free mean speed} = 75.88\ \text{kmph}$$
$$\text{Saturation volume} = 2168\text{vph}$$

The following details of density and volume for Dal gate area in the urban road under mixed traffic conditions (Fig. 8).

$$\text{Equation}: \ y = -0.858\,x^2 + 110.6\,x - 1845$$
$$\text{Jam Density} = 128\ \text{vpkpl}$$
$$\text{Volume to capacity ratio} = 0.87$$
$$\text{Level of service (LOS)} = \text{D}$$

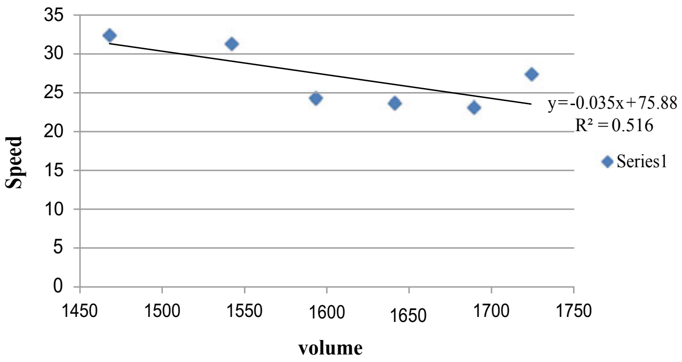


Fig. 7 Graph showing variation between speed and volume

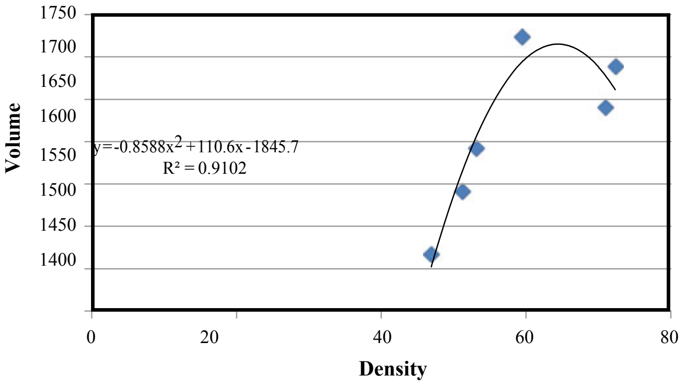


Fig. 8 Graph showing variation between volume and density

9 Conclusion

In this study, collected data from various sites chosen on road from NIT Srinagar to Dal gate were surveyed, analyzed and general traffic congestion trend was generated. Traffic volume count performed on the road segments can be effectively used in the future for design and modeling purposes. Percent traffic volume composition of each class of vehicle was determined. This data can be used to predict the composition of vehicles in a mixed flow in future years. Spot speed was analyzed for peak hours at each site which showed that the sites under consideration were not performing satisfactorily in peak hours. Rainawari was found to be the major congestion zones in peak hours. The capacity of this segment was frequently exceeded resulting in traffic jams. The level of service of each road segment was calculated separately. Except for the Khanyar site, no other road segment was found to meet the overwhelming traffic demand in peak hours. The relationship between volume and density showed a general trend as indicated in Greenberg's model. Separate statistical studies on speed indicated that the roads were primarily urban in nature and their classification ranged from collector streets to sub-arterial streets. Our research, therefore, comes to the conclusion that the concerned authorities should take proper remedial measures to control congestion issues on the road link between NIT Srinagar and Dal gate, especially Rainawari area.

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Appendix

Field datasheet for traffic volume count

TRAFFIC CENSUS
FIELD DATA SHEET

DATE AND DAY OF WEEK:- ____________________
ROAD CLASSIFICATION:- ____________________

KILOMETRAGE/MILEAGE:- ____________________
UP
ROUTE NO (if Any):-____________________
DIRECTION OF TRAFFIC ---------- FROM____________TO____________
DICTRICT:-____________STATE:- ____________
DOWN

HOURS OF COUNT	TRUCK	BUSES	MINIBUS / MINI TRUCK	CAR, JEEP , VANES, TATA SUMO	THREE VEHICLE S	MOTOR CYCLE	ANIMAL -DRAWN VEHICLE	OTHER e.g. HAND CART ETC.	REMARKS, INCLUDING WEATHER CONDITIONS
1	2	3	4	5	6	7	8	9	10
FROM:- ______ TO:- ______									
FROM:- ______ TO:- ______									
HOURLY TOTAL									
FROM:- ______ TO:- ______									
FROM:- ______ TO:- ______									
HOURLY TOTAL									

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User Perception of Automobile Level of Service: Tracking Traffic with GPS Enabled Mobile Phones

Drisya Manghat and Krishnamurthy Karuppanagounder

Abstract The Highway Capacity Manual 2010 (HCM 2010) specifies the importance of user perception in evaluating the Automobile Level of Service (ALOS). Hence, the objective of this study is set to develop a unified methodology for quantifying the ALOS on the divided urban corridors, based on the automobile user perception. To study the behaviour of automobiles at different flow conditions, speed profiles of test vehicles were collected with the in-vehicle Global Positioning System (GPS) enabled mobile phones. The 'Speed Tracker' application was used to record the travel data in every second along with the location coordinates. To have a wide variety of travel conditions, four divided urban corridors of length varying from 2.9 to 3.8 km were identified in the state of Kerala as the study stretches. The segmental analysis was carried out for studying the speed variation behaviour of the vehicles with corresponding flow values. The Acceleration Noise (AN) and the speed ratio (SR) were found to be the most significant measures for defining ALOS. Non-linear regression analysis was carried out to model these measures of effectiveness. k-means and fuzzy c-means (FCM) clustering algorithms were used to obtain the threshold values for ALOS. Silhouette coefficients were calculated for validating the cluster results, and the results showed that k-means algorithm is giving better results compared to FCM. This method can be used for assessing the quality of four-lane divided urban corridors incorporating the user perception.

Keywords ALOS · GPS · Acceleration noise · Speed ratio · k-means clustering · FCM clustering

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1 Introduction

The tremendous rate of population increase in metropolitan cities witnessing the rise in vehicular traffic on their urban corridors. It is critical to monitor the quality of urban corridors for providing the maintenance and improvement measures on the short-term and long-term basis. As per Highway Capacity Manual 2010 (HCM 2010) the quality measures should reflect travellers' perceptions (i.e. measures should reflect things travellers can perceive during their journey) and should be useful to the operating agencies for taking actions to improve the future quality of service. The urban road with mixed traffic is unique among various transport facilities because its right-of-way is shared by multiple modes of travel, having equal mobility and accessibility. To effectively evaluate the quality of service provided by the facility, current research has proposed multimodal Level of Service (LOS) concept. Six distinct classes of LOS measures suggested for automobile, bicycle, pedestrian and transit categories of road users for defining their quality of travel on a specified facility (HCM 2010; NCHRP 399 1998; Dowling et al. 2008). HCM claims to predict the user perception, but there is little evidence to prove the claim in case of urban streets Automobile LOS (ALOS), where the access is also essential along with the through movement, especially in developing countries (NCHRP 616 2008; Das and Pandit 2015).

To account the actual perception of the automobile user in the ALOS quantification, the microscopic parameters of traffic flow has introduced in this study. For identifying the factors affecting automobile drivers' perception, in-vehicle Global Positioning System (GPS) and questionnaire survey approach have used widely. The GPS data is helpful to analyse the variation in speed at every second while travelling through a corridor (Pecheux et al. 2004; Barth et al. 2004; Brennan et al. 2015; Li et al. 2015). The GPS receivers embedded in smartphones have a significant application for the Traffic Engineering community. These smartphones can provide accurate speed and location of each vehicle in every moment, cost-effectively with the help of network providers. Use of GPS enabled mobile phones in tracking traffic have a major impact in developing countries where traffic monitoring is lacking (Herrera 2008). The present study used this technique for collecting the speed data using the test vehicle (car).

Automobile drivers using a transportation facility evaluates its quality by the speed at which they can travel and its uniformity. They feel most comfortable when they can drive with uniform speed. So, the variability in traffic flow quality of the individual vehicle is a useful measure to communicate the quality of travel in an urban road facility (Jones and Potts 1962; Babu and Pattnaik 1995; Ko et al. 2006). Hence, the primary objective of this study is formulated for developing a suitable ALOS method based on the speed data collected with the help of GPS enabled mobilephones.

2 Background Studies

HCM (2000) delivers a broadly utilised strategy for the quality assessment of urban roads. The ALOS evaluation strategies suggested in this version of HCM are considering the four different urban road classes and the average speed of through vehicles. These road classes were identified based on the average free flow speed of those roads. Six LOS classes were defined for each category of road ranging from ‘A’ to ‘F’, representing the excellent to worst operating conditions. SAIC 2003 (Science Applications International Corporation) studied the performance of automobiles on urban roads and identified the 45 influencing factors. They are broadly classified into four-driver quality requirements like travel efficiency, safety, aesthetic appearance and positive guidance.

Many investigators have tried to formulate quality models for predicting the homogeneous traffic conditions. Homogeneous traffic is comprised with identical vehicles travelling with lane discipline. However, the heterogeneous traffic flow is characterised by a various mix of vehicles, where motorised two-wheelers, cars, three-wheelers, buses and non-motorised vehicles travel without any lane discipline. The traffic characteristics in this location are significantly diverse from homogeneous traffic conditions. Hence, the established models can not be used to define automobiles’ perceived satisfaction. In this concern, the current study concentrates on (1) the study of physical and operational factors (traffic flow parameters and roadway geometrics) affecting the perceived satisfaction of automobiles, (2) identification of the critical variables affecting ALOS, (3) formulation of quality prediction models using regression technique and (4) development of quality criteria based on k-means and fuzzy c-means algorithms and report the better one for the present context.

Stepwise regression analysis can be used to recognise the factors affecting the urban street LOS (Dowling et al. 2008). They identified the influencing factors like the number of stops per mile and the proportion of intersections with left-turn lanes. Ordered logit modelling framework was used to model the urban road LOS. In another study, an efficient and precise use of the signal coordination effects was incorporated for the assessment of urban arterial LOS (Deshpande et al. 2010).

The ALOS evaluation method given in HCM (2010) was integrated from NCHRP projects 3–70 and Bonneson et al. (2008). This method is similar to the HCM (2010) method considering the concept behind it. The ratio between the mean speed of a vehicle on a road and the free flow speed of that same road was used as the primary quality measure. Hence the urban road class concept was removed in HCM (2010). 360 arterial segments in Florida were studied by using this method and found that for shorter and lower speed arterials were not getting excellent or good LOS classes. Most of the arterial segments were rated with moderate to poor LOS values which had better LOS values as per HCM (2010) (Ozkul et al. 2013). So they suggested two service classes based on the posted speed limit.

Clustering is one of the best methods to classify the set of data into groups. Many researchers have used this method to develop the speed ranges to define the LOS classes from A to F. Global Positioning System (GPS) can be used as an effective tool to collect the real time traffic data. Affinity propagation (AP) clustering technique includes both the genetic algorithm (GA) and fuzzy C-means (FCM) is a hybrid clustering technique for the classification of continuous variables (Bhuyan and Mohapatra 2014). Similarly, the Hard competitive learning (Hardcl) clustering technique and the genetic programming (GP) clustering technique were also used to classify the speed ranges of LOS categories (Patnaik and Bhuyan 2016; Das and Bhuyan 2017). The result of these studies showed that the LOS ranges obtained for the heterogeneous traffic are different from HCM (2010) values. The basic reason observed for the variations in the speed ranges is due to the traffic and geometric characteristics of the heterogeneous urban roads in developing countries.

Moreover, mean speed alone cannot represent the quality of travel on an urban road. It affects various traffic, geometric and environmental factors. To analyse these factors speed variation characteristics of automobiles are studied in detail. For developing the ALOS criteria, a comprehensive investigation of influencing factors on heterogeneous traffic was carried out. Next section describes the methodology followed in the ALOS criteria development.

3 Method

Although LOS is a subjective measure, this study attempts to quantify the user perception by using speed variation characteristics. The basic concept used here is that the trip with desired uniform speed gives the maximum satisfaction to the driver and hence the highest ALOS. When the traffic on the road is very light, consciously or unconsciously the driver tries to travel at a steady speed which gives him the maximum comfort during the travel (Jones and Potts 1962). However, with the increase in traffic flow, the mutual interaction between the vehicles increases and the driver has to accelerate and frequently decelerate for his safe travel. This disturbance perceived by the automobile users can efficiently be quantified by using the Acceleration Noise (AN) which is the standard deviation of accelerations or the disturbance of the vehicle's speed from the mean speed (Ko et al. 2006). AN value of lesser magnitude provides a better quality of service on the road user point of view.

If A_i is the rate of change in speed of the vehicle from i_{th} interval to $(i + 1)_{th}$ interval and MA is the mean acceleration, the AN can mathematically express as given in Eq. 1.

$$\mathrm{AN} = \sqrt{\frac{\sum_{i=1}^{N} (A_i - \mathrm{MA})^2}{N - 2}} \tag{1}$$

Along with the variation in speed, the percentage of speed at which the driver can travel comparing the free flow speed of a particular road is also influencing the satisfaction of the user. Hence the term speed ratio (SR) has used here to quantify the amount of speed perceived during the travel. The speed ratio is the ratio of the mean speed of a trip to the free flow speed of that particular road. SR varies from zero to one with a higher driver satisfaction nearing to one. It can mathematically express as given in Eq. 2.

$$\mathrm{SR} = \frac{\text{Mean speed of a trip}}{\text{Free flow speed}} \tag{2}$$

By reviewing the previous literature and analysing the correlation between the field data this study uses the AN as the indicator of the quality parameter along with SR to quantify the ALOS of mixed traffic urban corridors. For identifying the threshold values of these measures of effectiveness, AN and SR values studied in detail for the divided urban corridors. The data mining techniques such as k-means and fuzzy c-means (FCM) clustering applied to AN and SR for defining the ALOS threshold values. The quality of the clusters validated by using silhouette measure. Conventional regression technique applied to determine the AN and SR models.

4 Data Collection and Description

This study uses three different types of data collected from the state of Kerala, India. Four major cities were selected for collecting the corridor traffic data. They are Ernakulam, Thrissur, Calicut and Kannur. Ernakulam city is the commercial capital of Kerala having the highest revenue income in the state. It is at 10°00′ N and 76°15′ E. Thrissur is located at a latitude of 10°31′ N and a longitude of 76°12′ E and it is the central part of Kerala. Calicut city is at a latitude of 11°15′ N and a longitude of 75°45′ E on the west coast of India. Kannur is one of the urbanised regions in Kerala, with over half of its inhabitants living in urban zones. The district lies between a latitude of 11°40’ N and a longitude of 74°52’ E. Four four-lane divided urban corridors selected as the study corridors, one from each city having a length varying from 2.9 to 3.8 km.

The geometric characteristics of the corridors were varying along the length. Hence, segmental evaluation was carried out for the analysis. A segment is a combination of midblock and its preceding intersection as shown in Fig. 1. The first, third

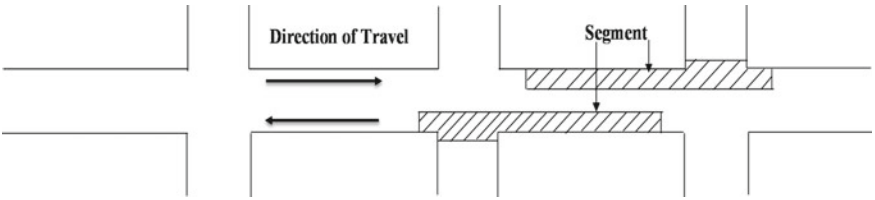

Fig. 1 Representation of urban corridor segments

and fourth corridors were divided into three segments and the second one was divided into four segments depending on the geometric characteristics. Traffic and geometric characteristics of the study segments are given in Table 1. The intersections of these corridors are un-signalised in the study stretch. The data from segments ($13 \times 2 = 26$ segments) were collected for the ALOS analysis.

From Table 1, it is clear that the width of the divided study corridors is varying from 6 to 8.60 m, and the length of segments are varying from 0.45 to 1.70 km. Maximum flow is ranging from 1770 to 3879 PCU/h. The number of approach roads means the number of local roads connecting the traffic to the urban road segments. It increases with the increases in length of the segment and the intensity of urbanisation. Hence, the magnitude is varying from zero for the segment having a length of 0.45 km at Thrissur to a number of fourteen for the 1.70 km length segment at Ernakulam. Distance from intersection shows the location of video graphic camera fixed for recording the traffic flow. Care has taken such that the camera location has less interference due to the performance of intersections. For the collection of speed data GPS based mobile application 'Speed Tracker' was used. The smartphone is having this application kept inside the test vehicle for the speed data collection. Free flow speed data also collected using the same application. Speed tracker gives the speed at every second, distance covered, delay and the total travel time taken. It helps to get all the necessary trip statistics. By starting the application, it will automatically record the speed, time, distance and location coordinates. The application records and saves the trip logs within the phone. The data can export in formats like .csv, .kml and .gpx. The accuracy of the data was checked with the speedometer values and the Trimble GPS values. The accuracy of the mobile application was 99.7% with the actual measurement.

The peak and off-peak speed data collected for around eight and a half hours along with volume data. Speed data collected using the test car, with one trip in every 15 min from morning 6.00 a.m. to 11.00 a.m. and evening 2.30 p.m. to 6.00 p.m. from the study stretches. Total of 484 trips was made to understand the traffic characteristics of the study corridors. Sample output obtained from the application is shown in Table 2. The geometric details of the study corridors collected manually. The composition of vehicles and the flow details recorded along with the speed data by using the video graphic camera fixed at each segment. Five-minute flow values were calculated for each segment and converted to PCU/h/m by using Passenger Car Equivalency factor given in IRC (Indian Road Congress) 86-1983.

4.1 Smoothening of Instantaneous Velocity

Test vehicles with in-vehicle mobile GPS collected the speed data on each second for the every 15 min interval for the specified time period above. The segmental analysis is performed to understand the quality of travel in each corridor. The instantaneous speed recorded using the GPS was fluctuating extensively without showing any particular trend in the data as shown the dotted line in Fig. 2. For eliminating the

Table 1 Characteristics of the segments on the study corridors

Segment No	Road width (m)	Segment length (km)	Max flow (PCU/h)	Max flow (PCU/h/m)	No. of approach roads	Bus stops	Parking	Distance from intersection (m)	Other features which affect the speed
Towards the city centre									
1	6.9	0.6	3672	532.17	1	1	Yes	300	Eye hospital
2	7.35	1.3	3054	415.51	5	2	Yes	400	Hotel, Supermarket
3	6.7	0.90	1878	280.29	3	1	No	450	Hotel, Bank
4	7.4	0.92	2640	356.76	5	2	No	200	Shopping mall
5	6.5	1.6	3486	536.30	8	2	No	700	School, Temple
6	6	0.90	2358	393	3	1	No	200	Temple
7	6.8	0.45	2106	309	1	1	No	400	Shopping mall
8	6.5	1.0	2916	448.61	2	1	No	150	–
9	7.2	0.6	2893	401.80	1	2	Yes	325	Hospital, Hotel
10	6.5	0.90	2916	448.61	2	1	Yes	200	Temple
11	8.6	1.7	3742	435.12	13	3	Yes	500	Temple
12	6	1.2	3495	582.5	5	2	No	450	Stadium
13	8.4	0.5	3879	461.79	2	1	No	120	School, District court
Away from the city centre									
1	7.35	0.6	3570	485.7	1	1	Yes	300	Hotel, Bank
2	7.10	1.3	3270	460.56	1	2	No	200	Hospital
3	6.7	0.90	1998	307.38	2	0	No	450	Park, Hospital

(continued)

Table 1 (continued)

Segment No	Road width (m)	Segment length (km)	Max flow (PCU/h)	Max flow (PCU/h/m)	No. of approach roads	Bus stops	Parking	Distance from intersection (m)	Other features which affect the speed
4	7	0.92	2922	417.43	4	2	No	200	Hotel
5	6.9	1.6	2214	320.87	4	3	Yes	700	Auditorium, School
6	6.2	0.90	1770	285.48	3	1	Yes	200	–
7	7	0.45	1872	267.42	0	1	Yes	400	Hospital, Stadium
8	6.7	1.0	2022	301.79	2	1	Yes	150	Super market
9	7	0.6	2394	342	2	2	Yes	325	Temple
10	5.85	0.90	2694	460.51	1	2	No	200	Church, Supermarket
11	6.3	1.7	3324	527.61	14	4	No	500	Church
12	6.8	1.2	3662	538.53	7	2	No	450	Shopping Mall
13	6.5	0.5	3604	554.46	2	1	No	120	Church, Book stall

Table 2 Sample output from speed tracker

Date	Time	Elapsed time	Distance (km)	Speed (km/h)	Latitude	Longitude
28/03/17	3:39:45 a.m. GMT + 5:30	00:00	0	11.5	9.995526	76.2926
28/03/17	3:39:46 a.m. GMT + 5:30	00:01	0	11.8	9.995545	76.29262
28/03/17	3:39:47 a.m. GMT + 5:30	00:02	0.01	14.4	9.995579	76.29266
28/03/17	3:39:48 a.m. GMT + 5:30	00:03	0.01	17.8	9.995605	76.29272
28/03/17	3:39:49 a.m. GMT + 5:30	00:04	0.03	24.6	9.995685	76.2928
28/03/17	3:39:50 a.m. GMT + 5:30	00:05	0.03	26	9.995725	76.29286
28/03/17	3:39:51 a.m. GMT + 5:30	00:06	0.04	26.9	9.99576	76.29292
28/03/17	3:39:52 a.m. GMT + 5:30	00:07	0.05	28.6	9.995809	76.29298
28/03/17	3:39:53 a.m. GMT + 5:30	00:08	0.06	28.5	9.995866	76.29303
28/03/17	3:39:54 a.m. GMT + 5:30	00:09	0.06	21.7	9.995897	76.29309

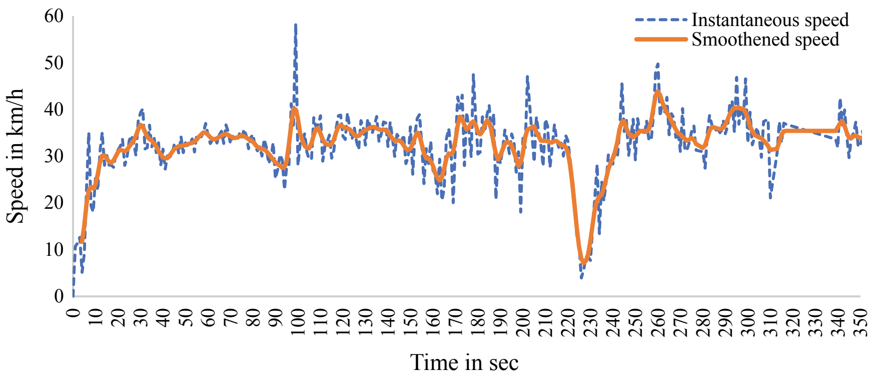

Fig. 2 Smoothening of speed profile data by using 3MA

randomness, the data treated with moving average algorithm in which each average is calculated by removing the previous instantaneous speed data and adding the next speed data point. The averaging continues through the data points until at every speed data for which all components of the average are accessible.

k MA or *k*th order moving average of instantaneous speed data can calculate by using Eq. 3 given below. T_t represents the moving average speed at time *t*. In this study, k was set to 3. Y_t is the instantaneous speed of the test vehicle at time t.

$$T_t = \frac{1}{k} \sum_{j=-m}^{m} Y_{t+j} \quad (3)$$

where k is an odd integer and

$$m = (k - 1)/2$$

By applying 3MA algorithm, the instantaneous speed points averaged and smoothened pattern of vehicle trajectory drawn as shown in Fig. 2. The smoothened data is used for further analysis.

5 Automobile Travel Behaviour Analysis

5.1 Acceleration Noise

AN is reflective of the overall behaviour of traffic, and a direct measure of discomfort received by the driver during his travel. The AN values starting from 0.92 km/h/s in free flow showing a maximum value of 3.64 km/h/s in a congested condition. The behaviour of this unconventional parameter of traffic flow in the heterogeneous traffic condition is studied in detail for understanding the quality of travel on the road user point of view. The primary reason for the maximum value of AN for the automobiles is due to the higher flow rate with roadside parking and the interference due to the direct entry and exit of other vehicles from the adjacent side buildings. It will affect the smooth movement of through traffic and results in higher values of AN.

To predict the AN values for automobiles conventional regression modelling approach was applied by using statistical software. The significant factors affecting AN were identified by using Pearsons' correlation test. The best-fitted model forms obtained is given by the following Eq. (4). The model predictions are reasonably good with an R^2 value 0.591 and RMSE (Root mean squared error) of 0.335. The parameter estimates of the models are given in Table 3. All the variables except mean speed are having a positive relationship with AN. When the flow increases, the interaction between the vehicles also increases, which will tend to increase the standard deviation of acceleration. Higher mean speed indirectly means the lower traffic volume, and hence when the mean speed increases, the AN shows a decreasing trend. Hence all the model forms show a logical relationship with the right predictions.

$$\text{AN} = a \times Q^{\text{b}} + c \times L + d \times \text{proportion of car} + e \times \text{MS} \quad (4)$$

Table 3 Parameter estimates for acceleration noise models

Mode	Parameter	Estimate	Std. Error	95% Confidence interval	
				Lower bound	Upper bound
Car	a	0.052	0.020	0.012	0.092
	b	0.643	0.061	0.523	0.762
	c	0.041	0.039	−0.034	0.117
	d	0.325	0.190	−0.050	0.699
	e	−0.003	0.002	−0.004	0.007
	R^2	**0.591**			
	RMSE	**0.335**			

where

MS Mean Speed in km/h.
Q Flow in PCU/h/m.
L Segment length in km.

5.2 *Speed Ratio*

Speed Ratio is the ratio of the mean speed of a vehicle travelling in a traffic stream to its free flow speed. Since each urban road has specific geometric characteristics, the free flow speed varies accordingly. Previously there were four classes of urban roads depending on their free flow speed. To standardise the criteria for all urban roads, SR has introduced for ALOS criteria development. For all the study corridors SR was calculated in low, medium and heavy traffic conditions. The SR values observed ranging from 0.23 to 1.0, depicting the congested to the free flow condition.

The traffic volume, proportion of vehicles, geometry of the road and the free flow speed of the vehicles can be collected easily in an urban corridor. By considering these factors as the independent variables, the model was developed to predict the mean speed for quantifying the ALOS. The correlation between the variables is calculated, and the independent variables were selected for modelling. For the observed dependent variables, the linear regression models were not providing the right prediction. Hence non-linear regression modelling approach was adopted for predicting the LOS measures. The influence of these variables on the user perception of quality is quantified by developing the models. Many model combinations were tried, and the best-fitted model forms shown in the following Eq. 5.

$$\mathrm{MS} = a \times Q^{\mathrm{b}} + c \times W + d \times \text{proportion of car} + e \times \mathrm{FFS} \tag{5}$$

where

MS Mean Speed in km/h.

Table 4 Parameter estimates for mean speed models

Mode	Parameter	Estimate	Std. Error	95% Confidence interval	
				Lower bound	Upper bound
Car	a	−0.389	0.379	−1.134	0.356
	b	0.749	0.143	0.468	1.029
	c	1.760	0.430	0.914	2.606
	d	−18.735	3.565	−25.739	−11.731
	e	0.835	0.055	0.728	0.943
	R^2	**0.603**			
	RMSE	**6.483**			

Q Flow in PCU/h/m.
FFS Free Flow Speed in km/h.
W Carriageway width in meters.

The estimated model parameters are given in Table 4. By analysing the parameter values, it can see that for the first independent variable, the model is showing a negative relationship which is logically very much true. When the flow increases the interaction between the vehicles increases, and the overturning opportunities reduce, which will tend to the reduction in mean speed. The second variable considered is the width of the carriageway, which is showing a positive relationship with mean speed. When the width of the road increases, the availability of space to manoeuvre increases, the overtaking manoeuvre increases, and hence the mean speed also increases. The third variable is the proportion of cars is negatively related to mean speed. When the proportion of car increases, the availability of space decreases and hence the mean speed also decreases. The last variable considered is free flow speed which is positively related to mean speed. Depending on the type of road, geometry, and the condition of the pavement, its free flow speed varies. When the free flow speed is high, the mean speed also will be high for that particular segment. So, all the model parameters can logically explain within a 95% confidence interval.

6 Threshold Values for Alos Rating

6.1 Cluster Analysis

Cluster analysis refers to a useful method for classifying or grouping of a collection of data points into subsets or groups, so that the members within a cluster are closely related, compared to the other cluster members. This study analyses two different clustering techniques, k-means and FCM to classify the ALOS thresholds for heterogeneous urban corridors. Six classes of SR and AN ranges obtained with letter grading A to F. ALOS A denotes the best quality of travel and ALOS F denotes the worst. k-means and FCM clustering methods are described in detail in the following sections.

6.1.1 k-Means Clustering

k-means is a non- hierarchical partitioning technique. In this clustering technique, within-cluster variation is used to form the homogenous clusters. The algorithm classifies the information in a manner that, inside the group, the variation is minimised. Here the user can allocate the number of groups required. Euclidian distance calculated from each centre on all distinct values. Every single value allocated considering the shortest distance to the cluster centre. k-means algorithm is superior to the other clustering methods since the presence of irrelevant variables and the effects of outliers are less affected by it.

"This clustering has three steps as follows

1. Divide all the variables into k initial clusters.
2. Calculate the centroid distance. Allocate the variables to clusters with less centroid distance. Again calculate the centroid distance with the new cluster variables and assign them to the least centroid distance clusters.
3. Step 2 has to be repeated till the variation in cluster membership becomes zero.

This method minimises the sum of variances within the cluster, which is given as follows

$$V_K = \sum_{k=1}^{K} \sum_{i=1}^{n} \delta_{ik} m_i d^2 (x_i - \bar{x}_k) \tag{6}$$

where $\delta_{ik} = 1$, if $i \in k$ or zero otherwise.

Moreover, the element x_{kj} of the vector x_k is the mean value of the variable j in the cluster k.

$$x_{ij} = \frac{1}{n_k} \sum_{i=1}^{I} \delta_{ik} m_i x_{ij}" \tag{7}$$

k-means clustering algorithm was applied using R software to the observed SR and AN values. Six groups were obtained for each parameter, which deviates from HCM (2010) suggested SR ranges for homogeneous traffic condition. The variation from HCM (2010) criterion values is due to the heterogeneity of the traffic, on-street parking, influence of pedestrian crossings, roadside encroachments and the non-lane based traffic movements in the developing countries.

The classification of SR and AN values attained by k-means clustering and the SR ranges recommended by HCM (2010) and Indo HCM (2017) are given in Table 5. The graphical representation of the clusters is shown in Fig. 3. Each cluster were

Table 5 Proposed threshold values of SR and AN for ALOS based on k-means and FCM clustering

ALOS classes	SR (Speed ratio)				AN (Acceleration noise in km/h/s)	
	k-means	FCM	HCM 2010	Indo HCM 2017	k-means	FCM
A	>0.80	>0.81	>0.85	>0.84	<1.40	<1.40
B	0.80–0.66	0.81–0.66	0.85–0.67	0.84–0.76	1.41–1.79	1.41–1.79
C	0.65–0.56	0.65–0.56	0.66–0.50	0.75–0.59	1.80–2.19	1.80–2.13
D	0.55–0.46	0.55–0.46	0.49–0.40	0.58–0.41	2.20–2.39	2.14–2.50
E	0.45–0.36	0.45–0.38	0.39–0.30	0.40–0.22	2.40–2.82	2.51–2.92
F	<0.36	<0.38	<0.30	<0.22	>2.82	<2.92

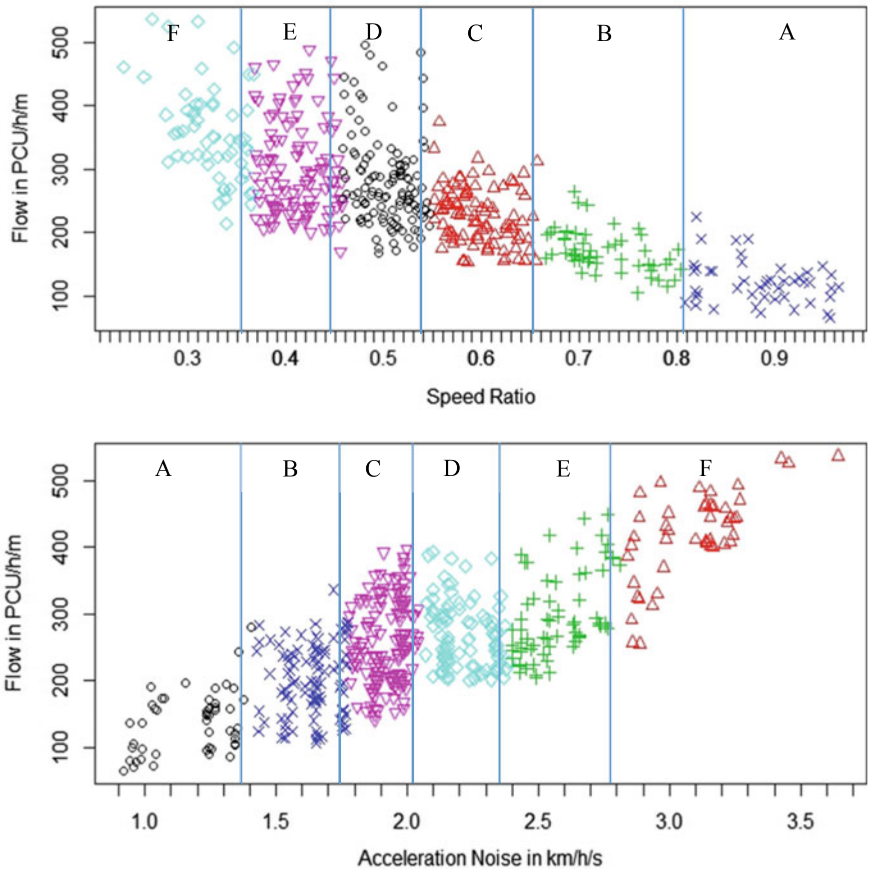

Fig. 3 Graphical representation of k-means clustering result for AN and SR values

differentiated by shapes and colours for better understanding. The class of each group is denoted by letters A to F denoting the ALOS 'A' to ALOS 'F'.

6.2 FCM Clustering

FCM clustering is used to overcome the rigid nature of k-means. Here every point has a probability of being to any cluster, rather than being in a single cluster as the case in the traditional k-means. FCM deals with the problem where points are somewhat in between centres by replacing distance with probability. Weighted centroid based values are used on those probabilities. The methods of initialisation, iteration and termination are similar like k-means. k-means is a particular case of FCM where the probability function is equal to one if the data point is near to a centroid and zero otherwise.

The FCM has four basic steps which are given below

1. Assume the number of clusters k initially.
2. Initialise the k-means μ_k with the clusters. Calculate the probability of each data point x_i is a member of a given cluster k.
3. Recalculate the cluster centroid as the weighted centroid with the probabilities of membership of all data points x_i.

$$\mu_k(n+1) = \frac{\sum_{x_i \in k} x_i \times P(\mu_k|x_i).^b}{\sum_{x_i \in k} P(\mu_k|x_i).^b} \tag{8}$$

4. Continue iteration till convergence or a user-defined number of iterations has been reached (the iteration can stop at some local maxima or minima).

The FCM clustering technique was applied using *R* software, and the results are tabulated in Table 5. The graphical representation of the clusters is shown in Fig. 4. For a better understanding of the result, each cluster were differentiated by using different shapes and colours. The class of each group is denoted by letters A to F denoting the ALOS 'A' to ALOS 'F'.

6.3 Validation Measure for Cluster Quality—Silhouette

The validity of clustering means that whether the given data grouped appropriately by using a specified technique. For the given number of groups, the clustering algorithms always provide the best-fit clusters. However, the clustering algorithms are not providing the significance of the clusters considering the dataset. Sometimes the cluster number may be wrong, or the cluster shapes may be wrong (Bensaid et al.

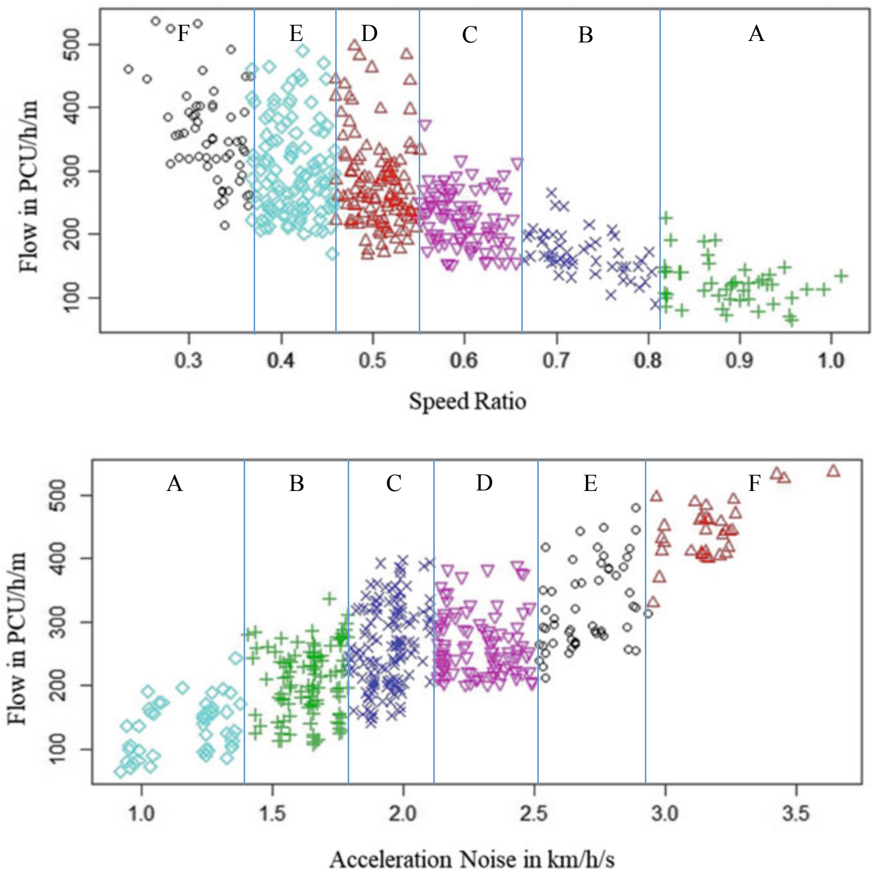

Fig. 4 Graphical representation of FCM clustering result for AN and SR values

1966; Bezdek and Pal 1988). Silhouette is a distance measure which can be used to measure the similarity between the variables in a single cluster. This concept was introduced by showing the graphical representation of silhouettes. The silhouettes can be plotted in a single diagram to compare the quality of clusters. The silhouette value summarises the significance of each cluster.

The silhouette can be calculated by using the following concept.

Consider the cluster A to which the object i is assigned. Calculate $a(i)$.

$a(i)$= average dissimilarity of i to other objects of A.

Consider the cluster C which is different from A. Calculate $d(i, C)$.

$d(i,C)$ = average dissimilarity of i to all objects of C.

$$b(i) = \min_{C \neq A} d(i, C) \tag{9}$$

The silhouette value $s(i)$ can be calculated by using the following equations:

$$s(i) = 1 - \frac{a(i)}{b(i)} \quad \text{if} \quad a(i) < b(i) \tag{10a}$$

$$s(i) = 0 \quad \text{if } a(i) < b(i)a(i) < b(i) \tag{10b}$$

$$s(i) = \tfrac{b(i)}{a(i)} - 1 \quad \text{if } a(i) > b(i) \tag{10c}$$

These equations can be combined as follows:

$$s(i) = \frac{b(i) - a(i)}{\max\{a(i), b(i)\}} \tag{11}$$

Hence, from the above equation, one can easily understand that the magnitude of $s(i)$ is

$$-1 \leq s(i) \leq 1 \tag{12}$$

For every value of $s(i)$ in a particular cluster, the graphical representation can be done by plotting the $s(i)$, ranked in decreasing order for all objects i, in that cluster. All the clusters in a dataset can be represented by a single plot so that the quality of clustering can be validated. The average silhouette width of the whole dataset, denoted by $\bar{s}(k)$, is used for the selection of best quality of cluster, where k is the number of clusters ($k = 2, 3, 4, \ldots, n-1$). The silhouette coefficient (SC) can be calculated as

$$SC = \max_{k} \bar{s}(k)" \tag{13}$$

For a better quality clustering, silhouette coefficient should be greater than 0.5. Researchers identified the use of Fuzzy, Self-organising Map (SOM) and k-means clustering algorithms for classifying the LOS standards (Bhuyan and Rao 2011; Kim and Yamashita 2007; Cheol and Stephen 2002). For validating the quality of the clusters, silhouette coefficients were calculated, and the graphical representation is shown in Fig. 5. Each peak of the silhouette width is showing the magnitude of the silhouette coefficient for clusters 1–6. The performance of both clustering techniques was equal considering the average silhouette width, 0.55 and 0.57 for SR and AN, respectively. For validating the quality of clustering techniques, the silhouette coefficients of each cluster were calculated and compared. Table 6 gives the silhouette coefficient values for k-means and FCM clustering.

From the analysis of the individual silhouette coefficient values, the FCM clustering shows bad quality clusters having values of 0.49 and 0.47 which is less than 0.50. As given in literature k-means is a simple clustering technique and requires less time to execute. The accuracy level of clustering was higher for k-means clustering

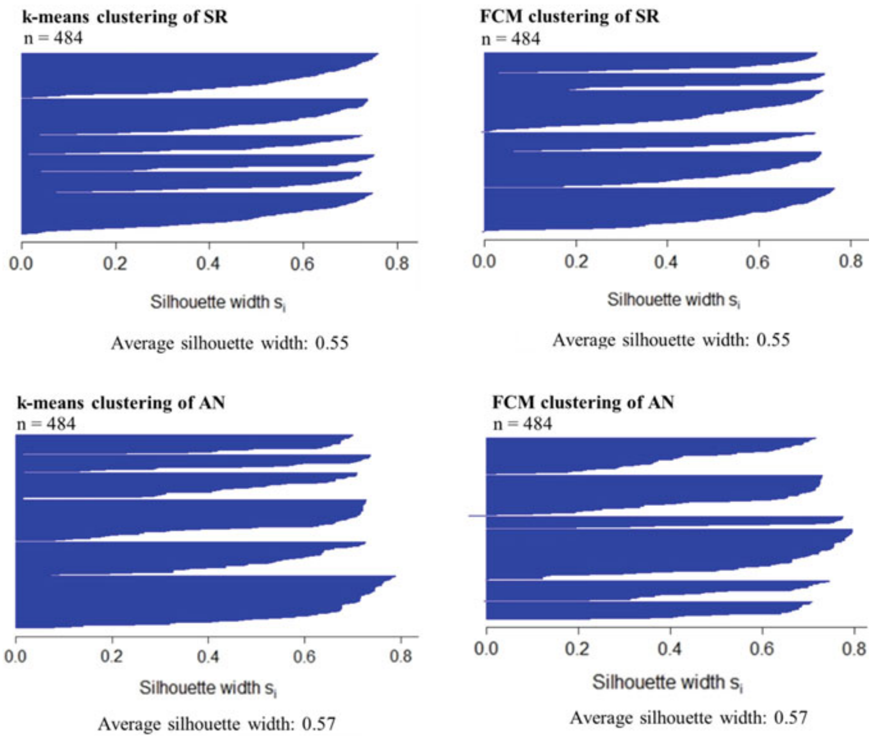

Fig. 5 Silhouette plot for SR and AN thresholds using k-means and FCM clustering

Table 6 Silhouette coefficient values for clusters using k-means and FCM techniques

Cluster no	Silhouette Coefficients			
	SR		AN	
	k-means	FCM	k-means	FCM
1	0.56	0.58	0.59	**0.47**
2	0.56	0.59	0.55	0.55
3	0.53	**0.49**	0.58	0.64
4	0.60	0.52	0.55	0.62
5	0.57	0.56	0.53	0.54
6	0.51	0.57	0.64	0.61
Average	0.555	0.552	0.573	0.572

compared with FCM which is not similar to the results obtained in previous literature (Marisamynathan and Vedagiri 2017; Cebeci and Yildiz 2015). By analysis performed and the results obtained, it can be stated that k-means algorithm gives more reliable classifications for the ALOS under the heterogeneous urban corridors.

7 Conclusions

The mean speed as a percentage of free flow speed and the volume-capacity ratio was widely accepted as the parameters for representing the performance quality of urban roads. Literature review revealed that the macroscopic parameters of traffic flow used in the existing methods do not accurately predict the ALOS at heterogeneous urban corridors. To overcome the gaps in the current research, the authors introduced the microscopic parameters of traffic flow to assess the ALOS. The use of GPS enabled mobile phones made the speed profile data collection smooth, accurate and cost-effective. Travel details of the test vehicle were recorded for every second by using the 'Speed Tracker' application and the smoothened data used for finding the measures of effectiveness. The accuracy of the application was validated with the actual measured values. The speed data collected from four divided urban corridors in Kerala was used for studying the speed variation characteristics of the heterogeneous traffic. The variability in travel speed was found to be the best measure for representing the quality of travel in the user point of view.

The primary measure for predicting the ALOS was AN, whose behaviour was studied and the reasons for variability were described. A non-linear regression model was developed to predict AN. The second measure proposed for predicting the ALOS was the SR, which is the standardised value of MS by considering the corresponding free flow speed of the particular road. Subsequently, MS distribution was studied, and a non-linear regression model proposed for predicting the ALOS and validated with field data. Two methods of clustering were used in this study to get a significant threshold values for ALOS criteria. The study results were compared with Indo HCM (2017) and HCM (2010) SR values. The authors demonstrated the use of data mining techniques for finding the threshold values of quality of urban corridors using k-means and FCM algorithms. The two methods were compared by plotting silhouette coefficients. The study results concluded that k-means algorithm performs better and more accurate for defining ALOS.

The results from this study will help the traffic engineers and planners to understand the current quality of urban roads in the user point of view. It helps them in planning and improving the urban roads for higher ALOS.

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Travel Time Delay Study on Congested Urban Road Links of Ahmedabad City

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Abstract Highly increasing vehicle population results in transportation problems, and thus it leads to the requirement of more efficient transportation systems. Indian urban road traffic consists of different types of vehicles with different sizes, speed, and maneuverability moving on the same right of way without lane discipline and without obeying general traffic rules. They create chaos on junctions and result in enormous delays, more fuel consumption, air and noise pollution, accidents, and also restrict the movement of emergency vehicles. Therefore, it is necessary to quantify the travel time delay in existing traffic conditions. The aim of this study is to quantify the delay in travel time and to identify its causes. As per that the various traffic surveys have been carried out on selected stretches of Kalupur region in Ahmedabad. In this study, it is proposed to measure the travel time delay and to ascertain the notable factors which result in these delays. License plate method and GPS in floating car method are used to collect travel time data during peak hours in the morning and evening, respectively. With the help of the data collected, Flow–Delay model is generated. Average travel time observed in this area is about 15–20 min/km during evening peak hours. It is found that the travel time delay depends on traffic flow, its composition, slow-moving vehicles, haphazard movement of vehicle without lane discipline, boarding alighting of passengers on the bus stops, and rickshaw stops. It is suggested to provide an efficient mass transport system at separate levels in this area.

Keywords Congestion · Travel time delay · License plate method · Mass transport system

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1 Introduction

Transportation plays a vital role in the development and growth of any nation. Transportation is significant for trade, organization, and social interaction while consuming considerable time and resource. Transportation sector in India is a very extensive system, comprising different modes of transport like Road, Railway, Aviation, Waterway, and Shipping, which facilitate easy and different conveyance of goods and people across the nation. The backbone of the economic development of India largely depends on its transportation. Road transportation system is the primary mode of transportation which plays an important role in the conveyance of goods, passengers and linking the centers of production, distribution, and consumption.

India is also one of the rapidly developing Nations. So, it is a necessity to have an adequate transportation facility to meet the existing and future transportation demand. India has experienced tremendous growth rate in vehicle growth, an average growth rate of 9% per year in the country. Some analysts predicted that India's motorization rate will continue to grow to 40 vehicles per 1000 by 2020 (Road Transport Year Book, 2012). Ahmedabad is one of the congested cities of India due to its large vehicle population. Pollution is the final result of traffic congestions. Ahmedabad is well known as an economic and industrial hub in India. It is the second-largest producer of cotton in India. Ahmedabad is the prime city of Gujarat as well as among the most congested cities of India because it consists of 1,682,111 vehicles including all types (RTYB, 2012).

1.1 Problem Statement

Due to various types of vehicles running on the same right of way of the urban streets tremendous delays are experienced during peak hours. In Indian traffic condition generally two-wheelers, three-wheelers, car, bus, truck, LCV, bicycle, nonmotorized vehicles moving without lane discipline. This may lead to traffic congestion, stop and go condition, reduced level of service, increase in fuel consumption, air and noise pollution, etc. Therefore, it is interesting to determine the relationship between mix traffic volume and travel time delay on the urban streets. In view of this, the proposed study is carried out to develop above-said relationship on the heavily congested road links of Ahmedabad city.

1.2 Aim of the Study

The aim of this study is to determine the travel time delay and its causes on the selected links of the Kalupur area. Also, it is aimed to develop a relationship between traffic flow and travel time for the different vehicular compositions on the same links.

1.3 Objectives of the Study

1. To determine the various types of delays experienced on the selected stretches.
2. To determine the travel time on selected stretches and its delay.
3. To determine the level of service on selected stretches.
4. To establish a relation between travel time delay and volume of vehicles on selected stretches.

1.4 Scope of the Study

1. Compilation of travel time data that may be used in trend studies to evaluate the change in efficiency and level of service with time.
2. Performance of economic studies in the evaluation of traffic operation alternatives that reduce travel time.
3. Determination of efficiency of a route with respect to its ability to carry Traffic.
4. This study may be useful for planning efficient traffic system management scheme.

2 Terminology Related to Travel Time and Delay

1. **Travel Time (TT)**: The period of time to traverse a route between any two points.
2. **Average Speed (AS)**: For a distance average speed of a test vehicle (in km per hour).
3. **Running Time (RT)**: While driving over a distance, the elapsed time (in seconds) excluding delay.
4. **Running Speed (RS)**: The average speed of a test vehicle (in km per hour) while the vehicle is in motion (does not include delay time).
   It can be calculated by the formula:

$$\text{RS} = \frac{\text{Distance}}{\text{TT} - \text{Delay}}$$

5. **Congestion delay**: The delay caused by the slowing down effect of overloaded intersections, parked cars, crowded pavement, inadequate carriageway widths, and similar factors.
6. **Fixed Delay**: In this delay, the amount of traffic volume and interference present is regardless.
7. **Operational Delay**: It is caused by interference from other components of the traffic stream.
8. **Stopped Delay**: It is the time elapsed in which a vehicle is not moving.

9. **Travel Time Delay**: It is the difference between the driver's expected travel time through the intersection (or any roadway segment) and the actual time taken.
10. **Approach Delay**: Approach delay includes stopped-time delay but adds the time loss due to deceleration from the approach speed to a stop and the time loss due to re-acceleration back to the desired speed.
11. **Delay**: The delay (in seconds) due to driving a vehicle at a speed of less than 5 kmph.

3 Review of Past Studies

John et al. (2009) have carried out travel time and delay survey using GPS system over a six major routes of Auckland, that study conducted was based on HCM (2000) LOS concept on three major arterial routes in Auckland City to measure the Level of Congestion experienced on these routes during peak hours. They concluded that the Ti Rakau Drive and Jellicoe Drive, in these sections, vehicles facing the delay of 8.7 min and the installation of an additional lane may be warranted as the peak flow is limited to one lane. Abojaradeh (2013) tries to encourage people for the use of public transport instead of using private cars by computing different travel time of both, public transit and private cars in between Amman and Sweilleh line in Jordan. The stretch is 10.65 km long and travel time and delay survey carried out in 32 runs by public transit and 10 runs by privet car in both directions. They concluded that there is a very high difference in the average running speed and travel time of both the modes. Das et al. (2016) have carried out congestion modeling by analyzing data of traffic volume survey and travel time delay survey from Udhana Darwaja to Udhana village in Surat. Traffic volume count carried out by videography and travel time delay survey carried out by GPS fitted vehicle with six trips and V-Box apparatus. Micro-level congestion analysis carried out and got Travel Speed Index (TSI) and Traffic Congestion Index (TCI) and stated that traffic flow is at the desired speed. Mahbub (2010) has explored travel time variability in Dhaka city. Travel time and delay data collected by GPS.He observed that the higher travel times in the morning off-peak and afternoon peak have the lower coefficient of variance but the smaller mean travel time in the morning peak has a higher variability than other times of the day. Solanki et al. (2016) have studied travel time delay and traffic flow under heterogeneous traffic conditions on CBD area (Surat–Rajmarg) of Surat City. They found maximum travel time on Saturday morning and evening peak hours.

4 Methodology

Methodology flow chart of the study is shown in Fig. 1.

4.1 Classified Volume Count Survey

As videography is a more convenient and accurate method for classified volume count, it was commenced for the study. It does not require more manpower and skilled person. To study the fluctuation of traffic volume per 5 min, videography was conducted at selected stretches and Traffic Volume count surveys carried out from 9 pm to 7 am for the one day. The camera was situated on the elevated building with suitable visibility of stretch length. The height of camera may be different for each stretch but not less than 4 m on any stretch. The classified volume per 5 min is calculated from video display.

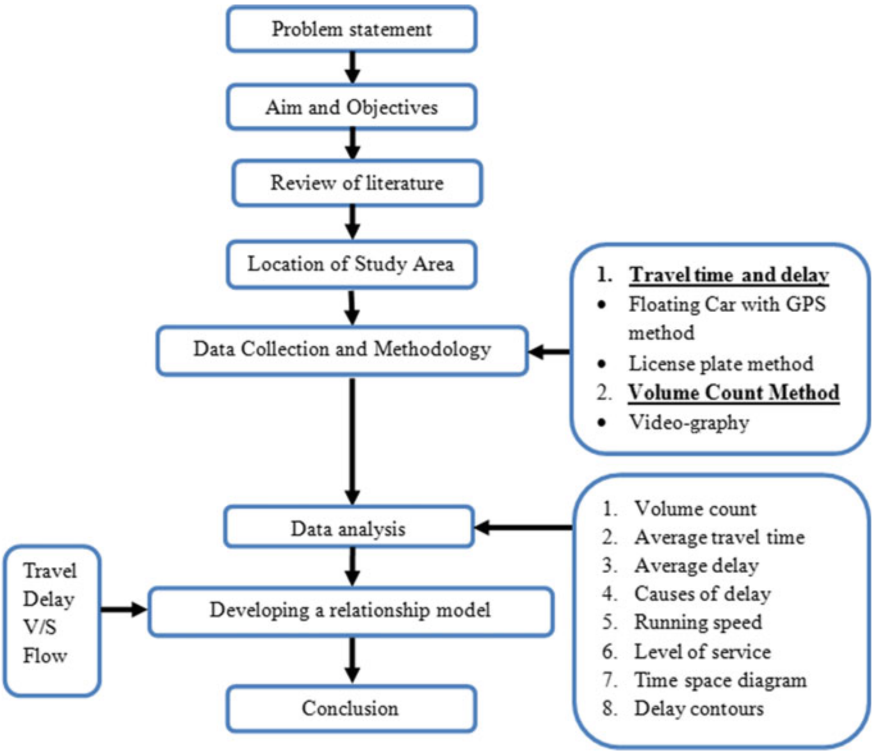

Fig. 1 Methodology flow chart

4.2 Travel Time and Delay Study Survey

Travel time and delay study measures average travel time and running times along section of route, while at the same time information regarding location, cause, and duration of delay is collected. Travel time data are good indicators of the level of service which is being provided and can be used as a relative measure to evaluate the efficiency of traffic flow.

Following methods for conducting travel time and delay study

1. Floating car method
2. License plate method
3. Photographic method
4. Interview method
5. Elevated observation method
6. GPS Method.

Each method has its own merits and demerits relative to the other. A particular method can be adopted depending upon the degree of accuracy required, cost, and time and by the practical limitations. In this study License plate method and GPS in floating car method are used to collect travel time data during peak hours in the morning and evening respectively.

5 Study Area and Data Collection

The maps of Kalupur area of Ahmedabad and selected stretch are shown in Figs. 2 and 3.

5.1 Geometry of Selected Stretches

Table 1 shows geometric details of selected stretches.

5.2 Analysis of Data

The data collection of classified volume count is carried out by videography. Travel time and delay is collected with GPS. License plate method and videography is used to collect Travel Time data.

Table 2 shows the total classified traffic volume on the selected stretches. Figure 4a shows the vehicle composition and Fig. 4b shows the hourly fluctuation of traffic flow of every 5-min interval of Sarangpur tank to Kalupur railway station stretch.

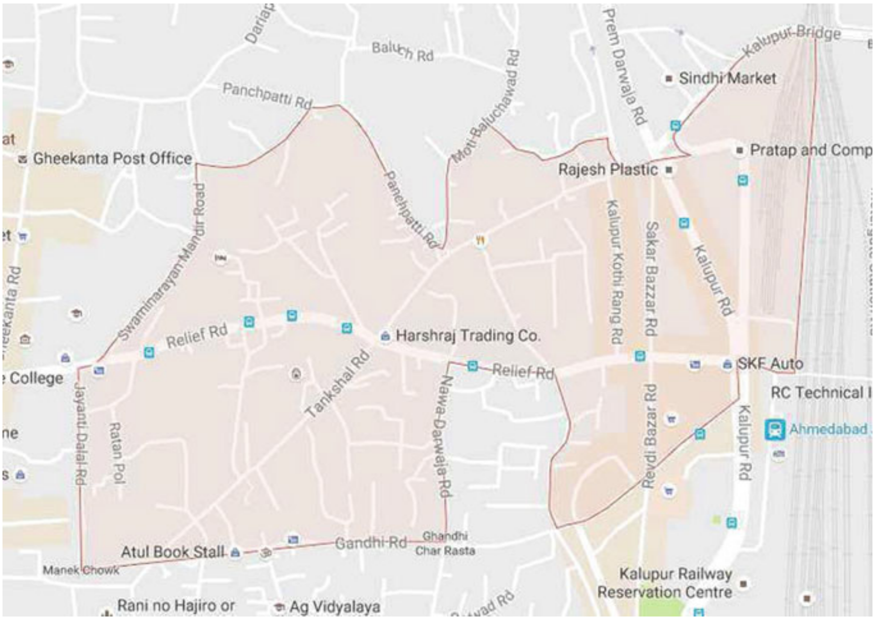

Fig. 2 Kalupur area of Ahmedabad

The same type of data analysis is carried out on all selected stretches. However, due to space constraint, details of only one stretch Sarangpur tank to Kalupur railway station is shown in this paper. The speed profile and time-space diagrams are obtained from GPS applications installed in mobile phone while traveling in a floating car with the traffic stream during the morning (Free flow condition) and evening peak periods. Figures 5, 6, 7 and 8 are showing the speed profile and time-space diagrams of the morning (Free flow condition) and evening peak periods for the stretch of Sarangpur tank to Kalupur railway station.

5.3 Flow–Delay Relationship from License Plate Method

To obtain the relationship between flow versus delay, free flow travel time is obtained by GPS data and actual travel time of vehicles is obtained by license plate method. On the entry and exit of link, license plate numbers are obtained using videography. From the entry and exit time of a particular vehicle, actual travel time is calculated. For the 5 min intervals, total vehicular flow was also collected through videography simultaneously. Typical travel time and data collection by license plate method of Relief road are shown in Table 3 for Kalupur station to Vijalighar stretch. Traffic volume count is converted in passenger car unit as per IRC-106:1990. PCU values for 2W-0.5, 3W-1.2, Car-1, Bus-2.2, LCV-1.4, NM-2 are taken.

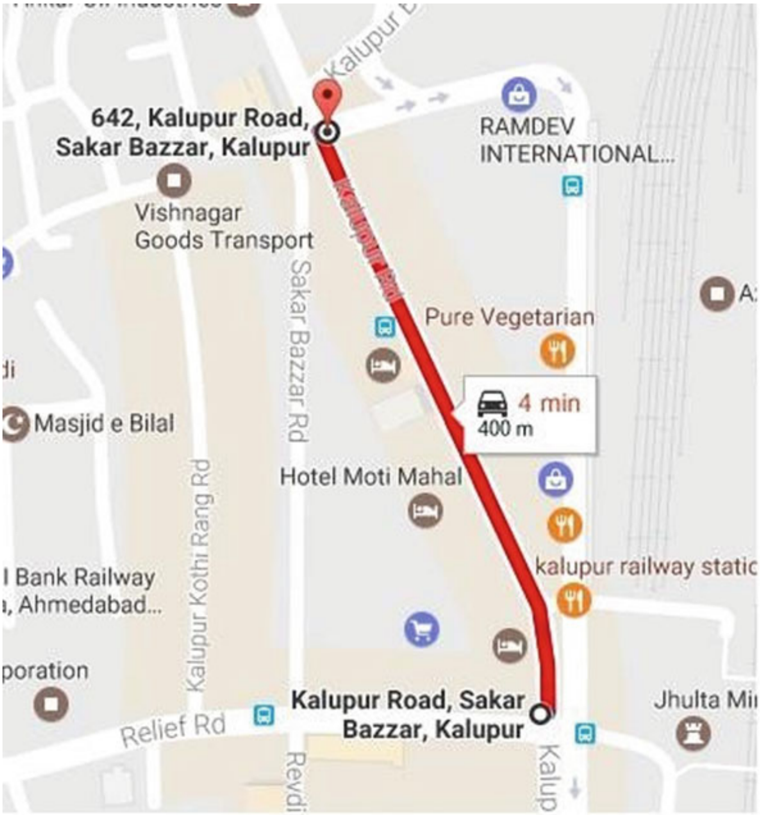

Fig. 3 Kalupur railway station to Kalupur circle road with a length of 0.4 km

Table 1 Geometry of selected stretches

Name of road	No. of lanes	Divided/undivided	Flow type	Length (km)	Width (m)
Relief road	Two lane	Undivided	Two way	1.95	9.5
Kalupur railway station to Sakar bazaar	Three lane	Undivided	One way	0.4	12
Kalupur railway station to Kalupur circle	Three lane	Undivided	One way	0.45	12
Sarangpur tank to Kalupur railway station	Four lane	Divided	Two way	0.6	18

Table 2 Total classified traffic volume for different stretches

Road name	2W	3W	Car	Bus/truck	LCV	Truck	NM
Kalupur railway station to Vijalighar (Relief road)	21,520	8804	553	02	19	–	1089
Vijalighar to Kalupur railway station (Relief road)	22,203	9200	623	43	16	05	1174
Kalupur railway station to Sakar bazaar	21,517	14,381	4378	1385	669	–	1566
Kalupur police station to Kalupur railway station	22,878	17,885	4046	1424	756	–	1638
Kalupur railway station to Sarangpur tank	16,985	12,818	2758	1002	458	–	1168
Sarangpur tank to Kalupur railway station	19,458	12,685	3758	1254	562	–	1245

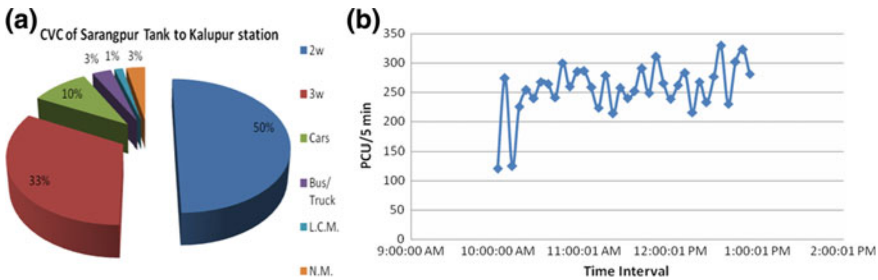

Fig. 4 **a** Total vehicle count, **b** hourly fluctuation of traffic flow data of 5 min interval from Sarangpur tank to Kalupur railway station

Figures 9 and 10 show the Flow versus Delay relationship for morning as well as evening period for the Sarangpur tank to Kalupur railway station stretch respectively. *Y*-axis shows average delay in a minute for the 5-min flow, whereas *X*-axis shows flow in PCU for the same 5 min. Linear trend fitting using MS Excel gives the model, which has good R^2 (Coefficient of determination) value. It seems that the average delay increases rapidly with the increase in flow. The same types of relationships have been obtained for the other selected stretches also.

Figure 11 shows the free flow speed contour map and Fig. 12 shows the peak period speed contour map. Contour lines show the travel time in min. These are plotted using GPS data. Table 4 shows the summary of data analysis of all the selected stretches.

Fig. 5 Free flow speed profile between Sarangpur tank–Kalupur railway station using GPS (morning free flow condition)

6 Conclusions

This study has been carried out on highly congested stretches of Kalupur region of Ahmedabad city. Field data was collected by videographic method for morning and evening peak hours. Travel time and delay data were collected using GPS and License Plate method. Based on data analysis conclusions are made as follows:

- At all selected stretches mixed traffic is found with composition of 2W—60 to 70%, 3W—25 to 35%, Cars—5 to 10%, Bus—1 to 5%, LCV—0 to 1%, NM—1 to 5%.
- Maximum numbers of vehicles observed on Relief road and Sakar bazaar road.
- Maximum delay is encountered on Sarangpur tank to Kalupur railway station road (0.6 km) is about 7 min (11.40 min/km travel time) in evening peak hour and on

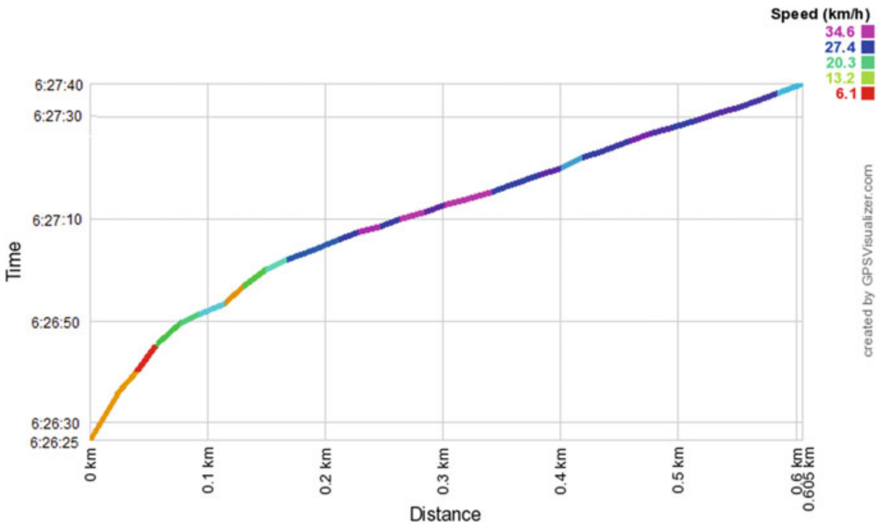

Fig. 6 Free flow speed with time-space diagram from Sarangpur tank–Kalupur railway station

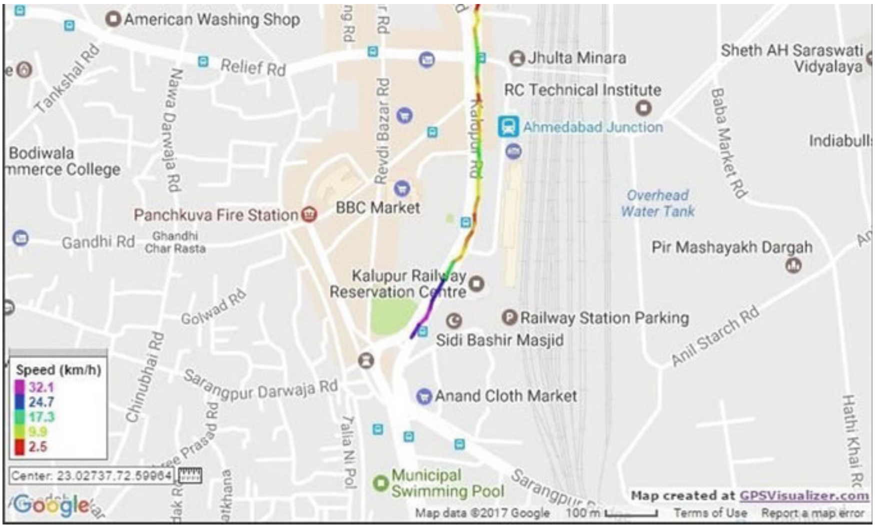

Fig. 7 Evening peak hour speed profile from Sarangpur tank to Kalupur railway station

Relief road (1.95 km) is about 7.14 min (3.40 min/km travel time) in morning peak hour.

- Travel time delay is also critical on Kalupur railway station to Sakar bazaar road (0.4 km) having average 6.55 min delay (16.22 min/km travel time) in morning and 7.10 min (17.45 min/km travel time) in evening peak hour.

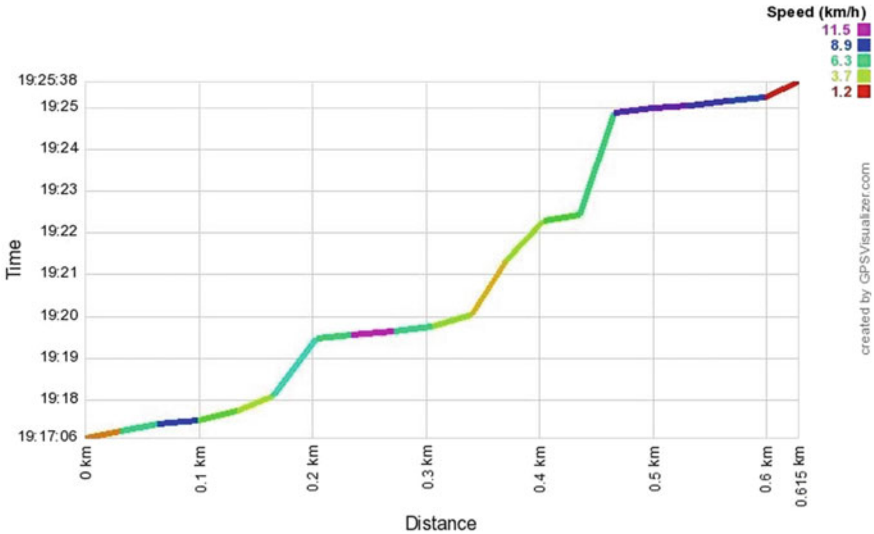

Fig. 8 Time-space diagram in evening peak hour from Sarangpur tank to Kalupur railway station

Table 3 Typical travel time and data collection by license plate method of Relief road

License plate method			Free flow speed = 43.78 kmph			
Kalupur station to Vijalighar			Length = 1.75 km			
Timing—10 am to 12 am			Travel time in FF (min) = 2.4 min			
Time (min)	Matching vehicle no.		Travel time (min)		Delay in min	
	License plate no.	Vehicle type	T.T. (min)	Avg. T.T. (min)	Delay (min)	Avg. delay (min)
10:00–10:05	4587	2W	10.07	10.295	7.67	7.895
	1254	3W	9.97		7.57	
	9652	2W	11		8.60	
	7541	2W	10.05		7.65	
	4454	2W	10.13		7.73	
	2013	3W	10.55		8.15	

For first vehicle 2W, Delay time = Total travel time − Travel time in free flow
Delay time = 10.07 − 2.4 = 7.67 min

- Lowest travel speed is observed on Kalupur railway station to Sakar bazaar road (0.4 km) in evening is about average 3.35 kmph.
- Types of delay observed during study:
  - Delay due to haphazardly movement of vehicles
  - Delay due to slow-moving vehicles like Nonmotorized vehicles

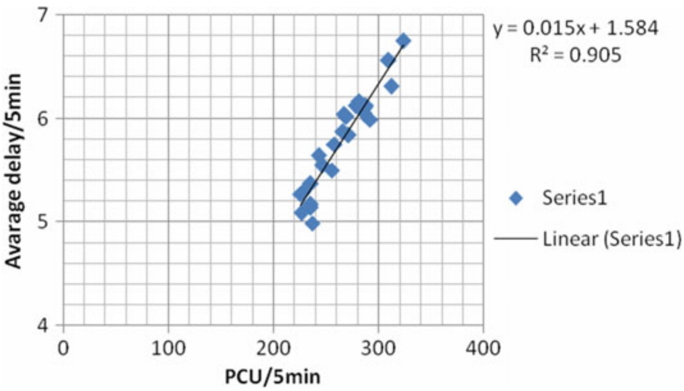

Fig. 9 Flow–delay relationship from Sarangpur tank to Kalupur railway station road (morning)

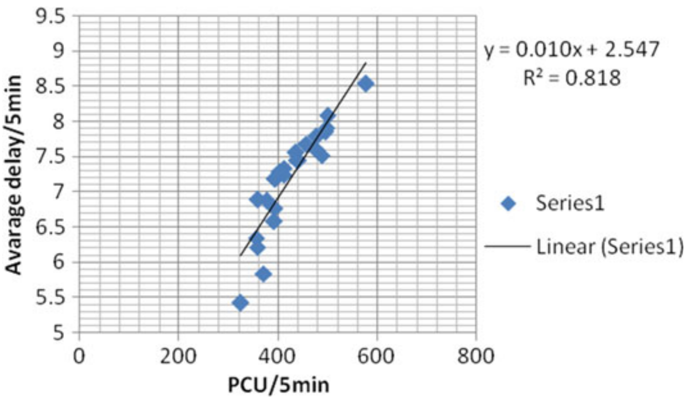

Fig. 10 Flow–delay relationship from Sarangpur tank to Kalupur railway station road (evening)

 - Delay due to no lane discipline
 - Delay due to on-street parking
 - Congestion delay
 - Approach delay
 - Stopped delay

- Major effect of vehicular flow on travel time is encountered on Relief road (one way of 1.9 km length), the coefficient of determination (R^2) is 0.933 and on Kalupur railway station to Sarangpur tank (morning) R^2 value is 0.905.
- Level of service which is a qualitative measure of traffic condition is very lower on selected stretches; it is "F" (As shown in Summary-Table 4). Condition of all stretches indicates that appropriate traffic regulations or changes need for improving current situation of traffic.

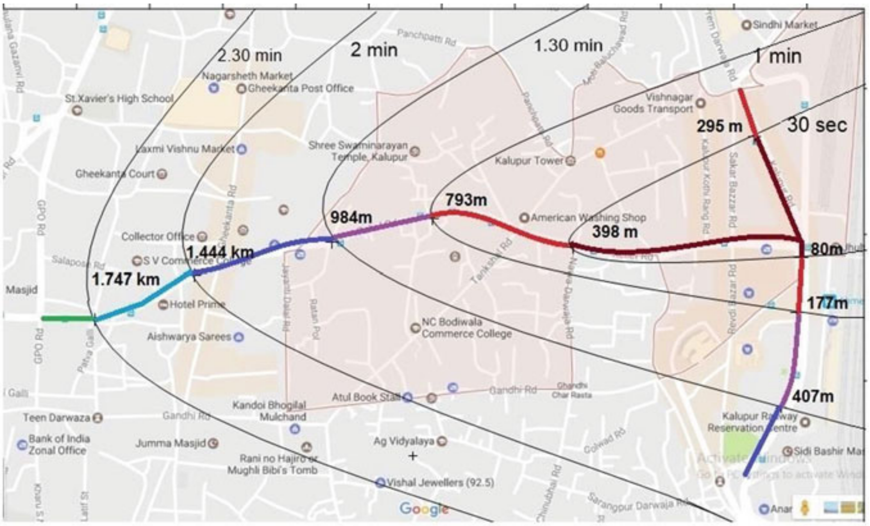

Fig. 11 Free flow speed contour map of Kalupur area

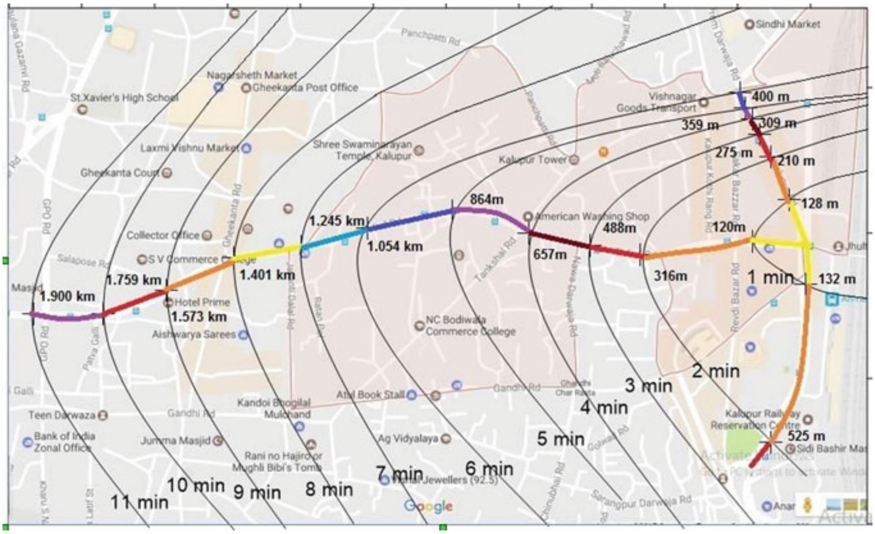

Fig. 12 Peak period speed contour map of Kalupur area

7 Suggestions

To improve the existing traffic conditions, some remedial measures are suggested:

- Regulate traffic by applying offensive traffic regulations.

Table 4 Summary of data analysis of all stretches

Stretch		No. of lane	Length (in km)	Width (in m)	Avg. FF speed (in km/h)	Avg. speed in peak period (in km/h)	FF travel time (mm:ss)	Avg. TT in peak hour (mm:ss)	Avg. delay (mm:ss)	R^2 of flow–delay model	L.O.S. as per HCM 2000
Relief road	Vijalighar to Kalupur stn.	Two lane Two way	1.95	9.5	42	11.7	2:46	10:00	7:14	0.933	F
	Kalupur stn. to Vijalighar		1.95		44	12		9:45	7:00	0.93	F
Kalupur railway station to Sakar bazaar	Morning	Three lane One way	0.4	12	40	3.5	0:40	6:55	6:15	0.871	F
	Evening		0.4		40	3.35		7:10	6:30	0.756	F
Kalupur railway station to Kalupur police station	Morning	Three lane One way	0.45	12	26	7.7	1:00	3:30	2:30	0.758	F
	Evening		0.45		26	7.7		3:30	2:30	0.816	F

(continued)

Table 4 (continued)

Stretch		No. of lane	Length (in km)	Width (in m)	Avg. FF speed (in km/h)	Avg. speed in peak period (in km/h)	FF travel time (mm:ss)	Avg. TT in peak hour (mm:ss)	Avg. delay (mm:ss)	R^2 of flow–delay model	L.O.S. as per HCM 2000
Kalupur railway station to Sarangpur tank	Morning	Two lane One way	0.6	18	35	5.8	1:10	6:10	5:00	0.905	F
	Evening					3.6		8:10	7:00	0.807	F
Sarangpur tank to Kalupur railway station	Morning	Two lane One way	0.6		40	5.14	1:00	7:00	6:00	0.905	F
	Evening					4.5		8:00	7:00	0.818	F

- Restrict the haphazard movement of traffic by various regulatory signs.
- Segregation of slow-moving traffic by providing level separated facilities.
- Providing appropriate parking facilities for auto-rickshaws.
- Removal of encroachments on the roadsides.
- Traffic System Management (TSM) techniques can be applied. For example, REID road which is parallel to Relief road is not fully utilized by vehicles. So, the vehicles may be diverted on this road by proper regulation.

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Development of Red Light Violation Detection System for Heterogeneous Traffic

Jinal Jariwala and Rajesh Gujar

Abstract In India, Ministry of Road Transport and Highways registers 480,652 road accidents in 2016 and it was 501,423 in 2015. Red Light Running is one of the major causes recorded at the signalized intersection for road accidents. Surat is one of the fastest-growing cities in India with a population of 6.04 million in 2016. The population of the city is increasing day by day as numbers of people are migrating. To reduce RLR and to improve traffic safety and security, Red Light Violation Detection (RLVD) system is introduced under the Intelligent Traffic Control System (ITCS).To detect the violating vehicle, Red Light Violation Detection Application (RLVA) software is completely designed and developed. This system is different from the traditional traffic signal system, in which a Smart camera with high capacity is placed to cover the area over which vehicles are restricted to passed. The main purpose of this research is to check the effectiveness of camera enforcement system at the signalized intersection for heterogeneous traffic.

Keywords Traffic-management · Red light violation detection system · Heterogeneous traffic · Automated enforcement · Red light cameras

1 Introduction

As urbanization is increased, it will increase traffic on the road. Many serious crashes happenes at signalized intersections. In India, Heterogeneous traffic can adequately be controlled by the police department. For that police department has used a traditional traffic signal system to control the traffic on the road. The traditional traffic signal system only instructs the driver of the vehicle whether to stop or go, but this

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system fails when the driver of the vehicle ignore the red signal and jump the intersection. So in that case policeman have to stop vehicle manually and take finds from the owner of the vehicle as he or she has violated the red signal, but practically it isn't possible for a policeman to stop every vehicle before the stop-line (Here Stop-line is the line drawn before the zebra-line and perpendicular to the traffic flow.) and make a list of violating vehicle and take finds. Therefore, it is required to develop a new system which is cost-effective and will help to minimize intersection violation.

The objective of this research is to compare the violation rate before the system is installed and after the system is installed for the same intersection and develop the automated system to detect the violating vehicle with type of violation, date, time, site name and location of the infraction, registration number of the vehicle through ANPR (Automatic Number Plate Reader) camera system for each vehicle identified for infraction. Image Analytics are given in the software.

Surat is one of the fastest-growing cities in India with a population of 6.04 million. People are migrating from different places to Surat for a better lifestyle and earning. Increase growth in population has created problems like road accidents, traffic jams, red light violation.

2 Literature Study

In this study, an automatic traffic recorder (ATR) used to detect a vehicle's speed and the number plate. The failure of a piezo sensor or inductive-loop sensor often results in missing data. In this study, the Weibull, normal, gamma, and lognormal distributions were fitted using maintenance histories censored for 60 months. According to the analysis, loop sensors have a relatively longer life than piezo sensors (Jung and Oh 2017).

This paper introduced RFID (Radio Frequency Identification) system in which RF antennas/RF tags are mounted at the roadside and receivers are placed inside vehicles. Each antenna transmits one set of the unique identification number which will be received by the driver of the vehicle so this way driver of the vehicle would previously be aware about red light signal. This system reduced the violation rate (Qiao et al. 2012). He introduced, a dot-matrix detector method, the system identifies the vehicle license plate location. Background interference is reduced by this system and computing time can also be saved (Dhole and Undre 2014).

3 System Architecture

To detect the violation, two things are mainly required to be monitored (1) Vehicle entry at intersection. (2) Current traffic signal position. Here as shown in Fig. 1, the 2-ANPR camera is installed to detect the number plate of the vehicle which is always focused towards the vehicle. Here one overview camera is placed between ANPR

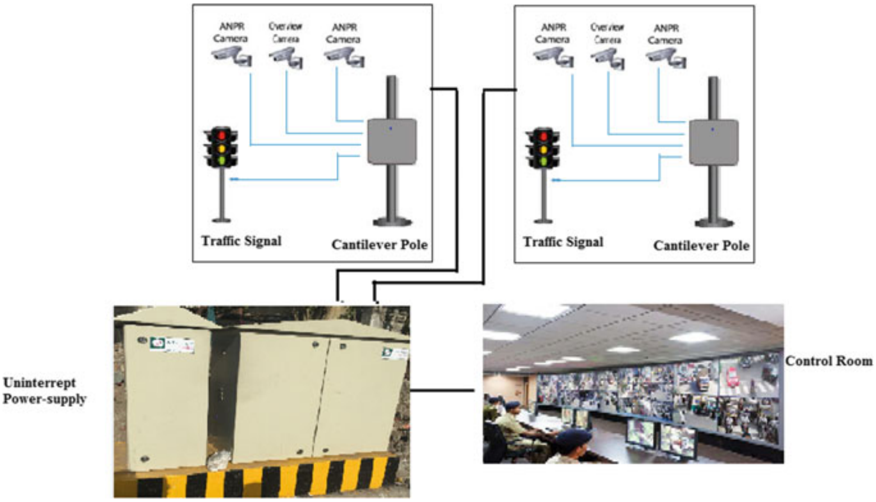

Fig. 1 System architecture

camera which is focused towards a red light. Overview camera will take 10 s. Video of violating vehicle.

One virtual loop is created at stop-line which is called as detection zone so when vehicle enters in this detection zone, camera will sense it and then overview camera will check the signal status if a signal is red and vehicle captured in detection zone then overview camera give an alarm to both ANPR camera to capture the number plate of the vehicle.

So this way vehicle data is collected and sent to the control room by OFC (optical fiber cable) where all the data are manually checked. This way challan is created.

3.1 RLVA Software Concept

System integration is done by three major components

1. Roadside Processing Components
2. Server-side processing Components
3. E-challan System Integration Platform.

The system collects passage data from different sensors to detect vehicle passage and judge the traffic violations within the road-network. This section describes the concept of execution among the system components and flow of execution.

The first step in enforcing red light violation detection is to monitor the traffic movements through the enforced intersection in real-time using video analytics running in roadside processing station. The method adopted here is to place ANPR and evidence camera sensors on the road and monitor vehicle movements through its focus area called detection zone. When the red light for an RLVD enforced lane is turned red, the system gets "red light on" trigger through "Red Light Status Trigger" module and also through video analytics running in Traffic Incident Detection client (TID Client) component, Traffic Analytics Engine (TA Engine) starts traffic monitoring through that lane. Every time the camera detects the vehicle, the TID Client component running in the roadside processing station (RPS) conducts analytics on the live video stream from overview camera and will monitor the vehicle trajectory to detect any stop-line/red light violations (Fig. 2).

4 Data Collection

Feasibility study covers three major junctions of the city. Feasibility study includes site layout that shows priority for RLVD pole installation, site characteristics, Violation data of every arm, evidence image of every arm of that intersection, signal condition whether it is working or not, stop-line and zebra-line visibility, Lane marking, distance between stop-line and zebra-line. The study area is selected where the traffic flow rate is high and maximum chances of the violation. The selected intersection had different geometry designs, signal time intervals, traffic flow rate for different time duration.

List of the junction where study data is collected as follows:

- Udhna Teen Rasta
- South Zone
- Udhna Darwaza.

For udhna teen rasta, all three arms are observed for the red signal phase to the next red signal phase which is given by the policeman not as per signal phase. To collect the data, first vehicles are classified. After that data of a total number of passing a vehicle and the total number of violating vehicles are collected manually for every arm of that intersection. Here total 10 sample readings are taken for each class of vehicle for both working days as well as for non-working days.

The data are collected in the morning peak hour (10:00–1:00), evening peak hour (6:00–8:00) and afternoon non-peak hour (3:00–5:00) for both working days as well as for non-working days.

ARM-3 at udhna teen rasta:

Here Table 1 shows a total number of passing vehicles and the total number of violating vehicle for both working days as well as for non-working days. For each kind of classified vehicle minimum three samples reading are taken, and according to these data, the graph is plotted to show comparison.

In Table 2, we can see clearly that in the afternoon non-peak hour, the vehicle movement is low as compared to morning peak hour and so violation rate is also less.

In Table 3, here violation rate is maximum as compared to morning and afternoon time for Arm-3 at udhna teen rasta.

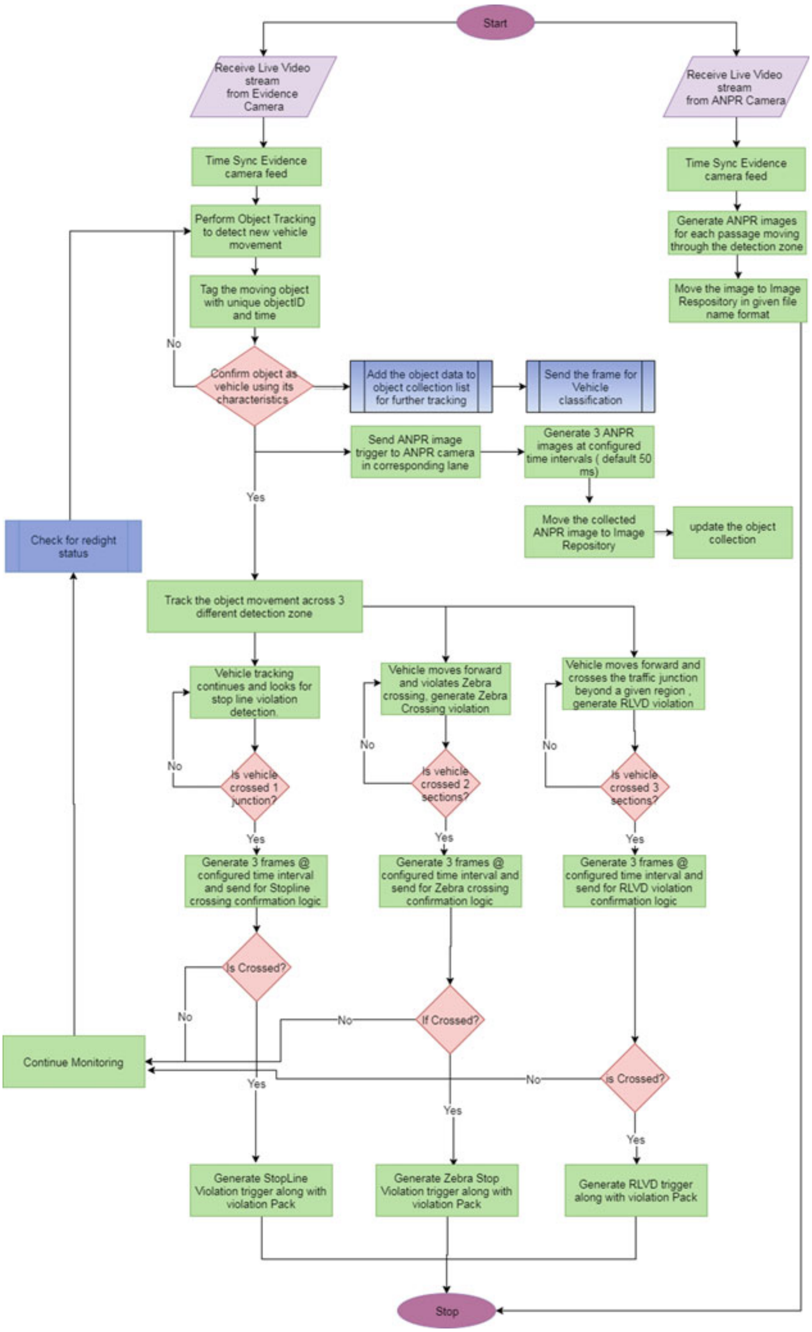

Fig. 2 Video analytics engine. *Source* ARS T&TT integrated traffic control system (ITCS) for Surat municipal corporation for SSDD traffic enforcement system

Table 1 Data collection at morning peak

Working day arm-3							Non-working day arm-3					
Morning peak hour												
Type of vehicle	Total number of passed vehicles			Total number of violating vehicles			Total number of passed vehicles			Total number of violating vehicles		
	1	2	3	1	2	3	1	2	3	1	2	3
2W	52	24	15	12	9	6	18	6	14	9	4	7
3W	28	19	19	6	10	7	10	9	12	2	–	5
Car	10	8	8	2	2	3	6	6	5	2	1	–
Bus	2	–	1	1	–	–	–	–	–	–	–	–
Truck	–	–	–	–	–	–	–	2	–	–	1	–

Table 2 Data collection at afternoon non-peak

Working day arm-3							Non-working day arm-3					
Afternoon non-peak hour												
Type of vehicle	Total number of passed vehicles			Total number of violating vehicles			Total number of passed vehicles			Total number of violating vehicles		
	1	2	3	1	2	3	1	2	3	1	2	3
2W	48	35	31	15	10	9	15	14	15	7	4	8
3W	12	16	12	6	4	5	5	7	8	2	3	3
Car	10	4	6	3	4	4	6	10	9	1	3	2
Bus	1	–	1	–	–	1	–	–	2	–	–	1
Truck	–	–	3	–	–	–	–	–	1	–	–	–

Table 3 Data collection at evening peak

Working day arm-3							Non-working day arm-3					
Evening peak hour												
Type of vehicle	Total number of passed vehicles			Total number of violating vehicles			Total number of passed vehicles			Total number of violating vehicles		
	1	2	3	1	2	3	1	2	3	1	2	3
2W	70	52	92	15	4	10	26	22	18	9	9	4
3W	34	13	33	8	5	4	4	6	5	–	2	3
Car	15	14	18	2	2	3	14	11	8	5	4	3
Bus	1	1	1	1	–	–	1	–	2	–	–	1
Truck	–	–	1	–	–	1	–	–	–	–	–	–

5 Data-Analysis

Here, the graph represents a comparison between different kinds of vehicles with respect to the violation, total number of passed vehicle versus total number of violating vehicles, working day versus non-working day vehicle movement for the udhna teen rasta at Arm-3.

Figure 3 shows two-wheeler does maximum violation in the morning peak time as compared to other vehicles. The violation rate for working days is 31.18% and it is 35.22% for non-working days.

Figure 4 shows that vehicle movement is decreased but the violation rate is highest among other time duration. Here violation rate is 40.93 and 36.95% for working days and non-working days, respectively.

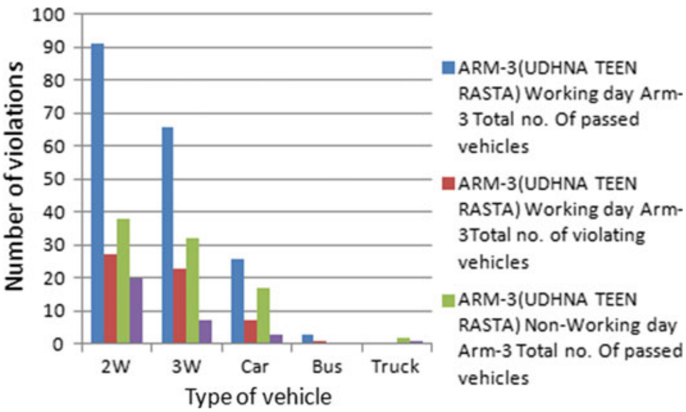

Fig. 3 Violation comparison at arm-3 morning peak

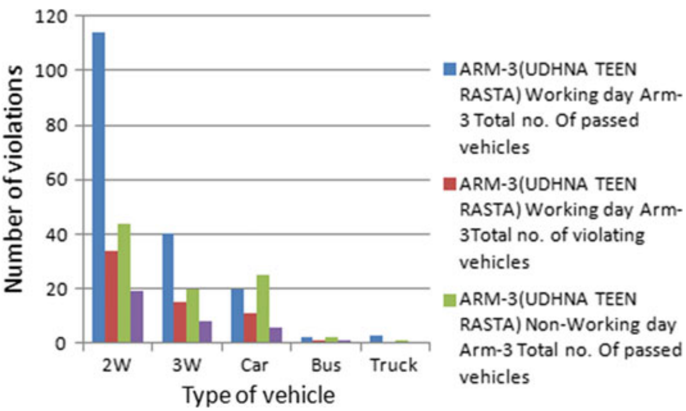

Fig. 4 Violation comparison at arm-3 afternoon non-peak

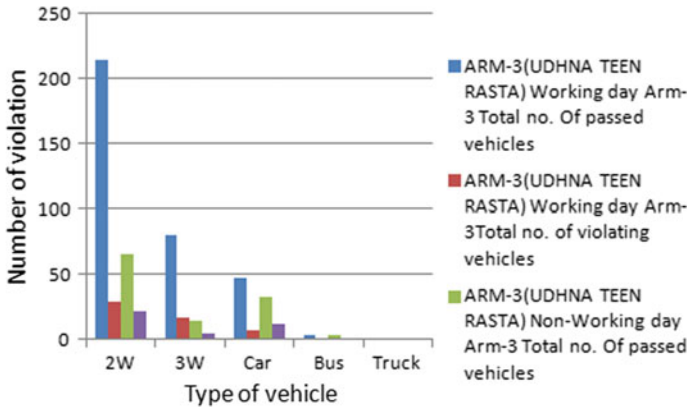

Fig. 5 Violation comparison at arm-3 evening peak

Figure 5 shows that in the evening peak time more trips and violations are done by two-wheelers and three-wheelers as compared to other vehicles for both working day and non-working day. Off course vehicle movement is increase as compared to afternoon non-peak hour and morning peak hour. During the evening period bus and truck movement for the non-working day is very low. Violation rate for working day is 15.94% and for non-working day violation rate is 34.18%.

6 Software Snapshots

The following are the snapshots of Red Light Violation Application which is totally designed and develop for detecting violating vehicle.

Figure 6 shows a list of violating vehicles with vehicle number plates and date when a violation occurs (Fig. 7).

Violation count versus Time graph shows an average hourly violation for the last 24 h, last week, last month (Fig. 7).

Here, when vehicle violates stop-line or red light, the system will capture four snapshots with one video with its surroundings, location, violation type, violation date, registration number. Here violation manually can be Re-checked as well as reject.

7 Conclusion

The study is about to develop an automated system that detects the violation during red light at intersections in Surat city. For the development of the system initial input data are required like traffic volume and violation rate at intersections, system

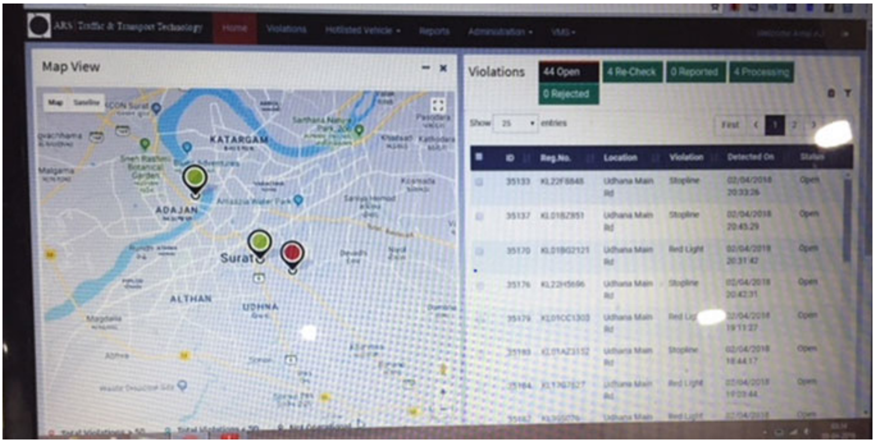

Fig. 6 List of violating vehicle

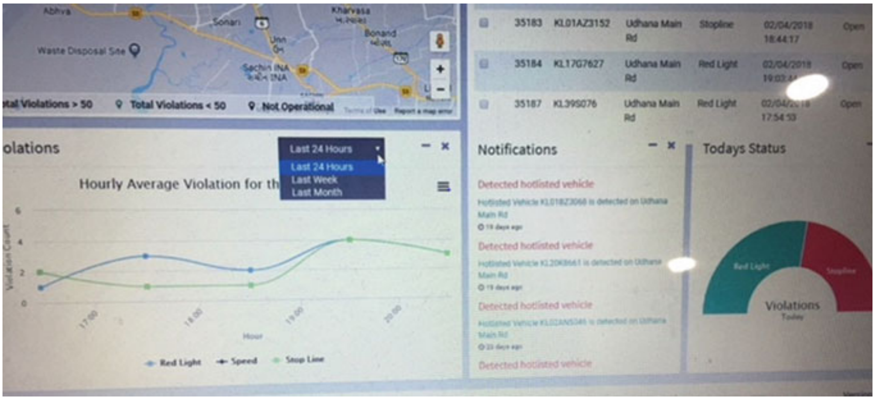

Fig. 7 Violation graph

implementation feasibility on site, etc., these data are collected by doing field survey and from that it is concluded that violation rate is much higher on the intersection and another is infrastructure necessity is not adequate on some of the intersections.

The proposed Red light violation detection system has the capability to reduce red light violation as it will automatically detect violation with details like vehicle class, vehicle registration number, date, time and place, and generate e-challan for the vehicle which has violated the stop-line. So the implementation of the system will efficiently reduce red light violations in the Surat city. Violation rate for udhna teen rasta at Arm-3 for the working day is 31.17%, 40.93%, 15.94%, and for non-working days it is 35.22%, 36.95%, 34.18% for morning peak hour, afternoon non-peak hour, evening peak hour respectively. Time-saving and no proper knowledge of traffic signals are the main reason behind the red light violations.

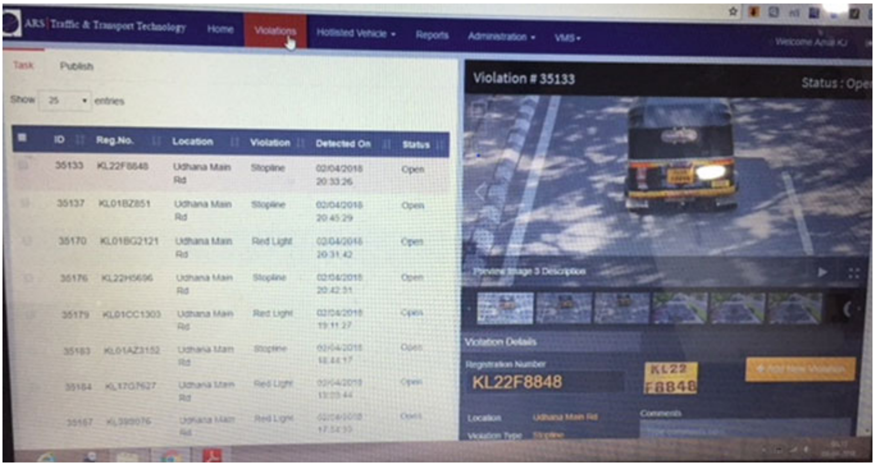

Fig. 8 Violating vehicle with registration number

Feasibility study, Data collection, and software development are part of this research. Here data before the system installation is collected and software is developed but due to some dispute (between ARS T&TT and SMC), after study data is not collected right now but this data will be collected within the next month.

8 Future Scope

This study is done only for three major junctions of Surat city which are situated on the important corridor of the city. This type of research can be done on other major intersections where a huge amount of traffic passes. And by implementing this system, we can reduce the violation count and make a safe and reliable journey.

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Comparative Analysis of Saturation Flow Using Various PCU Estimation Methods

Satyajit Mondal, Vijay Kumar Arya, and Ankit Gupta

Abstract The magnitude and nature of traffic flow in developing countries are difficult to evaluate due to its mix traffic conditions. In transportation network, intersection plays a vital role to increase the efficiency of the entire road network. Analysis of these nodal points (intersection) is required to evaluate the performance of the intersection through the assessment of operational parameters such as saturation flow and its level of service (LOS). However, the complexity of discharge flow in Indian scenario is mainly due to its mixed properties of traffic stream where both motorized and non-motorized vehicles are traveling in the same stretch without any lane discipline. Also, no single vehicle dominates the traffic stream consequently prediction of saturation flow is more sensitive to that mixed traffic. The passenger car unit is a common platform for the conversion of mixed traffic into a standard unit by taking passenger car as a conventional vehicle. The present study focuses toward the analysis saturation flow at signalized intersection using various PCU estimation methods under mixed traffic conditions. A detailed comparison of the saturation flow obtained by the each methodology with standard saturation flow value given in HCM (2010) is also presented. Traffic and vehicular data were collected from six signalized intersections from three Indian cities such as Delhi, Chandigarh and Allahabad using video graphic method. The prospective method resulted in a lower difference in saturation flow respect to HCM (2010) is proposed for non-lane-based mixed traffic stream.

Keywords Mixed traffic · Passenger car unit · Signalized intersection · Saturation flow

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1 Introduction

In last few decades, substantial development is observed in India both in industrialization as well as urbanization. The economy of India has been growing day by day, has a significant impact on its transportation system. In transportation network, intersection plays a vital role to increase the efficiency of the entire road network. The magnitude and nature of traffic flow in developing countries are difficult to evaluate due to its mix traffic conditions (Gupta et al. 2019). Indian traffic stream is highly heterogeneous and consists of variety of vehicles with widely varying static and dynamic characteristics, traveling in the same stretch without any lane separation (Mondal and Gupta 2019a). In such mixed traffic stream, no single vehicle dominates the traffic stream consequently prediction of saturation flow is more sensitive to that mixed traffic than in western countries where the traffic is mainly motorized and car-dominated. Also, lane discipline is not followed during the entry and exit from the intersection instead the vehicles have a tendency to use lateral gaps to reach at the head of the queue through filling the small gaps between the larger vehicles. This practice mainly makes the intersection more congested with uneven distribution of traffic over it (Mondal and Gupta 2018). The most common platform for analyzing traffic flow in developing countries is to convert the mixed traffic into homogeneous one using passenger car unit (PCU) (Mondal et al. 2017). The HCM (2010) defined passenger car equivalent (PCE) as the number of passenger cars which will result in the same operational condition as a single heavy vehicle of a particular type under specified roadway, traffic and control conditions. IRC SP 41:1994 (1994) is the only guideline for the design of at-grade intersections in rural and urban areas for Indian traffic stream. In a number studies, PCU proposed as a static parameter though it changes with several factors like traffic and vehicular characteristics, road geometry even with the timing of the control system makes it dynamic. Due to fundamental differences, the standard western relationships for predicting the value of saturation flows and PCUs are not appropriate for developing countries having mix traffic conditions. The present study focuses toward the appropriate estimation of passenger car unit and a step toward the assessment of saturation flow for non-lane-based mixed traffic stream.

A significant effort has been made in last few years to evaluate the operational parameters of a signalized intersection, especially for mixed traffic conditions. Characteristics of discharge vehicle mainly the saturation flow is an utmost parameter for the evaluation as well as measurement of performance parameters. HCM (2000) presents a methodology for analyzing signalized intersection considering details of each parameter for a lane-based car-dominated traffic stream, with limited applicability for the mixed traffic conditions. Researchers have estimated the saturation flow value for mixed traffic conditions through converting the mixed traffic into a homogeneous one using PCUs of vehicles. A number of methodologies have proposed by the various researchers in previous studies to evaluate PCUs of vehicles at signal controlled intersection using various traffic stream parameters. Headway ratio method (Greenshields et al. 1947; Saha et al. 2009; Biswas and Ghosh 2017) is one of the useful techniques to calculate the PCUs of vehicles. This method considers the ratio of headway of a particular vehicle type to the headway of car. Some other methods that are effectively utilized by the various researches are Delay Method (Rahman et al. 2003; Benekohal and Zhao 2000), Regression analysis (Branston and Zuylen 1978), Saturation flow ratio method (Demarchi and Setti 1852), Optimization technique (Radhakrishnan and Mathew 2011), and queue clearance rate method (Mohan and Chandra 2017). Among them, regression technique is one of the most useful tools to calculate the PCUs of vehicles at signalized intersections. Though most of the methods are developed based on homogeneous lane-based traffic stream, India has a non-lane-based mixed traffic stream with variety of vehicles traveling in a same stretch. Therefore, the prediction of an appropriate method is difficult to identify, suitable for mixed traffic stream due to different methodological background in each approach. Thus, PCUs of vehicles are estimated using four different methods and further used to evaluate the saturation flow value. A comparative analysis is also done between the obtained saturation flow using each methodology with standard saturation flow value given in HCM (2000) to identify the most rational and appropriate methodology for mixed traffic stream.

2 Data Collection and Extraction

The methodology for data collection is being adopted as per the guidelines of Highway Capacity Manual (2000). Presently, six signalized intersections were selected for the data collection from three different cities of India. All the intersections are having a channelized section for left turn movement; there is no nearby bus stop and no roadside parking. Percentage of non-motorized vehicles is also negligible. All the selected intersections are free from pedestrian activities with a pre timed signal characteristics. The details of the intersections are listed in Table 1. The data were collected at peak hours using the videographic technique. The camera was mounted at vantage point nearside the intersection to cover the upstream traffic from approach stop line. Once the video is recorded, it is played at the workstation to extract several traffic stream parameters. The entire traffic was classified into six

Table 1 Details of signalized intersection

No of intersection	City	Approach width (m)	Cycle Length (s)	Green time (s)	No of cycles observed
I-1	Allahabad	7.13	122	42	77
I-2		8.95	145	36	75
I-3		8.35	112	30	55
I-4	Delhi	10.2	170	56	51
I-5		10.2	170	60	40
I-6	Chandigarh	7.70	120	30	44

different categories such as two-wheelers (2W), motorized three-wheelers (M3W), Car (C), Big Car (BC), BUS and LCV. These classifications were based on the static and dynamic characteristics of each vehicle Classified vehicles count was done to obtain the traffic volume and compositional share of individual vehicle class at each study section. The compositional share of single vehicle class is shown in Fig. 1. The proportion of each vehicle type at the selected signalized intersections are varied from 25 to 66% for 2W, 6 to 13% for M3W, 18 to 52% for Car, 5 to 9% for BC, 1 to 3% for LCV and 1 to 6% for Bus respectively.

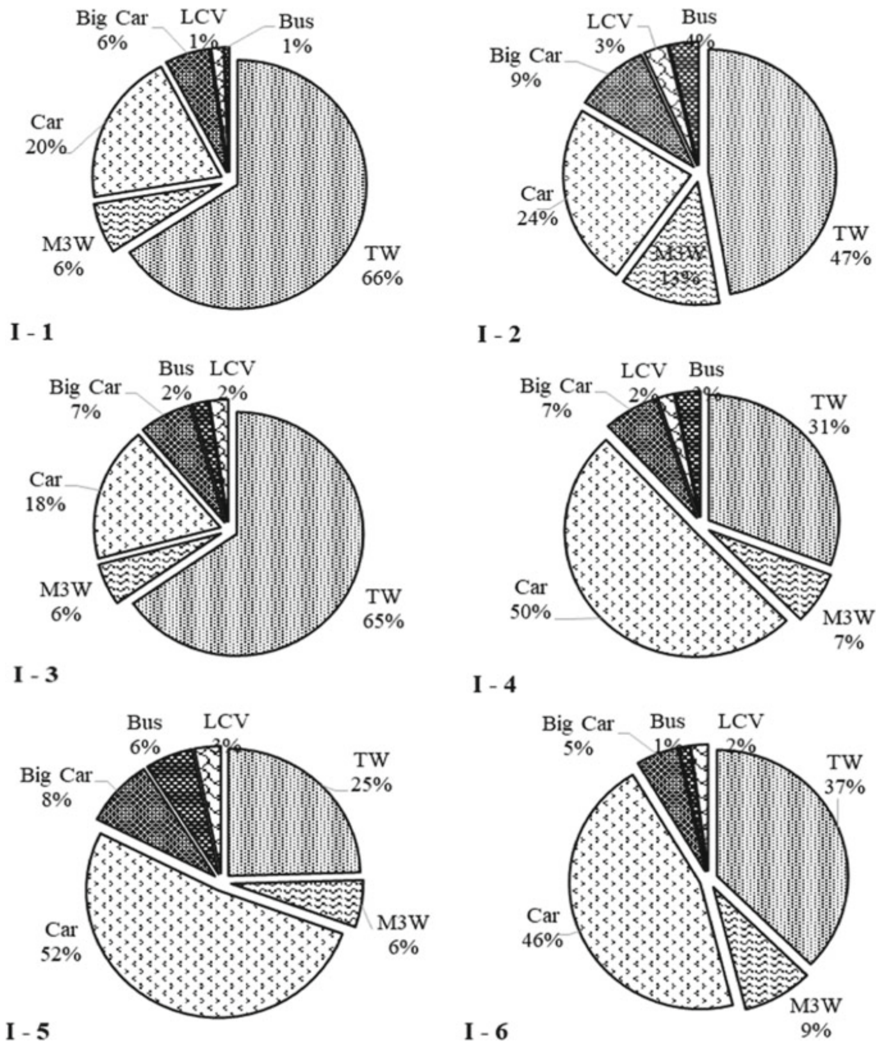

Fig. 1 Vehicular composition at all the selected intersections

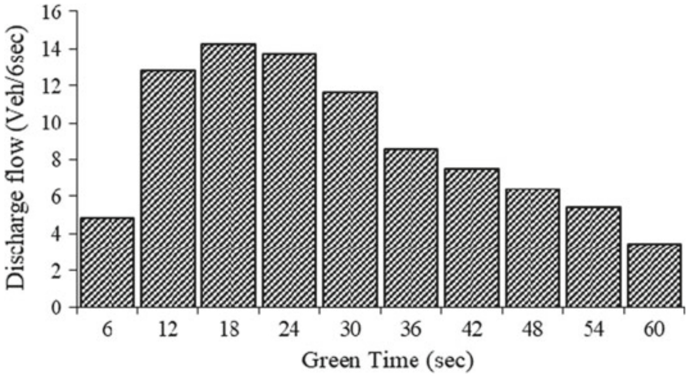

Fig. 2 Discharge of vehicles through the intersection during green time

The departure of vehicles was extracted at the 6-s interval (equal slot of green time) from the collected data as shown in Fig. 2. At the initiation of green, vehicles start to cross the section at an increasing rate. Vehicles soon reach a stable state where they are following one another with a constant gap or headway. At the end of the green, flow decreases with an increasing rate and become zero when the signal shows red. The fluctuation in flow is due to the heterogeneity of the traffic stream.

3 Research Methodology

Previous studies show that different methods can be used for the estimation of PCU at a signalized intersection (Mondal and Gupta 2019b; Mondal et al. 2019). Among them, regression was effectively used by the researches to find out PCU values of vehicles. The present study adopts four different methods namely Queue clearance rate method, Regression method, Optimization by using Theil's coefficient and Optimization through normalizing the flow to estimate the PCUs of vehicles and corresponding to evaluate the saturation flow value at the signalized intersection for mixed traffic conditions. The background of the current methodology can be depicted from the flowchart given in Fig. 3.

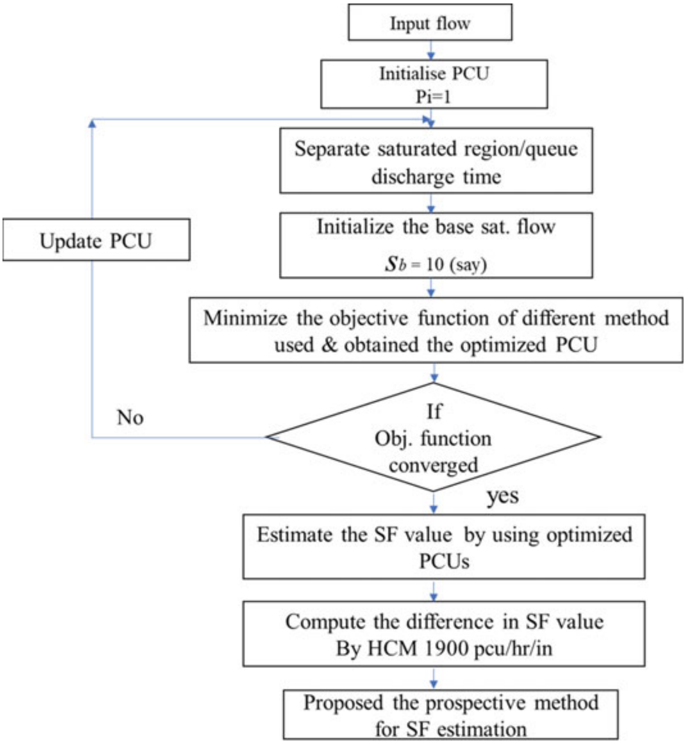

Fig. 3 Flow chart of the present methodology

4 Analysis of PCUs Using Various Estimation Methods

4.1 Methodologies of PCU Estimation

4.1.1 Optimization Technique (Theil's Coefficient)

An optimization technique is a process of minimizing the difference between an ideal saturation flow curve and the observed saturation flow curve. HCM (2000) has suggested that the base saturation flow should be estimated with an assumption that vehicle moves in a queue, whereas in mixed traffic, vehicles occupy the position as per the gaps available in a queue and haphazardly discharge through the intersection. Therefore, a significant difference is observed between the ideal flow and observed flow. In this optimization technique, this difference can be minimized using Theil's coefficient (Z_{min}) as the objective function. This optimization problem is formulated as:

$$z_{\min} = \frac{\sqrt{1/N \sum_{i=1}^{N} (S_b - S_i)^2}}{\sqrt{1/N \sum_{i=1}^{N} (S_b)^2} + \sqrt{1/N \sum_{i=1}^{N} (S_i)^2}} \tag{1}$$

$$S_i = \sum_{j=1}^{m} n_j P_j, \quad P_j \geq P_{j\min} \tag{2}$$

where S_b is the base saturation flow, S_i is the saturation flow during interval i, n_j, p_j are the number and PCE of vehicle type j following steps are used for the computation of optimized PCU values.

Step 1: Initialize the all PCU values equal to 1
Step 2: Separate the saturated portion and estimate the saturation flow value at initial PCUs
Step 3: Using solver in excel solve the formulation (Eqs. 1 and 2) to get the optimized PCUs of vehicles.

4.1.2 Regression Technique

Regression technique (RT) is one of the useful methodologies for the analysis of different traffic stream parameters. Number of vehicles are calculated that are crossing the intersection during the saturated green time. The background of the methodology is given in the following equation.

$$\tau = \varepsilon + \sum_{i \neq 1} a_i n_i \tag{3}$$

where τ represents saturated green time (excluding start-up lost time and end loss time) and a_i is the weight associated with n_i, n_i is the number of classified vehicles and ε stands for the fixed error term.

The ratio of the coefficient a_i for particular vehicle type to the coefficient of car a_c is known as the PCU factor for the corresponding vehicle type as:

$$\text{PCU}_i = \frac{a_i}{a_c} \tag{4}$$

where a_c = Coefficient for car.

Following are the steps used for the computation of PCU values by regression method.

Step 1: counting of the vehicle during queue discharge time (T_{sec}).
Step 2: compute the coefficient a_i by using queue discharge time (T) and classified vehicle count in the form of Eq. (3).
Step 3: solve Eq. (4) to obtain the PCU for different category of vehicle.

4.1.3 Queue Clearance Rate Method

Queue clearance rate (QCR) method was developed to estimate the PCU at un-signalized intersections for heterogeneous traffic conditions (Benekohal and Zhao 2000). This method shows a reasonable estimation of PCU for non-lane-based traffic behavior. This method estimates the ratio between a number of vehicles in the queue (N) and the time taken by the queue to clear the conflict area given in Eq. 5.

$$\mathrm{QCR} = \frac{N}{T} \tag{5}$$

where N is the number of vehicles in the queue (in PCE), T is the time taken by the queue to clear the intersection area (in second). Here Denominator 'T' shows the time elapsed between the arrival of the front end of the first vehicle of the queue and exit of the rear bumper of the last vehicle in the queue from the conflict area. The numerator in Eq. 5 can be formulated as:

$$N = \sum_{j=1}^{k} n_j \frac{W_{\mathrm{car}}}{W_j} \mathrm{PCE}_j \tag{6}$$

where W_{car} is the width of the standard car (m), W_j is the width of vehicle type j (m), PCE_j is passenger car equivalent for vehicle type j, k is the number of vehicle categories in the traffic stream.

This method assumes that the variation in QCR will be the minimum if the queues were comprised only of passenger cars. Hence, a linear programming problem could be developed using Eqs. 5 and 6 with the objective of minimizing the coefficient of variation in QCR for different queues by considering PCE of different vehicle types as the design variable. Following are the steps used for the computation of PCU values by QCR method.

Step 1: Initialize the all PCU values to 1.
Step 2: Compute the time (T) taken by the vehicle to clear the intersection.
Step 3: Compute the value of (N) by using Eq. 6.
Step 4: Compute QCR using Eq. 5.
Step 5: Minimize the error obtained by the ratio of std dev. of QCR to the mean of QCR and get the optimized PCUs.

4.1.4 Optimization by Normalizing the Flow

Optimization technique (OPT) attempts to minimize the difference between an ideal saturation flow curve and the observed flow curve. Saturation green region is estimated using statistical test analysis of variance (ANOVA) to separate the saturated zone from the entire green time in a cycle. The flows within the saturated intervals are normalized as per Standard Score Method.

Table 2 Comparison of PCUs obtained by four different methods

Vehicle type	IRC PCUs	PCU (in literature)		QCR		RT		TC		OPT	
		Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
2W	0.5	0.18[a]	1.22[a]	0.2	0.253	0.04	0.511	0.2	0.7	0.15	0.36
M3W	1.0	0.79[a]	3.48[a]	0.7	1.20	0.52	2.33	0.7	1.4	0.7	1.2
Car	1.0	1.00	1.00	1.0	1.0	1.00	1.00	1.0	1.0	1.0	1.0
BC	1.0	1.25[b]	1.99[b]	1.2	1.6	0.156	1.33	1.4	1.8	1.2	1.6
BUS	3.0	1.23[a]	5.16[a]	2.9	4.0	0.03	2.485	2.9	4.0	1.4	2.9
LCV	1.50	1.09[a]	3.63[a]	1.4	1.62	0.53	2.86	1.4	1.9	1.4	2.9

[a]Praveen and Arasan (2013)
[b]Chandra (2004)

$$\text{Normalized Flow} = \frac{x - \text{mean}}{\text{std dev}} \tag{7}$$

where x is the flow within saturated green intervals.

Following are the steps used for the computation of PCU values by optimization approach.

Step 1: Initialize the all PCU values to 1.
Step 2: Perform the ANOVA test for finding statistical equivalency among the flow in successive intervals determine the saturated green region.
Step 3: Normalized the flow within the saturated intervals using Eq. 7.
Step 4: minimize the error (using SOLVER) obtained by the summation of the std dev of the saturated interval and get the optimized PCUs.

4.2 PCU Estimation

The above four methods are utilized to estimate the PCU values of individual vehicle type shown in Table 2. Values listed in IRC and previous literature are also shown in Table 2, where a significant difference is observed between the values given in IRC and estimated PCU values. Figure 4 shows the average PCU value of individual vehicle type obtained by each method.

5 Saturation Flow Estimation

Saturation flow is one of the critical parameters to evaluate the performance of a signalized intersection. In the present study, the conventional method is used (Eq. 9) for saturation flow measurement for each study location. In the beginning, classified

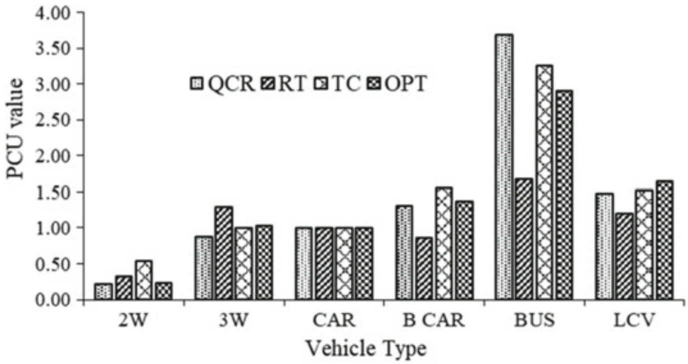

Fig. 4 Mean PCU values of each vehicle type

vehicles count was carried out to calculate the number of vehicles flowing through the intersection per green phase. The equivalance unit for each vehicle type was multiplied with corresponding its number of vehicle to estimate saturation flow value in terms of PCU/h using Eq. 8.

$$\text{Saturation flow (SF)} = \left[\sum n_i \text{PCU}_i\right] \frac{3600}{g_\text{e}} \tag{8}$$

where n_i is the number of the vehicle category i, PCU_i is the PCU of vehicle category i, g_e represents the effective green time (s).

Effective green time has a significant impact on saturation flow. It basically involves the green time and corresponding the lost times, i.e., start-up lost time (SULT) and clearance lost time (CLT). Initially at the beginning of green time, an amount of time is lost due the driver's reaction time to start and accelerate the vehicle known as start up lost time. It is the total time consumed by all the driver's standing in a queue to react and accelerate. A basic methodology is proposed by the HCM 2010 simply adding the difference between the saturation headway and vehicle's headway up to a certain queue position (until it reaches saturation headway). But due to non-lane base mixed traffic stream, determination of vehicles headway is become a quite challenging one. Also, at the start of green, vehicles standing in front of queue (Generally Two wheeler observed in all the study locations) are moving haphazardly as per their static and dynamic characteristics. Therefore, the SULT is calculated using the average discharge flow through the section and the discharge flow at first green slice shown in Eq. 9.

$$\text{SULT} = t - \frac{q_o * t}{s} \tag{9}$$

where t is duration of the time slice, q_o is discharge flow at the first green slice and s is the average discharge flow.

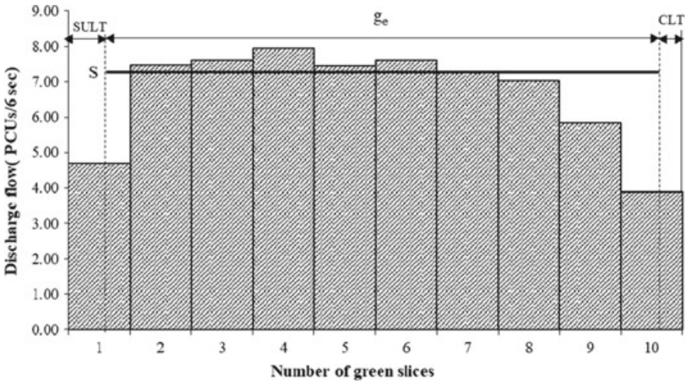

Fig. 5 Discharge of vehicles during the green time

The clearance lost time (CLT) is often not perceived as the chances of some vehicles are waiting at the red time be still waiting after the green end is less. Therefore, it is also calculated as per the above equation using the flow value of the end green slice. Figure 5 shows the basic elements of lost time estimation.

HCM 2010 has provided a basic formulation regarding the effective green (g_e) using various parameters shown in Eq. 10.

$$g_e = G + A + \mathrm{AR} - (\mathrm{SULT} + \mathrm{CLT}) \tag{10}$$

where g_e is the effective green time (s), G is actual green time (s), A is the provided amber time (s), AR is all red (s) time.

Table 3 shows the values of lost time and corresponding its effective green time for each selected signalized intersection obtained using the Eq. 10.

The mean PCUs presented in Fig. 4 have been used to find out the flow values (in PCU) at each green slices for each cycle. The effective green time given in Table 3 has been used to estimate the saturation flow value of the each study location using Eq. 8. Table 4 depicts the mean saturation flow value obtained by the various PCU

Table 3 Values of effective green time of the selected intersection

No of intersection	Actual green time (s)	Amber time (s)	SULT (s)	CLT (s)	Effective green time (s)
I-1	38	4	2.87	2.42	37.11
I-2	32	4	2.16	3.06	30.78
I-3	26	4	1.86	2.77	25.37
I-4	52	4	1.99	3.23	50.78
I-5	56	4	1.77	2.33	55.90
I-6	26	4	2.33	2.72	24.95

Table 4 Comparative analysis of SF values

	Saturation flow values (PCU/h/ln)			
Methods	QCR	RT	TC	OPT
Estimated saturation flow	1814	1784	2155	1830
Difference between HCM and estimated SF (%)	4.52	6.10	13.42	3.68

estimation methods. A comparative analysis has also been presented between the observed saturation flow value and saturation flow value prescribed in HCM 2010.

Table 4 shows that three different methodologies namely Queue clearance (QCR), Regression technique (RT) and Optimization technique (OPT) are quiet useful to estimate the PCU values as the variation of equivalent flow from the value mentioned in HCM is below 10% for each methods. Although, OPT gives best estimation of PCUs in terms of percentage difference (3.68%) in saturation flow value. Student *t*-test has been performed to check the statistical significance in the mean value of saturation flow between OPT and other methods. It can be found that the difference in saturation flow between OPT and other methods is significant in terms of *t*-statistics and *p*-values. Tables 5, 6 and 7 gives the result of the paired *t*-test between OPT and other methods. The accuracy of the optimization technique is due the consideration of saturated green time in its estimation procedure.

Table 5 Result of paired *t*-test between OPT and QCR

Group	Mean	Variance	SD	df	*P* value	t/t_{critical}	Significance
OPT	1830	209,254.84	457.44	239	0.0011	3.319/1.974	Yes
QCR	1814	230,514.04	480.12				

Table 6 Result of paired *t*-test between OPT and RT

Group	Mean	Variance	SD	df	*P* value	t/t_{critical}	Significance
OPT	1830	209,254.84	457.44	239	2.03E−05	4.393/1.974	Yes
RT	1784	145,204.28	381.0568				

Table 7 Result of paired *t*-test between OPT and TC

Group	Mean	Variance	SD	df	*P* value	t/t_{critical}	Significance
OPT	1830	209,254.84	457.44	239	1.26E−63	−28.13/1.974	Yes
TC	2155	177,801.64	421.67				

6 Conclusion

The present study analyzed the saturation flow of signalized intersection for heterogeneous traffic condition using various estimation methods. Data were collected from three different cities of India using the videographic technique. Several traffic stream parameters were extracted from the collected data to analyze the PCUs and saturation flow value. Four different methods namely queue clearance rate method (QCR), regression technique (RT), optimization using Theil's coefficient (TC) and optimization by normalized flow (OPT) are utilized to estimate dynamic PCUs of different vehicle type. Saturation flow has been calculated using basic formulation provided in HCM 2010 using effective green time at each location. SULT and CLT have been incorporate in effective green time by considering the average discharge flow through the section and the discharge flow at first green slice. A range of SULT of 1.77–2.87 s and CLT of 2.33–3.23 s is observed at each selected location. The mean PCUs obtained by each method are used to estimate the flow value. It has been observed that three different methodologies namely Queue clearance (QCR), Regression technique (RT) and Optimization technique (OPT) are quiet useful to estimate the PCU values as the variation of equivalent flow from the value mentioned in HCM is below 10% for each methods. Although, OPT gives best estimation of PCUs in terms of percentage difference (3.68%) in saturation flow value. Student *t*-test has been performed to check the statistical significance in the mean value of saturation flow between OPT and other methods. It can be found that the difference in saturation flow between OPT and other methods is significant in terms of *t*-statistics and *p*-values. The accuracy of the optimization technique is due the consideration of saturated green time between the entire green region and estimates the saturation flow value through normalizing the flow as per standard score method. Result gives a flow value of 1830 PCU/h/lane which is near to the proposed saturation flow by the Highway Capacity Manual 2010. Therefore, the optimization technique through normalizing the flow as per score method can be used to estimate the dynamic PCUs and Saturation flow value for the non-lane-based heterogeneous traffic.

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Empirical Travel Time Reliability Assessment of Indian Urban Roads

Rushikesh Amrutsamanvar, Gaurang Joshi, Shriniwas S. Arkatkar, and Ravi Sekhar Chalumuri

Abstract This chapter focuses on the performance evaluation of Indian urban roads from the perspective of travel time reliability. Travel time data on two important road corridors of Surat city were collected using a license plate matching technique for this purpose, and the performance of these corridors was assessed using and various travel time reliability metrics. Statistical models were developed to identify the functional relationship between space mean speed (SMS) and planning time index (PTI). As Indian traffic comprises of multiple vehicle classes, an attempt was made to analyze the effect of vehicle composition on the average travel time using artificial neural network (ANN)-based approach. It was observed that travelers must consider the higher cushion time while planning a trip on both of these corridors. The developed regression models demonstrated the strong functional relationship between SMS and PTI on both of the sections. Developed ANN models revealed that the percentage of car and auto-rickshaws present in the traffic stream significantly affects the average travel time along with the total volume of traffic.

Keywords Travel time reliability · Neural network · Road performance

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1 Introduction

With widespread urban sprawls and increased congestion, urban dwellers spend a substantial portion of their time in a daily commute. Despite the accessibility to different transportation modes, the reliability that a person can reach the desired destination in expected time is still unpredictable. As a consequence, commuters often strategically select the departure time, mode, and route of their trip based on past travel experiences. The information of reliable time window in which the trip can be completed is critical to the road user, and this ultimately results in associating the level of service of any road link with the experienced travel time variability. Today, more than 32% of the Indian population lives in cities, with more than 40% of them traveling for at least half an hour a day to reach their workplaces (MoUD 2016). On the other front, increased pull supply chain-based logistics practices have significantly amplified the intra-city freight transport, demanding faster mobility more than ever. This fast-paced urban scenario with ever-increasing traffic demand and constraints on roadway capacity expansion clearly divulges the need of studying the travel time variation characteristics of Indian urban roads. These studies can facilitate individual trip decision making, and helping in better traffic management of the overall system. With this motivation, the present paper tries to answer the three distinct research questions:

1. How is the performance of a typical Indian urban road on weekday peak hours in terms of travel time reliability?
2. What is the inter-class travel time variability on the typical urban road on weekday peak hours?
3. Can we use neural networks to analyze the effect of vehicle composition on the travel time of these corridors?

To answer these questions, the travel time data on two typical urban roads in peak hours were collected using the license plate recognition method. The performance of these corridors was evaluated using travel time reliability metrics. The used travel time reliability metrics, the results from the analysis, and the developed models are discussed in the subsequent sections.

2 Travel Time Reliability Measures

Travel time reliability (TTR) is a widely recognized decisive factor for travelers to secure their on-time travels. It can be defined as the probability that a particular trip can be completed within a specified duration of time (Rakha et al. 2010; Cambridge Systematics 2008). Some of the commonly used travel time reliability measures include 95th percentile travel time, travel time index (TTI), buffer time index (BTI), and the planning time index (PTI). From these measures, buffer time index signifies the cushion time (buffer) that traveler must consider for on-time arrival compared to the average travel time. It is estimated as the ratio of the difference between the 95th percentile and average travel time divided to the average travel time. Travel time index is obtained as the ratio of the average travel time to the free-flow travel time. This metric demonstrates the average extra time required for a commuter to complete a trip compared to the free-flow time, which is commonly considered as the desired travel time of commuter to complete a trip. Next to that is PTI which is estimated as the ratio of the 95th percentile travel time, to the free-flow travel time. In addition to these metrics, Van Lint and Van Zuylen (2005) proposed the use of skewness to measure the level of reliability. Two new reliability metrics namely the skew and width of the travel time distribution were proposed, which can be estimated as Eqs. (1) and (2), respectively.

$$\lambda_{\text{skew}} = \left(\frac{T_{90} - T_{50}}{T_{50} - T_{10}} \right) \tag{1}$$

$$\lambda_{\text{variance}} = \left(\frac{T_{90} - T_{10}}{T_{50}} \right) \tag{2}$$

where T_{90}, T_{50}, and T_{10} are the 90th, 50th, and 10th percentile travel time, respectively. This study concluded that larger the value of λ_{skew}, more unreliable are travel times on the corridor. In the present study, all the above-mentioned travel time reliability metrics are estimated for both the test road corridors, in order to evaluate the performance of these corridors.

2.1 Previous Work

Several studies (Lomax et al. 2003; Higatani et al. 2009; Bullock et al. 2011; Rakha et al. 2010) have been reported in the literature which observed and modeled travel time reliability. However, very few attempts have been reported from India. A few initial (Gangopadhyay et al. 2010; Amrutsamanvar 2013; Bharati et al. 2013) studies attempted to study the travel time variability on Indian urban roads. Later on, Chepuri et al. (2017) collected travel time data of powered two-wheelers with traditional test vehicle method and observed the functional relationship between BTI and volume to capacity ratio. In extension, Chepuri et al. (2018) attempted to establish level-of-service (LOS) criteria using reliability indicators from the travel time data of buses. However, both of these studies considered travel time data from a single vehicle mode to establish these relationships. However, using travel time data from single mode to establish such relationships in disordered heterogeneous traffic which encompasses the multiple vehicle classes may not be a pragmatic approach and lead toward the biased inferences. Recently, Remias et al. (2017) collected travel time data on urban streets in Chennai using Bluetooth scanners. However, the study was focused on testing the Bluetooth scanners in the Indian context. Therefore, any functional relationship of travel time or travel time reliability indices with other variables were not explored. In the present study, the attempt has been made to establish the functional relationship of PTI with space mean speed (SMS). Keeping in view the limitation of previous studies in terms of collected travel time data, a traditional license plate matching technique is used in this study to obtain a sample from different vehicle classes. Further, the effect of the vehicle composition on average travel time is observed using the neural network modeling approach.

3 Data Collection and Processing

Two road links from Surat metropolitan of India city were selected for the present study. Both of these links are part of the Aathwa line corridor, which connects the residential area of the city with the industrial and area. This corridor also connects the airport, several shopping malls and recreational centers of the city, which gives an opportunity to capture the diverse trip data, mostly in the evening peak period. Recent census data of India (MoUD 2016) revealed that the average work trip length of the majority of commuters in urban areas is less than 5 km. Maximum people in the urban area have work trip length of 2–5 km followed by 0–2 km. Using this information, the length of segments for present study was considered to match with this statistic approximately. Table 1 provides the detailed information of the considered segments.

The videographic survey was carried out to obtain data on the required variables. In total, three cameras time synchronized were used for the survey. Two cameras (A and B) were used for license plate recognition (LPR) survey to collect travel time data, and one camera (C) was used as a conventional speed trap. Figure 1 demonstrates the empirical setting of data collection.

While processing the data, registration numbers of vehicles were noted down along with their type and time stamp at the start and end node of the study segment. An attempt was made to capture maximum possible registration number data. In the next step, these data were matched to obtain class-wise travel time data. The obtained travel time data are then arranged according to entry time stamp to estimate the average travel time at 5-min interval. On the other hand, classified volume count (q) and time mean speed (v_{tms}) data were manually extracted from the video footage taken from camera C and arranged at the same time interval. The space mean speed (v_{sms}) of the stream is then obtained using Eq. (3), where N is the sample space.

Table 1 Details of study segments

Road link	Length	Type of facility	Land use	Time of survey
Link-A	2.2 km	Three lane divided	Institutional	8:00–10:30 AM[a] 4:00–06:30 PM[b]
Link-B	1.1 km	Two lane divided	Mixed	8:00–10:30 AM[a] 4:00–06:30 PM[b]

[a]Morning peak, [b]Evening peak

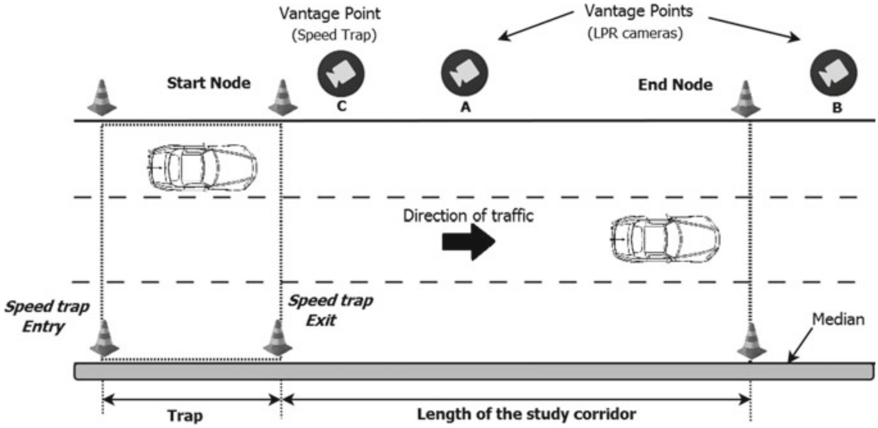


Fig. 1 Empirical setting for data collection

$$v_{\text{sms}} = \frac{N}{\sum_{n=1}^{N} \frac{1}{v_{\text{tms}}}} \tag{3}$$

The present study is arranged in four sections. In the first section, travel time variability on these corridors is analyzed. Various travel time reliability metrics are estimated and studied in the next. The third section discusses the functional relationship between space mean speed and planning time index. Finally, the developed ANN model to map the effect of vehicle composition on the average travel time (ATT) is discussed in the last section.

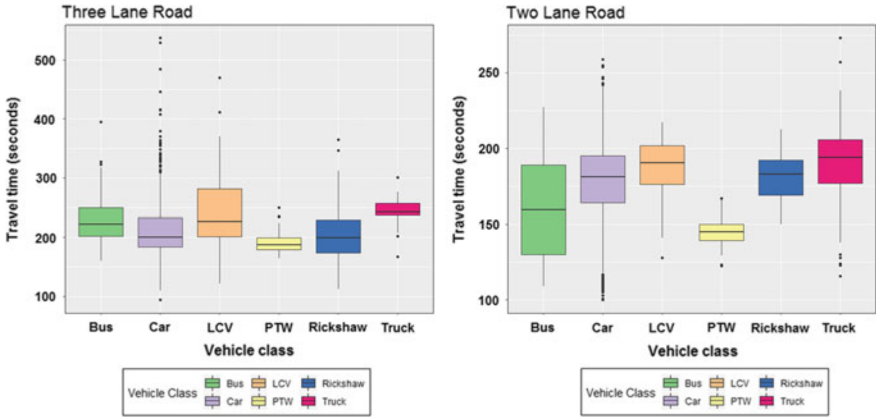
4 Analysis of Travel Time Variability

The obtained travel time data on both of these corridors are explored in two different steps to observe the travel time variability on these sections. In the first step, inter-class travel time variability is observed. Four different vehicle classes are considered for this purpose. In the next step, best fitting distribution to obtained travel time data is observed. Observations from these two steps are discussed below.

4.1 Inter-class Travel Time Variability

To gain initial insight, the inter-class travel time variability was observed at first. Figure 2 represents the observed travel time variation in different vehicle classes.

It was observed from Fig. 2 that travel time required to complete a trip significantly differs according to the mode of travel used for the trip. Travel time data collected



(a) Three Lane Road (b) Two Lane Road

Fig. 2 Inter-class travel time variability

on both of these road sections revealed that powered two-wheelers (PTW) rider completes the considered trip in lesser time compared to other modes, making PTW as the fastest mode of travel on both of these corridors. The observed mean travel time for powered two-wheelers on a three-lane road is 190.54 s, whereas it is 145.20 s on the two-lane road. It can also be perceived that powered two-wheelers move significantly faster on the three-lane road compared to that on the two-lane road. Subsequently, buses are observed as the second fastest mode on the two-lane road with an average travel time of 160.98 s. However, it is to be noticed that maximum observations in the bus travel time data on this section are of school buses and the buses shuttling for the industrial companies. In both cases, delay due to bus-stops is not involved. Moreover, most of these buses are minibusses; hence, we cannot compare their travel time with the usual public transit buses.

In case of a three-lane road, the car is a second fastest mode with an average travel time of 214.94 s followed by the buses for which the observed average travel time is 238.85 s. On this corridor higher variation in bus travel time data can be observed. Trucks and light commercial vehicles (LCV) such as mini trucks and pick-up vans were observed to require a higher travel time on both of the corridors. The observed average travel times for trucks and LCVs on the three-lane road are 243.27 and 242.87 s, respectively, whereas it is observed as 190.22 and 185.55 s, respectively, on the two-lane roads. The higher travel time of these vehicle classes may be due to the loading conditions, and difficulty of driving the bigger vehicle in the urban traffic. On the other hand, despite being quicker, auto-rickshaws require higher travel time on the three-lane corridor as they frequently stop on intermediate station in between the corridor to collect or drop the passengers. In case of a two-lane road, they cannot easily execute the overtaking maneuver, which ultimately results in higher travel time for this vehicle class on these roads, which is reflected in the database.

4.2 *Travel Time Distribution*

In this step, an attempt was made to find the best fitting distribution to model travel time data from both of the study segments. Five well-known distributions viz. normal, lognormal, Weibull, log-logistic, and Burr were evaluated for both the travel time datasets. The Freedman–Diaconis rule (Freedman and Diaconis 1981) was used to decide the optimal bin size. According to this rule, the bin width (h) is given by Eq. (4) where IQR is the inter-quartile range of the data (x).

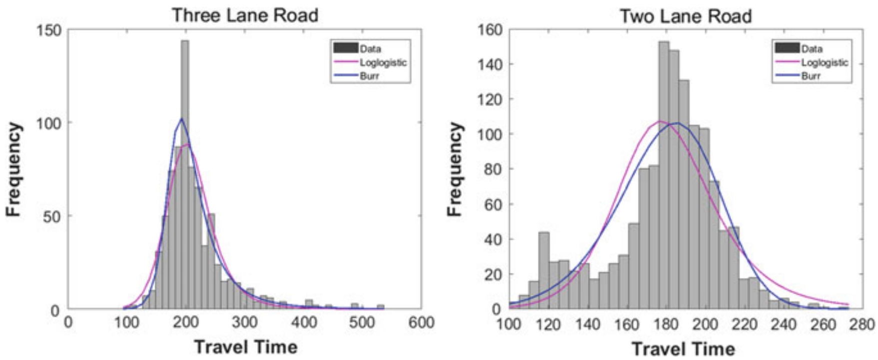
$$h = 2\frac{\mathrm{IQR}(x)}{n^{\frac{1}{3}}} \tag{4}$$

The analysis revealed that the travel time data are skewed and follow Burr as well as log-logistic distribution. The statistical validity of these fits was ascertained by the Kolmogorov–Smirnov (K–S) test. Though there are some other distributions which fit the data, these two distributions were selected because of their ability to describe data with very long upper tails. Table 2 provides the summary of statistical fits, and Fig. 3 demonstrates the fits for both the road segments.

Table 2 Comparison of estimated BTI using mean and median

Road facility	Name of the distribution	Kolmogorov–Smirnov test[a]	
		D	D_{Critical}
Three lane	Log-logistic	0.0492	0.0553
		0.0439	0.0553
Two lane	Burr	0.0206	0.0340
		0.0213	0.0340

[a]All values are at 95% confidence interval



(a) Three Lane Road (b) Two Lane Road

Fig. 3 Travel time distribution

5 Estimating TTR Performance Metrics

As discussed earlier, the collected travel time data on both the corridors demonstrate the positively skewed distribution. For such distributions, measures of central tendency, i.e., mean, and median deviates from each other. In such cases, considering the mean for the estimation of reliability metrics may underestimate these values of these metrics. Therefore in the present study, BTI was calculated for two cases, the first using mean as average travel time and then the second using the median as average travel time. It was observed that the buffer index estimated using median is 7–18% higher compared to the buffer index estimated with mean. Table 3 demonstrates the average results obtained from this analysis. To account for these critical values, median travel time is used to calculate the BTI in the present study.

5.1 Analysis of Travel Time Reliability

All the reliability metrics were estimated for morning and evening hours at an interval of 15 min. Tables 4 and 5 represent the obtained values of these metrics for three-lane and two-lane roads, respectively. From Table 4, it can be identified that in the morning hours between 8:15 and 9.15 am the planning time index is more than 2 for the 3-lane section. This emphasizes that the vehicles that are entering on this corridor during 8:15 to 9.15 am time segment need to plan for double travel time. Similarly, in evening hours, PTI is more than 2 for the vehicles entering after 5:00 pm. In the case of the two-lane section, PTI is around 2 for most of the time segments. So commuters should plan trip wisely on this corridor in both morning and evening peak period. In the case of BTI, it can be observed that BTI for three-lane road is always above 1.2. This shows that one must consider at least 20% cushion time on this corridor for any trip in peak hours. The highest BTI value for this road section is observed as 1.66 in evening peak hours. In case of two-lane road corridor, BTI values consistently fall below 1.25 showing the lesser travel time variation compared to the three-lane corridor. As three-lane corridor comprises one roundabout, the recurrent delay occurring on this corridor is captured through this metric. On the contrary, on

Table 3 Comparison of estimated BTI using mean and median

Time segment	Three-lane road				Two-lane road			
	ATT	Estimated BTI	Median TT[a]	Estimated BTI	ATT	Estimated BTI	Median TT[a]	Estimated BTI
Morning peak	230	1.22	219	1.29	465	1.12	163	1.13
Evening peak	212	1.32	200	1.40	186	1.12	183	1.14

[a]Median travel time values are in seconds

Table 4 Performance of three-lane road

Time of the day	Time segment	Sample size	SMS	ATT	PTI	BTI*	TTI	λ_{skew}	$\lambda_{variance}$
Morning peak	8:00–8:15	37	42.11	188	1.84	1.31	1.29	1.19	0.33
	8:15–8:30	14	34.18	235	2.11	1.19	1.17	1.12	0.27
	8:30–8:45	15	29.2	272	2.42	1.24	1.17	11.91	0.21
	8:45–9:00	34	25.43	312	2.66	1.14	1.12	1.87	0.21
	9:00–9:15	26	30.91	260	2.27	1.21	1.16	16.85	0.25
	9:15–9:30	28	34.06	233	1.96	1.15	1.12	1.88	0.22
	9:30–9:45	49	41.61	193	1.8	1.24	1.23	1.06	0.33
	9:45–10:00	31	46.9	169	1.49	1.18	1.17	1.07	0.23
	10:00–10:15	22	40.03	199	2.01	1.5	1.34	7.34	0.43
	10:15–10:30	24	34.88	231	2.62	1.7	1.49	3.89	0.7
Evening peak	4:00–4:15	63	38.16	213	2.07	1.35	1.27	4.02	0.3
	4:15–4:30	54	37.44	217	2.24	1.44	1.34	3.67	0.24
	4:30–4:45	41	40.91	196	1.79	1.24	1.2	1.96	0.27
	4:45–5:00	53	41.01	203	2.33	1.66	1.51	3.36	0.46
	5:00–5:15	58	40.58	204	1.93	1.31	1.24	1.84	0.3
	5:15–5:30	42	38.01	215	2.15	1.41	1.32	2.54	0.31
	5:30–5:45	79	35.33	237	2.44	1.5	1.35	3.92	0.53
	5:45–6:00	43	40.89	201	2.04	1.4	1.33	1.91	0.35
	6:00–6:15	35	39.46	208	2.1	1.43	1.34	2.98	0.46
	6:15–6:30	59	35.43	229	2.17	1.27	1.24	1.5	0.38

*BTI estimated using median value

two-lane corridor does not involve any intersection delay; therefore, observed travel times are consistent in most of the cases.

Observing at magnitudes of λ_{skew}, and $\lambda_{variance}$ parameters, it can be inferred that travel time on both of these roads are unreliable as $\lambda_{skew} > 1$. However, the value of λ_{skew} is higher for three-lane corridor compared to two-lane corridor which shows more unreliable travel time on three-lane road.

6 Relation Between SMS and PTI

As discussed earlier, collecting stream travel time data in disordered heterogeneous traffic is a challenging task due to the presence of multiple vehicle classes. Therefore, any inference drawn from the travel time data of single mode may lead to the biased inferences. On the other hand, collecting the SMS data at the entry point of the corridor is a relatively easy task with some advanced location-based sensors, and

Table 5 Performance of two-lane road

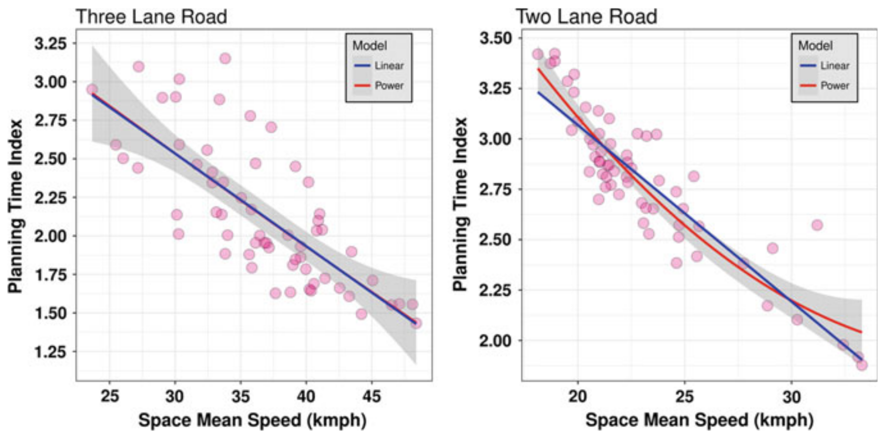
Time of the day	Time segment	Sample size	SMS	ATT	PTI	BTI*	TTI	λ_{skew}	$\lambda_{variance}$
Morning peak	8:00–8:15	52	25.13	160	1.9	1.19	1.17	2.07	0.18
	8:15–8:30	71	26.4	153	1.76	1.19	1.14	2.43	0.24
	8:30–8:45	72	30.82	130	1.54	1.21	1.17	3.02	0.2
	8:45–9:00	84	32.95	122	1.5	1.24	1.22	1.49	0.2
	9:00–9:15	66	21.65	183	1.97	1.07	1.06	1.24	0.1
	9:15–9:30	75	21.58	184	2.03	1.1	1.09	1.85	0.14
	9:30–9:45	77	22.64	173	1.98	1.12	1.13	0.95	0.13
	9:45–10:00	81	22.82	174	1.93	1.1	1.1	1.19	0.14
	10:00–10:15	99	21.17	187	2.02	1.07	1.07	1.17	0.1
	10:15–10:30	67	21.21	187	2.01	1.06	1.07	0.95	0.09
Evening peak	4:00–4:15	119	19.26	207	2.36	1.15	1.13	2.09	0.15
	4:15–4:30	94	20.8	192	2.19	1.14	1.13	2.82	0.17
	4:30–4:45	114	20.28	196	2.13	1.07	1.08	1.13	0.12
	4:45–5:00	92	19.96	200	2.27	1.12	1.12	1.17	0.19
	5:00–5:15	68	22.64	176	1.94	1.09	1.09	0.97	0.16
	5:15–5:30	101	21.75	183	2.04	1.11	1.1	1.15	0.15
	5:30–5:45	54	21	189	2.26	1.21	1.17	5.72	0.21
	5:45–6:00	79	25.22	158	1.76	1.1	1.1	1.38	0.17
	6:00–6:15	88	23.73	169	1.92	1.13	1.13	1.32	0.2
	6:15–6:30	55	23.55	172	2.05	1.22	1.19	3.31	0.23

*BTI estimated using the median value

travel time can be indirectly estimated using these data. Considering this point of view, the functional relationship between SMS and the planning time index (PTI) is investigated in this section. Regression analysis was carried out for this purpose. Five different models viz. linear, exponential, power, inverse, and logarithmic were considered for this purpose. Out of these, exponential and power models were identified as the best models for both two-lane and three-lane road. Figure 4 shows the graphical representation of the developed models.

Three goodness of fit parameters viz. coefficient of determination (R^2), t-statistic, and F-statistic were observed along with the root-mean-square error (RMSE) to identify the best fitting model. Summary of model coefficients and goodness of fit values for three-lane and two-lane road section is presented in Table 6. The RMSE for all these models was calculated using Eq. (5).

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}|Y_i - \hat{Y}|^2} \tag{5}$$



(a) Three Lane Road (b) Two Lane Road

Fig. 4 Relation between space mean speed and planning time index

Table 6 Summary of model coefficients and their goodness of fit values

Section	Model	Model equation	R^2	F-value	RMSE
Three lane road	Linear	PTI $= -0.059 \times$ SMS $+ 4.281$	0.564	75.13	0.1900
	Exponential	PTI $= 6.055 \times e^{(-0.034\times\text{SMS})}$	0.595	85.03	0.1229
	Power	PTI $= 39.175 \times \text{SMS}^{(-0.846)}$	0.582	79.94	0.1383
	Logarithmic	PTI $= -9.712 + (-2.210 \times \ln(\text{SMS}))$	0.538	73.12	0.1979
	Inverse	PTI $= 0.178 + (69.569/\text{SMS})$	0.538	67.63	0.2037
Two lane road	Linear	PTI $= -0.088 \times$ SMS $+ 4.281$	0.796	226.85	0.1378
	Exponential	PTI $= 6.15 \times e^{(-0.030\times\text{SMS})}$	0.823	270.17	0.1322
	Power	PTI $= 38.175 \times \text{SMS}^{(-0.736)}$	0.829	281.15	0.1344
	Logarithamic	PTI $= -9.559 + (-2.071 \times \ln(\text{SMS}))$	0.812	271.13	0.1347
	Inverse	PTI $= 0.178 + (53.974/\text{SMS})$	0.815	255.367	0.1362

After evaluating these parameters, exponential and power models were identified as best fitting models. It can be observed from Table 6 that values of t-statistic and F-statistic obtained for exponential and power models demonstrate the significant relationship between these variables, and more than 70% of the variation in the PTI for one of these sections can be captured using exponential and power models.

Validation of Regression Models

The validation of developed regression models was carried out using the F test, which evaluates the difference in variance of observed and predicted PTI values. As 70%

Table 7 Statistical summary of validation

Model	Three-lane road			Two-lane road		
	Predicted PTI	Variance	R^2	Predicted PTI	Variance	R^2
Power	2.668	0.112	0.842	2.268	0.081	0.757
Exponential	2.657	0.121	0.854	2.253	0.07	0.765

of the total data were used for building the models, the remaining 30% data were used for validation. The mathematical form of F test can be expressed as:

$$F = \frac{\sigma_o^2}{\sigma_m^2} \tag{6}$$

The obtained F-values obtained from the conducted test are reported in Table 7. It can be observed from Table 7 that obtained F-values are higher than critical F-value from standard F-table. However, the exponential model is statistically weak as compared to the power model due to its lower F-value.

7 Analysis of Travel Time Variation Using ANN

Artificial neural network (ANN) is a biologically inspired network, which is designed as an interconnected system of processing elements, called neurons (Kustrin and Beresford 2000). Each of these neurons has a specified number of inputs and outputs. The developed network obtains knowledge through learning (e.g., recognizing patterns) from these data and stores these learning in the form of synaptic weight associated with each of the interconnecting links. Figure 5 represents the typical example of an artificial neuron.

Mathematically, a synaptic weight of the connection at the typical jth neuron shown in Fig. 5 can be expressed as:

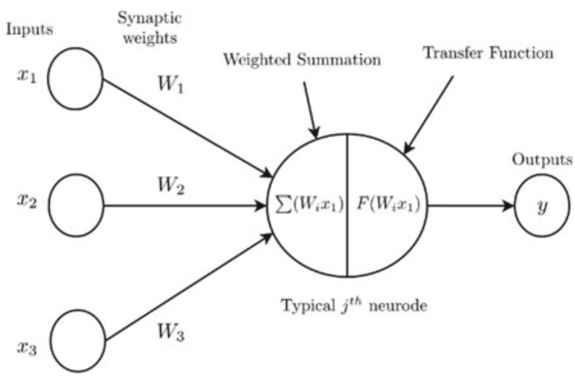


Fig. 5 Function of artificial neuron

$$\mathrm{Net}_j = \sum_{i=1}^{n} w_i x_i + \theta_j \tag{7}$$

where $(x_1, x_2, \ldots, x_n)$ are the input values given to the neuron, $(w_1, w_2, \ldots, w_n)$ are the values of associated synaptic weights, and θ_j is the node threshold. Now, the output at jth neuron, i.e., O_j can be obtained from Eq. (7), where $f(\mathrm{Net}_j)$ is the activation function which is used for mapping the input pattern of a neuron to the specified output range.

$$O_j = f(\mathrm{Net}_j) \tag{8}$$

The present study uses the Log sigmoid activation function for both input layer and hidden layer activation.

7.1 Development of Back-Propagation Neural Network (BPNN) Model

Developing the neural network model consists of two basic steps. In the first step domain of the problem is selected. In the next step, the design of network architecture is carried out. Designing the network architecture encompasses the selection of four parameters, i.e., input and output data, number of hidden layers, tolerance level, and type of learning paradigm to train the network. In the present study, a feed-forward neural network with single hidden layer is considered. The log-sigmoid transfer function was used in the network. The supervised training approach with a back-propagation algorithm was adopted to train the developed network. MATLAB (2017) neural network toolbox was used to achieve the objective. Figure 6 represents the developed network architecture.

7.1.1 Optimization of Network Architecture

In the present study, the optimum network is recognized as the network that yields the acceptable performance, i.e., minimum error at minimal number of nodes and the hidden layers. To obtain such network, sensitivity analysis needs to be carried out. To achieve this objective, the prediction error was estimated for the fixed range of number of hidden neurons and the number of hidden layers in the network. In case of hidden neurons, 5 cases were considered in which hidden layer neurons were changed from 10 to 20 at an interval of 2 neurons. Further, for each considered case of a hidden neuron, 10 subcases were considered where a number of iteration was changed in steps of 100 starting from 100 up to 1000. Therefore, in total 25 cases were considered and for each scenario, and RMSE for these cases were estimated using Eq. (5). The minimum error for a three-lane model and two-lane model were observed

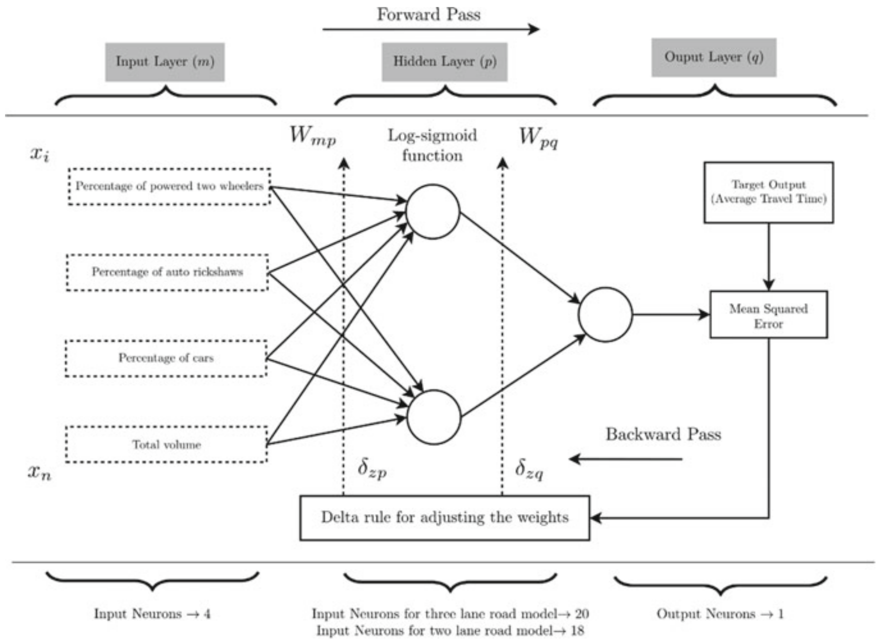
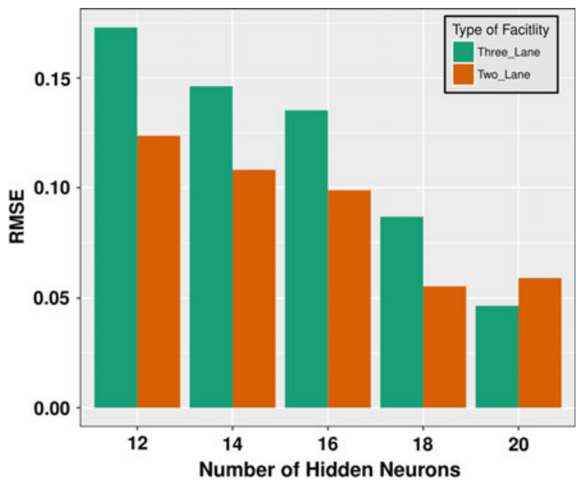
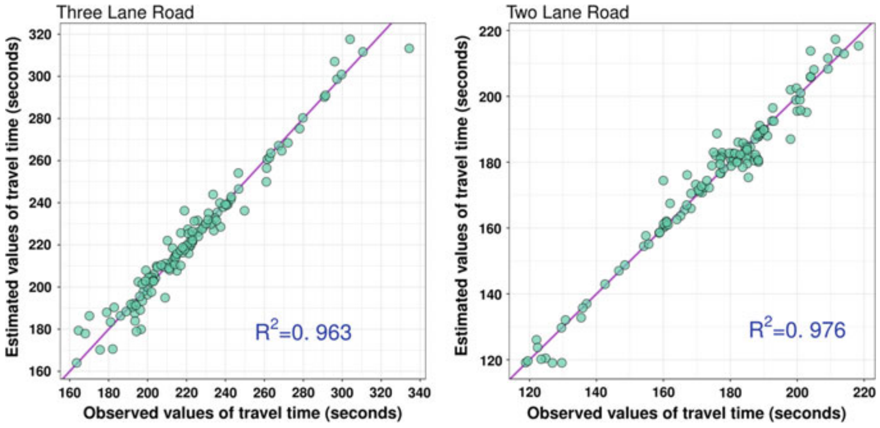


Fig. 6 ANN architecture of the developed model

at 20 and 18 hidden layer neurons, respectively, at 500 number of iterations. Figure 7 shows the error propagation for consideration of a different number of hidden layer neurons at 500 number of iterations.

Fig. 7 Optimum number of hidden nodes for the developed models





(a) Three Lane Road (b) Two Lane Road

Fig. 8 Performance of the developed model for training database

Table 8 Summary of validation results

	Three-lane road model	Two-lane road model
t-statistic	−0.098	0.1
t-critical	1.983	1.983

7.1.2 Performance of ANN Model During Training and Testing

The 70–30% data split is considered for the training and testing purpose of the developed ANN models. The value of R^2 obtained during the training of two-lane model was 0.976, whereas it was 0.963 for the three-lane model. This demonstrates that the developed neural network model is capable of capturing the different values of travel time. The estimated values of travel time during the model training are presented in Fig. 8. To validate the developed models, Student's t-test was carried out to evaluate the significant difference between the means of observed travel time and estimated travel time values. The value of Student's t-statistic is estimated using Eq. (9).

$$t = \frac{x_o - x_m}{\sqrt{\frac{\sigma_o^2}{N_o} + \frac{\sigma_m^2}{N_m}}} \tag{9}$$

where x_o and x_m are the observed and modeled average travel time values, σ_o^2 and σ_m^2 are the variance of observed and modeled travel time, and N_o and N_m are their respective sample sizes.

The obtained results from this test for both the models are presented in Table 8. It can be perceived from these values that estimated values of average travel time from the model are not significantly different from the obtained values.

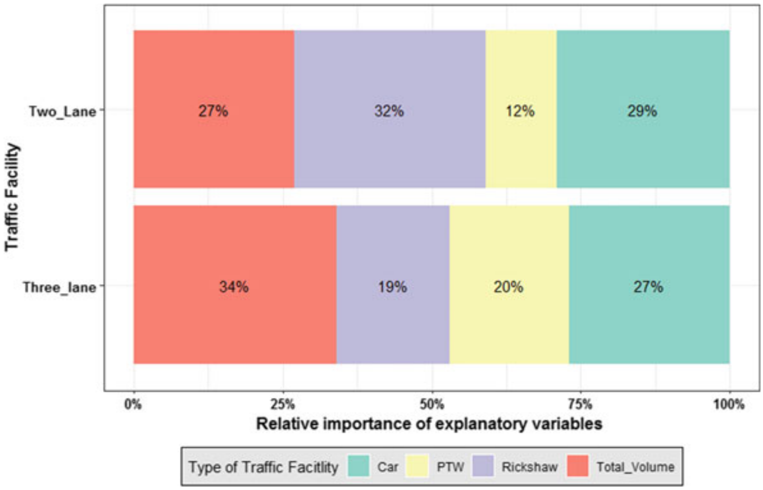


Fig. 9 Sensitivity input parameters toward the output parameters

7.2 *The Relative Importance of Input Parameters*

The knowledge that neural network obtains during the learning process remains implicitly encoded in the form of weights and bias values. However, the information regarding the relative importance of explanatory variables considered in the network to represent the dependent variable can be obtained. In the present study, Garsons' algorithm (Garson 1991) is used to achieve this objective. The results obtained from this algorithm for three-lane model and two-lane are represented in Fig. 9. It is observed that total volume and percentage of cars are the significant variables for three-lane road model, whereas total volume along with the percentage of auto-rickshaw is more significant in case of the two-lane model.

8 Summary and Conclusion

Traveler's decision of selecting a route, mode, and departure time heavily relies on the expected travel time to complete the trip, as well as the reliability associated with this travel time. This connection of travel time reliability with the decision-making process of travelers makes it one of the highly cited metric for evaluating the performance of traffic facilities. However, very limited studies have been reported in the literature which tries to evaluate this aspect of Indian roads. Today's rapidly changing urban scenario clearly divulges the need of studying the travel time characteristics on Indian urban roads. With this motivation, travel time variability of two Indian road corridors is assessed in the present study using travel time reliability metrics. Both of these corridors demonstrated the higher travel time variability, i.e., lower reliability to complete the trip in expected time. Therefore, this suggests travelers to consider an appropriate cushion time while planning the trip. The developed regression models revealed that more than 60% of the variation in PTI for both of these corridors can be captured using the SMS measured at the entry point of the section. As the study was conducted in disordered heterogeneous traffic conditions, an attempt was made to identify the effect of vehicle composition in traffic stream on the variation of travel time. ANN approach is considered to map this relation. It was observed from the developed models that, along with the total volume of traffic, composition of auto-rickshaw, as well as cars, significantly affect the average travel time of the stream.

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Fuzzy Rule-Based Travel Time Estimation Modelling: A Case Study of Surat City Traffic Corridor

Krishna Saw, Bhimaji K. Katti, and Gaurang J. Joshi

Abstract Traffic and transport planning in fast-growing metropolitan cities in India is the most challenging task for the urban transport planner, in view of the faster traffic and transport demand growth observed recently. Increase in travel times and their variation are significant issues on urban corridors owing to heavy traffic volume and congestion. The level of service is decreasing, and vehicular delays are intolerable during peak periods. The situations call for in-depth analysis of associated attributes. An important traffic corridor of Surat, a fast-growing metropolitan city in the state of Gujarat in India, is selected for studying the attributes and developing the estimation model as a typical case of urban corridor. The attributes associated with travel time are uncertain and imprecise in nature due to the dynamic traffic environment. Therefore, a soft technique fuzzy rule-based approach has been advocated for developing the travel time estimation model. Estimation model is further validated with field data, and sensitivity analysis with respect to identified attributes has been carried out.

Keywords Fuzzy logic · Intersection factor · Dynamic traffic · Travel time attributes

1 Introduction

Accurate and reliable travel time estimation of traffic corridor is one of the important objectives in view of smart urban mobility for both transport planner and trip makers against the fast-growing urban vehicle and traffic. In recent years, traffic pressure, owing to the significant growth in vehicle population in most of the metropolitan cities, particularly in developing countries, has reached an alarming proportion owing to increased economic activities. It has adversely affected the trip makers with anxiety and stress. Travel time has bearing on travel delays, route choice and setting of departure time on part of the trip makers. Travel time of the corridor is affected by many dynamics such as traffic condition, intersection and control system and

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varies from place to place with time. This essentially calls to probe the influencing attributes related to travel time for recurrent conditions in particular, for their role in the development of travel time estimation model, useful in urban traffic control and management. The present study attempts to incorporate these very factors in travel time estimation modelling. As travel time attributes are embedded in uncertainty and impreciseness due to the dynamic traffic environment, fuzzy logic soft computing technique is attempted to employed here for travel time estimation model to deal with vague and imprecise information. With this objective, an important corridor reflecting functional characteristics of arterial have been selected in Surat, a fast-growing metropolitan city of Gujarat state in India for the study purpose.

2 Travel Time: Literature Review

Travel time on urban corridors is considered one of the vital parameters for the planning, management and operation of the transport system and is mainly influenced by physical features of the corridors and varied traffic characteristics in terms of mixed traffic. Road width, geometry and intersection controls are part of physical characteristics, whereas traffic flow and density are related to the traffic characteristics under mixed traffic conditions. Das et al. (2016) have dealt some of these attributes and their impact in traffic congestion modelling by the speedo-graph approach. Wang et al. (2016) recognize the influence of lane width and number of traffic streams in their study. The impacts of traffic volume on travel time are covered by Jiang and Zhang (2001) and Lin et al. (2005) in their travel time prediction study and observed the significant increase in travel time when traffic volume reaches near capacity of the roads. Vlahogianni (2006) noted influence traffic composition on flow speed along with the impact on motorcycle speed significantly influenced by the presence of the truck. The intersection configuration and control system are equally important as they act as sinks and bulging spots traffic interruption. Jenelius and Koutsopoulos (2013) covered both signal and no signal intersections in his study "travel time of urban road using probe vehicle data, Stockholm, Sweden" and found that delay at the non-signalized intersection is shorter than the signalized intersection. It is also found that delay at intersection constitutes approximately 25% of total travel time.

As travel time governed by various dynamic factors, estimation of accurate and reliable travel times becomes complex. Various approaches have been employed by research workers to develop estimation modelling for travel time such as multi-linear regression, soft computing technique, analytical approach, time series and simulation technique. Multi-linear regression approach has been advocated for travel time estimation due to its simplicity by Sisiopiku et al. (1994) and Fils (2012). Similarly, Yang (2011), Ravi Sekhar et al. (2012), Saw et al. (2018) have adopted for developing regression model in their study. Skabardonis and Geroliminis (2005) and Bhaskar et al. (2010) have adopted loop detector and signal control data in developing analytical model, where the relationship is formed in accordance with traffic flow theory. Time series method for the estimation of travel is suitable when huge historical data is available. Time series data analysis is part of the proposed model by Yang (2005) and Hu and Ho (2010). They found that the performance of the time series model is largely affected by the outliers of the historical time series data. Khoei et al. (2013) have applied SARIMA modelling on Bluetooth data. Artificial neural network (ANN) has been proved to be effective to represent nonlinear behaviour (Jiang et al. 2017). Dharia and Adeli (2003) and Lee (2009) have adopted ANN for travel time modelling. Liu et al. (2006) has adopted the microscopic simulation analysis technique in their study for travel time estimation for arterial road considering total travel time as a sum of link cruising times and intersection delays.

However, the traditional models are not effective to deal with uncertainty and impreciseness prevails in travel time attributes due to significant fluctuations in the traffic environment including driver behaviour. Therefore, the use of fuzzy logic advocated by Zadeh (1975) to address the uncertainty in database bears importance in developing travel time estimation model.

3 Methodology

The present study progresses in four phases, commencement with the identification of a study corridor and noting the physical features covering intersection control system and lane configuration, intersections interspacing and adjacent land use pattern. For the purpose of the study, the corridor has been segregated into three segments based on physical and traffic characteristics to capture the spatial and temporal variations in travel. Different field studies such as classified traffic volume count and traffic composition and speed profile survey to note travel time were carried out in the second phase. Videography technique was employed for classified volume count and composition, and probe vehicle technique was employed to note the travel time on segment basis. Both traffic volume and speed profile were carried out concurrently. In the third phase, data analysis such as traffic volume and composition, intersection impacts and travel time on segment basis has been conducted. The fourth stage covers the building of fuzzy rule-based travel time estimation model and sensitivity analysis.

4 Study Corridor and Field Studies

4.1 Corridor Location

A traffic corridor of length 10.8 km of Surat city has been selected as a typical case for the present study. Surat is a fast-developing industrial city in Gujarat state having nearly 4.5 million population to date. The corridor starts from Rahul Raj a popular mall having a significant number of consumers on one end and continues up to railway station via Athwa gate-Ring Road. Some part of this corridor come under South West

Zone, i.e. 5.5 km, and remaining parts come under Central Zone. The district court and airport are also situated on this corridor. The corridor has been divided in three segments of length 3.8 km, 1.7 km and 5.3 km, respectively, based on traffic and physical characteristics. Segment I and III are 3 lanes and 2 lanes, respectively, and Segment II is 2.5 lanes. Service lane available on both sides of Segment I and Segment III passes through flyovers. Segment I is a commercial dominated area, Segment II is recognized for educational and government institutions, whereas Segment III is dominated by textile markets. The location diagram of study corridor with classified volume count (CVC) locations is shown in Fig. 1a, and the schematic diagram of the corridor is depicted in Fig. 1b indicating the location of intersection C_1 to C_5. C_1, C_2 and C_3 are uncontrolled three-legged intersections, C_4 is five-legged signalized intersections, whereas C_5 is three-legged signalized. Interspacing of intersection is shown in Table 1.

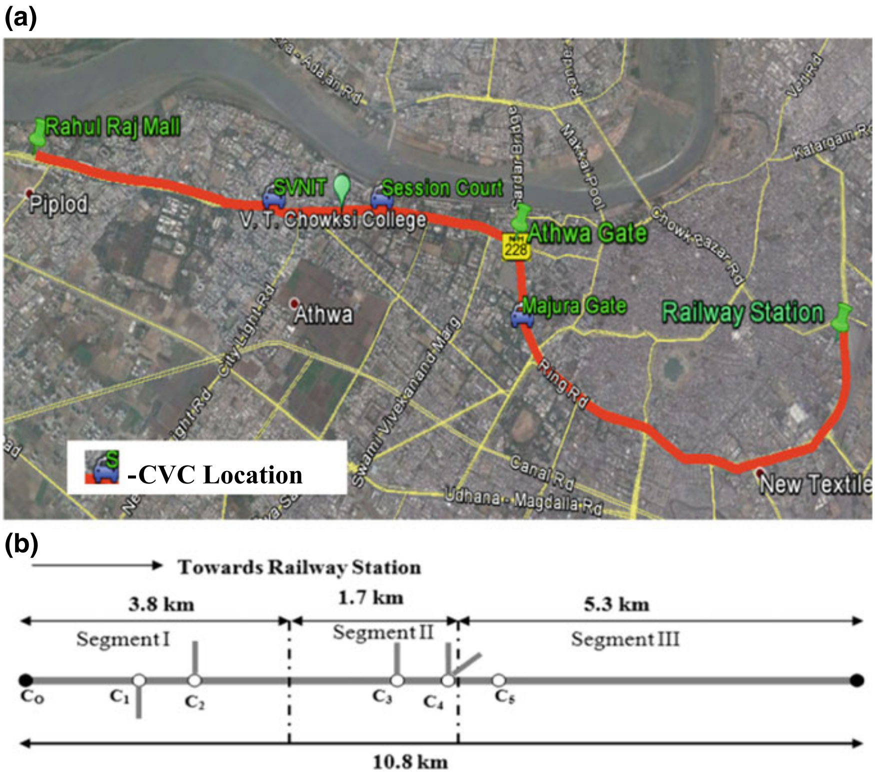


Fig. 1 **a** Study corridor. **b** Schematic diagram for corridor I

Table 1 Interspacing of intersection

	C_0–C_1	C_1–C_2	C_2–C_3	C_3–C_4	C_4–C_5	C_5–End
Distance (m)	1500	600	3000	400	500	5300

4.2 Field Surveys

The main field surveys carried under this study to capture the necessary traffic data are traffic volume and composition survey and speed profile survey. Both surveys were conducted simultaneously covering morning and evening peak and off-peak periods.

Traffic Volume and Composition Survey
One of the fundamental measures of traffic characteristics on road is traffic volume in a given time period. Videography technique was employed in the present study to capture the mix traffic flow at the identified midway locations as shown in Fig. 1a. Videography recording was carried segment wise for both morning and evening peaks and off-peak periods simultaneously at the three locations of the corridors along with other surveys. The classified volume data is extracted at the interval of five minutes. Vehicles are categorized, namely two wheelers (2Ws), auto rickshaw (3Ws), car (4Ws) and commercial vehicles (CVs).

Speed Profile Survey
Travel time survey in the present study was conducted using GPS laden Probe Vehicle of 2Ws being dominated mode of transport in the study area. Basically, GPS laden Probe Vehicle provides the spot to spot speed profile. The sample size in such method results in lower as compared to the licence plate method but has the merit of the spot to spot speed at any particular time. In this study, a total of 13 runs in each direction has been carried out over a length of 10.8 km covering peak and off-peak periods. Overall 78 samples form with both up and down three segments' movements together. The platoon effect in traffic flow enhances the concept of sample size as shown in Fig. 2.

5 Field Observation and Analysis

5.1 Traffic Volume

Hourly traffic volume analysis on segment basis has been carried out as shown in Fig. 3. Spatial and temporal variations are observed in traffic flow due to variation in land use and activity pattern along the segment. Segment II has the highest hourly traffic flow, whereas Segment I has the lowest. Traffic variations can be noticed for peak and off-peak periods. Further, the hourly traffic volume ranges for three

Fig. 2 Platoon flow behaviour

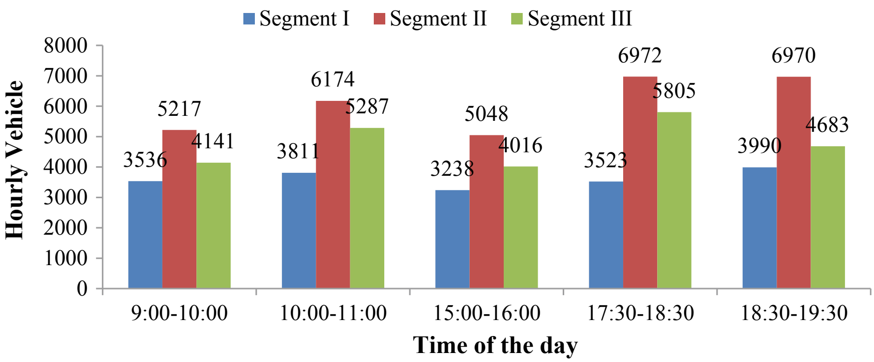


Fig. 3 Traffic volume

segments for different time periods on the lane basis are shown in Table 2 for comparison purpose. Average hourly traffic volume of the day for three segments is presented in the last column of the table. It indicates that average hourly traffic flow per lane is same for Segment II and Segment III.

Table 2 Traffic volume of corridor I (vehicle per hour per lane)

Segment	Morning peak	Afternoon off peak	Evening peak	Segment (average)
Segment I	1025–1340	950–1325	1050–1400	1180
Segment II	1790–2750	1760–2180	2570–3050	2450
Segment III	2200–3050	2070–2320	2140–3280	2480

Table 3 Traffic composition (%)

Segment	Time period	2W	3W	4W	CV
Segment I	Morning peak	50	17	31	2
	Afternoon off peak	50	14	32	2
	Evening peak	53	14	31	2
Segment II	Morning peak	60	14	25	1
	Afternoon off peak	64	12	24	1
	Evening peak	65	12	22	1
Segment III	Morning peak	64	11	22	2
	Afternoon off peak	65	11	19	2
	Evening peak	64	17	18	1

5.2 Traffic Composition

The mixed traffic in India comprises 2Ws, 3Ws, 4Ws and CVs. The presence of bicycles or animal-drawn vehicles is hardly seen on the main corridors. Table 3 provides traffic composition for the different time periods of the day on segment basis. 2Ws are dominating modes of traffic with a variation of 50–64%, followed by 4Ws 18–32%. % of 3Ws is approximately the same except few and % of CVs is very negligible. Traffic flow behaviour under such mixed traffic conditions plays a vital role in vehicular movement but very few studies have considered the influence of such mixed flow on travel time variation. The mixed traffic is converted into passenger car unit (PCU) by referring to PCU factors specified by IRC 106-1990 for urban mid-block sections in the development of a model to reflect the impact of traffic composition.

5.3 Intersection Factor (IF)

The study corridor has a number of intersections. Intersection type and configurations, control system and level of crossing traffic matter on the degree of impedance for main traffic flow and accordingly main traffic speeds drop down (Susilawati 2011; Jenelius and Koutsopoulos 2013). This fact has been considered here by introducing intersection factor (IF) defined as average segment speed to the drop-down speed at the intersection. If more number of intersections is there in segment, the cumulative effects are to be considered as segment intersection factor and it is

$$\textbf{Intersection factor (IF)} = \sum_{i=1}^{n} \left(\frac{\boldsymbol{Average\ Segment\ Speed}}{\boldsymbol{Drop{-}down\ Speed\ at\ intersection\ i}} \right) \quad (1)$$

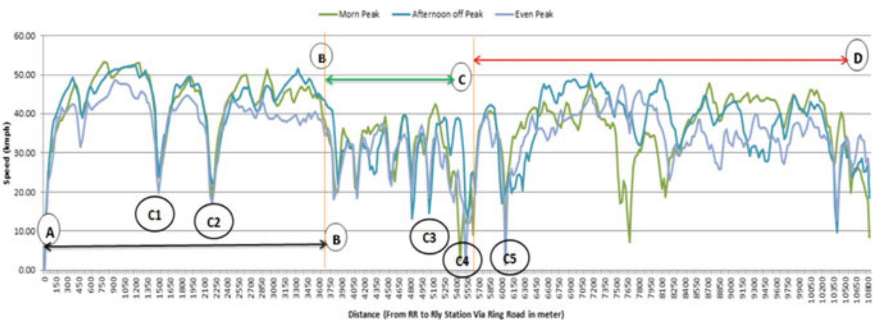
where,

n Numbers of intersections in the segment.

The drop-down speeds are observed from the speedo-graphs obtained in the speed profile survey. A drop-down speed is the crawling speed of the Probe Vehicle at the intersection. IF is computed using Eq. (1), and the statistical observations for are shown in Table 4. It is observed that Segment II bears higher IF on account of two signalized intersections followed and lowest in Segment I which consist of minor intersections. Typical space-speed profiles of morning and evening peaks and afternoon off peak are indicated in Fig. 4. The speed drop with respect to average speed can be noted at intersections C_1, C_2, C_3, C_4 and C_5 at 1500 m, 2100 m, 5100 m, 5500 and 6000 m, respectively.

Table 4 Intersection factor

Segment	Average	Standard deviation	Minimum	Maximum
Segment I	6.4	4.0	3.3	17.2
Segment II	22.2	7.6	8.3	30.0
Segment III	14.1	10.4	1.3	28.5



AB- Segment I, BC – Segment II, CD – Segment III

Fig. 4 Space-speed profile of corridor

Table 5 Travel time statistics (min/km)

Time period		Segment I	Segment II	Segment III
Morning peak	Average	1.5	2.86	1.74
	Standard deviation	0.18	1.09	0.24
	Minimum	1.36	1.82	1.5
	Maximum	1.82	4.29	2.1
Afternoon off peak	Average	1.38	2.11	1.87
	Standard deviation	0.07	0.11	0.12
	Minimum	1.3	2	1.73
	Maximum	1.43	2.22	1.94
Evening peak	Average	1.61	2.88	2.34
	Standard deviation	0.17	0.85	0.57
	Minimum	1.43	2	1.88
	Maximum	1.88	4	3.33

5.4 Travel Time Analysis

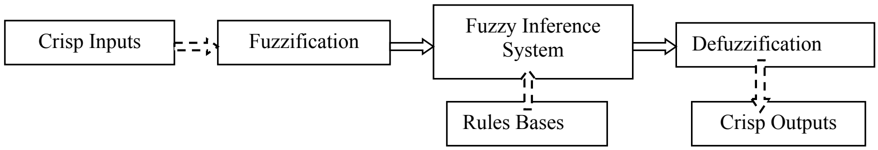
Observed travel time/km is noted on segment basis referring to speed profile collected Probe Vehicle and statistical observations of travel time are summarized in Table 5. Variation in average travel time (ATT) is observed with respect to space and time on account of variations in traffic flow and IF. Highest travel time is to be observed in Segment II, followed by Segment III and lowest in Segment I. Lowest values of traffic flow per lane and IF in Segment I are responsible for low travel time. Similar is the pattern of standard deviation for these segments.

6 Development of Fuzzy Rule-Based Travel Time Estimation Model (FRB–TTEM)

This approach addresses the uncertainty prevailing in attributes pertaining to the traffic condition, and the model building follows Mamdani's consideration. The model structure and particulars of inputs are briefed in subsequent sections. The model developed here considering conversant of mixed mode into equivalent PCU.

6.1 Model Structure

Development operation of fuzzy rule-based model operation takes place in three steps of fuzzification of crisp inputs, fuzzy inferences system and defuzzification of fuzzy value into a crisp value.



6.2 *Fuzzy Model Operation*

6.2.1 Fuzzy Inputs and Fuzzification

Fuzzification is an important step in the fuzzy logic theory, which converts crisp inputs into fuzzy sets. The main inputs of the model on segment basis are as under.

- Traffic volume in PCU/h/lane (000);
- Intersection factor (IF).

Travel time/km (min) is a model output variable.

Table 6 provides a categorization of variables in linguistic terms, the membership shape adopted and input ranges for a particular input level. Here, triangular and trapezoidal membership functions (MFs) are preferred for their simplicity without losing accuracy (Ishizaka 2014). Their ranges and overlapping are based on a trial basis. Four levels are considered from low intensity to very high intensity for all the inputs and outputs.

MFs' particulars for traffic variable and travel time output are shown in Fig. 5a, b, respectively, for illustration purpose.

Table 6 Range of variable used in FRB-TTM

Variable	No. of MFs	Linguistic variable	Type of MF	Fuzzy no.
PCU/h/lane	4	Low	Trapezoidal	[0.5 0.5 0.6 0.85]
		Medium	Triangular	[0.7 1 1.3]
		High	Triangular	[1.2 1.5 1.9]
		Very high	Trapezoidal	[1.65 1.9 2.0 2.0]
IF	4	Low	Trapezoidal	[0 0 2.5 7]
		Medium	Triangular	[5.0 10.0 15.0]
		High	Triangular	[12.5 17.5 22.5]
		Very high	Trapezoidal	[20.0 25.0 30.0 30.0]
TT/km	4	Low	Trapezoidal	[1.3 1.3 1.5 1.7]
		Medium	Triangular	[1.5 1.8 2.3]
		High	Triangular	[2.0 2.3 2.8]
		Very high	Trapezoidal	[2.6 3.2 3.5 3.5]

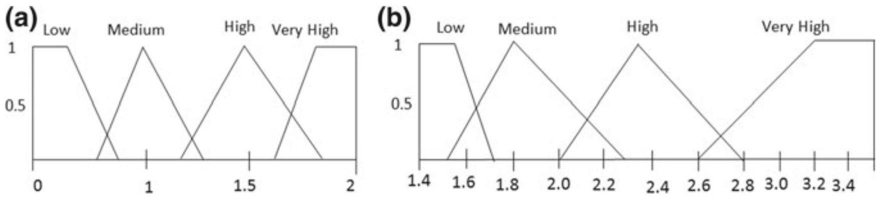


Fig. 5 **a** MFs for PCU. **b** MFs for travel time

6.2.2 Fuzzy Inference System

Fuzzy inference is a process of mapping given input to an output. The mapping provides a basis from which decisions can be made. In this study, fuzzy inference system addresses the above-said inputs and output and follows Mamdani fuzzy inference system for the development of the model. Fuzzy rule-based system is generated with reference to inputs and the output. The number of MFs of input variables is decisive in deciding number of "IF-THEN" rules. Here a total 16 "IF-THEN" rules (4 * 4) were framed. Some rules are mentioned below for illustration purpose.

IF<PCU/lane/h is low> and <IF is low>**THEN**<TT is low>
IF<PCU/lane/h is medium> and <IF is medium>**THEN**<TT is medium>
IF<PCU/lane/h is High> and <IF is medium>**THEN**<TT is medium>
IF<PCU/lane/h is Very High> and <IF is High>**THEN**<TT is Very High>

The antecedents which are the first part of the rule provide fuzzy set input, whereas the second part of **THEN** are consequents to represent the output for the rules framed.

6.2.3 Defuzzification

This is the process of generating a crisp result in fuzzy logic for which generally centroid method is adopted to obtain the crisp outputs. This method builds the resultant MFs by taking the algebraic sum of outputs from each of the contributing fuzzy sets. The typical output value of travel time/km is 2.38 min, for the considered inputs, as shown in the MATLAB snapshot (Fig. 6).

Surface plots for predicted travel time/km in min by the developed model are shown in Fig. 7.

6.3 Measurement of Model Effectiveness and Validation

Three different measures are used for evaluating the performance of the FRB-TTEM; root mean square error (RMSE), mean percentage error (MPE) and mean absolute

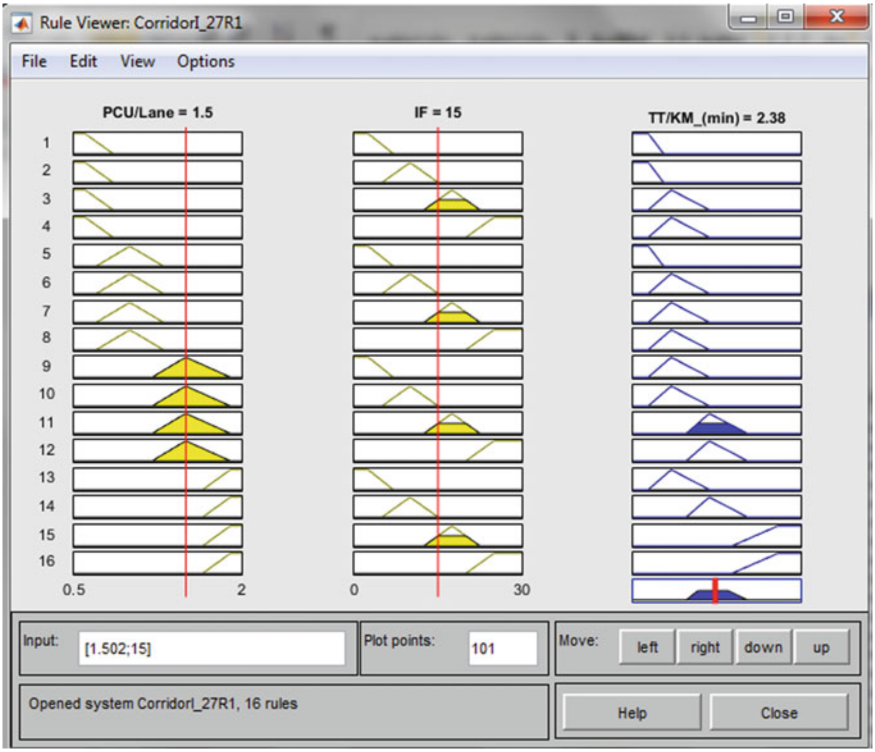


Fig. 6 Typical MATLAB snapshot of rule viewer window

Fig. 7 Surface plots for predicted travel time

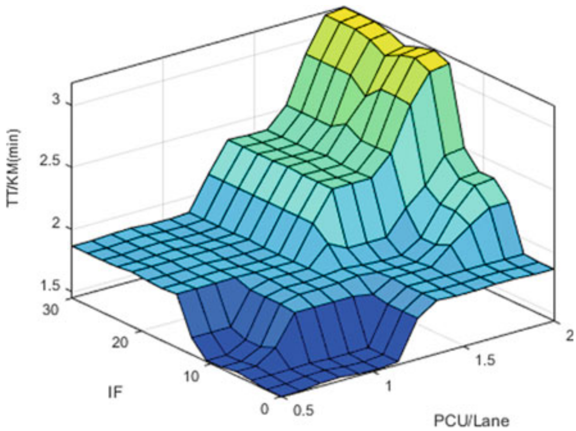
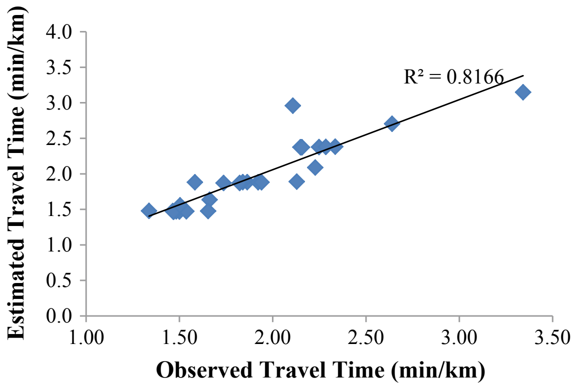


Fig. 8 Observed and estimated travel time



percentage error (MAPE). RMSE and MAPE provide information about error variance, whereas MPE gives an indication of estimation bias (Krishnamoorthy 2008). The performance measures RMSE, MPE and MAPE are 0.21 min, −3.16% and 6.70%, respectively, and are quite satisfactory. Further, the estimated values obtained using model are then compared against the field observed values as shown in Fig. 8 indicating good agreement between them with R^2 as 0.81. Moreover, estimated travel times of model for typical situation are verified with observed one and are found to be satisfactory.

7 Sensitivity Analysis

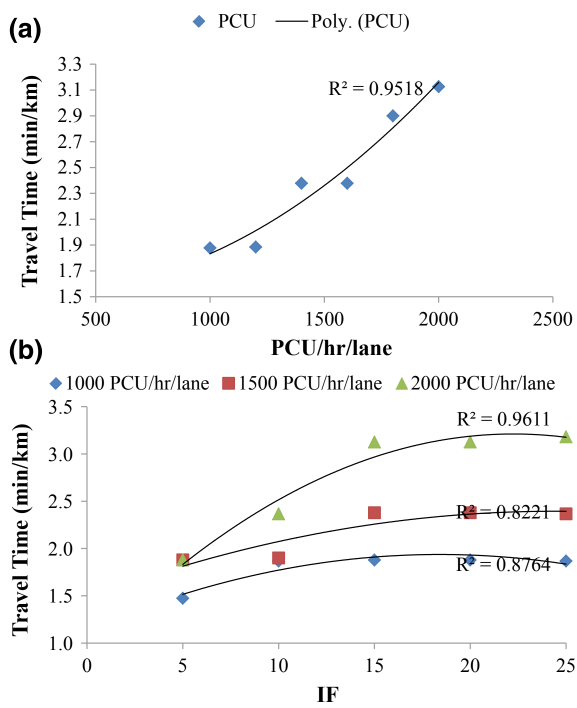
Model FRB-TTEM is further subjected to sensitivity analysis to realize the impact on travel time with reference to two parameters:

1. Traffic volume (pcu/h/lane);
2. Intersection factor (IF).

7.1 Traffic Volume

Variation in travel time variation with reference to increment in hourly traffic volume in the range of 1000–2000 vehicles keeping IF as 15 is shown in Fig. 9a. As anticipated, travel time increases as traffic flow increases. It indicates that travel time (min/km) increases by 66% with an increment of hourly traffic volume from 1000 per lane to 2000 per lane at IF value of 15.

Fig. 9 **a** Travel time with variation in traffic volume/lane. **b** TT/km with variation in intersection factor



7.2 *Intersection Factor (IF)*

Travel time variations with respect IF at three levels of traffic volume is shown in Fig. 9b. As expected, when IF increases, the speeds diminish and travel times are certain to rise because of the increase in traffic impedance. It is to be observed that the impact of IF at high traffic on travel time volume is more.

8 Conclusion

Unprecedented growth in urban traffic and travel demand is taken place in metropolitan cities in India due to economic development and urbanization. Eventually, this has a significant impact on the urban road network and leads to substantial degradation in traffic quality which is reflecting an increase in travel time. It is vital to understand the associated parameters of travel time closely for their likely impacts. The study carried out in this regard on an important city corridor reveals the role of intersection factors (IF) apart from the varying traffic volume and composition under mixed traffic environment. Looking at the uncertainty and impreciseness component in attributes, fuzzy rule-based travel time estimation model (FRB-TTEM) is found suitable in estimating the travel times in dynamic traffic environment. The model finds wide application in measuring the traffic flow status and provides the base for improvement measures as part of traffic control and management of urban corridors. The result of sensitivity analysis of the model clearly reveals that traffic volume and intersection factor have considerable impact and travel time significantly increases at higher traffic flow at higher intersection factor.

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Calibration of SUMO for Indian Heterogeneous Traffic Conditions

Yadavilli Sashank, Nitin A. Navali, Arjuna Bhanuprakash, B. Anil Kumar, and Lelitha Vanajakshi

Abstract Efficient modelling of vehicular traffic is a challenging task in the context of Indian traffic conditions. One of the approaches for modelling traffic is using simulation. Though there are several traffic simulation software available, all of them are developed for the lane based and homogeneous traffic conditions. However, traffic conditions in many countries are heterogeneous and lane-less and for simulating such traffic, either specific software needs to be developed or calibration of existing software for such traffic conditions is required. For example, one of the commonly used software, namely VISSIM can be calibrated for such traffic conditions and is already reported in literature. However, VISSIM being licensed software, researchers have developed an open source software, namely Simulation of Urban MObility (SUMO). Though the initial development of SUMO focused on homogeneous and lane disciplined traffic, later researchers started developing modules for the Indian traffic with its wide mix of vehicle types and improper lane discipline. This paper presents a methodology for the calibration of SUMO for Indian heterogeneous traffic conditions by calibrating its parameters. Data from a 2 km segment in Chennai was used for the calibration. In the first level, parameters that can affect the driving behaviour under such conditions were identified using sensitivity analysis and one-way ANOVA test. Then optimal combination of parameters were identified using

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Genetic Algorithm (GA). Performance comparison was done with calibrated VISSIM for the same test bed. Average speed obtained from both the simulation software (VISSIM and SUMO) were compared and the errors were calculated in terms of Mean Absolute Percentage Error (MAPE) with respect to actual speed values. Results were found to be comparable, indicating that SUMO can be calibrated for simulating Indian traffic.

Keywords SUMO · Calibration · Genetic algorithm · ANOVA

1 Introduction

Management of road traffic is becoming an increasing concern, especially in urban areas. Understanding and resolving this problem is an immediate need to reduce the negative impacts caused due to congestion. Various operational and management strategies are being proposed by researchers to address the problem of congestion and each of them should be tested in simulation environment before its implementation, as it is "faster, safer, and cheaper than field implementation" (Park and Schneeberger 2002). It is a common practice to experiment with traffic networks in a simulated environment as experimenting with traffic in the real environment is not practical. For example an intersection can be simulated for different signal timing plans and its effect (reduction in delay, queue length, etc.) can be found before implementing in the field.

In a broad sense, traffic simulation software can be classified depending on the detail of the classification as Macroscopic, Mesoscopic, and Microscopic (Ratrout and Rahman 2008). Currently, there are several traffic simulation software and they all use one of these traffic modelling approaches. The available software include TRANSIMS (Smith et al. 1995), Transmodeler (TransModeler Manual), MATSim (Horni et al. 2016), MITSIMLab (Yang and Koutsopoulos 1996), AIMSUN (Casas et al. 2010), CORSIM (Halati et al. 1997), Paramics (Cameron and Duncan 1996), SimTraffic (Husch and Albeck 2000), VISSIM (VISSIM Manual 2010), SUMO (Behrisch et al. 2011), etc. Some of these focuses on the simulation of a wide area, while some others concentrates on the behaviour of the vehicle. VISSIM, AIMSUN, MATSim, and SUMO simulate traffic continuously, whereas ARCHISIM, TRANSIMS, Paramics, and CORSIM, use a discrete system. In terms of ease of coding of the road network, VISSIM, SUMO, and SimTraffic are relatively easy whereas AIMSUN and ARCHISIM are slightly advanced. In terms of coding flexibility of various infrastructure elements, VISSIM, Paramics, AIMSUN, and SUMO are more flexible than others. In addition, majority of them are licensed software, with very limited open source solutions such as SUMO. Thus, SUMO has an edge over others, mainly in terms of ease of understanding and implementation, and cost-benefit.

SUMO is a microscopic inter and multi-modal, time-discrete, and space continuous traffic flow simulation platform. It consists of a microscopic simulator for multi-modal road traffic and a host of applications for preparing simulation input data (network import and modification, traffic import, routing etc.) and for working with simulation outputs. It uses its own format for traffic demand and road networks, which have to be generated or imported from existing sources. The software was developed by DLR institute in Germany and the simulations done are mainly focused on the European traffic conditions, which is homogeneous and lane disciplined in nature. The default simulation parameters that are used under such traffic conditions may not be suitable for lane-less and heterogeneous traffic conditions such as the one considered in the present study. Under such heterogeneous and lane-less traffic conditions, a variety of motorized vehicles such as buses, passenger cars, auto rickshaws, two-wheelers, and trucks, move along with non-motorized vehicles such as animal-drawn carts and bicycles, sharing the same road space without any segregation for the various vehicle types. This will lead to a high level of uncertainties and variability in the traffic characteristics. Another aspect of this traffic is the lack of lane marking and lane discipline, leading to unique behaviour at mid-blocks and near intersections while queuing, making the case for conducting an elaborate calibration in the present work.

2 Literature Review

Any model created in simulation models needs to be calibrated to represent the field conditions efficiently. Calibration is the process of refining the input parameters of the model to accurately replicate the observed traffic conditions. In calibration, the parameters of the model are adjusted in such a way that the outputs are similar to the observed data (Manjunatha et al. 2013; Ge and Menendez 2012; Punzo and Ciuffo 2009). During calibration, the default values of each parameter are changed till the error between the simulated measure and actual are minimized. Before calibration is done, it is important to identify the parameters that affect the output of the model. SUMO has many parameters which affect its driving behaviour. Calibrating SUMO for all these parameters may not be necessary as all factors may not affect the driving behaviour of a particular model in a significant way. Park and Qi (2005) developed a methodology to calibrate micro simulation models for isolated intersections using VISSIM and CORSIM. They used ANOVA test to find the sensitive parameters from a set of eight parameters that were to be calibrated.

Limited studies were reported on calibrating VISSIM for Indian traffic conditions. Mathew and Radhakrishnan (2010) conducted a study to calibrate VISSIM that can represent heterogeneous traffic. A sensitivity analysis was conducted to identify significant parameters by increasing each parameter values by 10% while keeping other parameter values constant. Then, delay found using default parameters was compared with simulated delay from the model. Further, the significant parameter was identified based on the difference in delay with the parameter. If changing the default parameter values affects the delay, then it was considered as a significant parameter and those parameters were chosen for calibration. Anand et al. (2014) calibrated VISSIM model for Indian traffic conditions considering a study stretch from Chennai. They adjusted the parameter values manually to reduce the error between field and simulated data simultaneously, ensuring realistic behaviour of traffic. Siddarth and Ramadurai (2013) proposed a method to calibrate VISSIM model for an intersection in Chennai using Visual C++ COM interface of VISSIM. Elementary Effects (EE) was used to screen important parameters based on one at a time approach. In this method, at each step the value of one parameter is decreased or increased keeping all other values constant. Then, Genetic Algorithm (GA) was used to find the optimal combination of sensitive parameters during calibration.

Although researchers calibrated VISSIM to represent the Indian traffic (Anand et al. 2014; Siddarth and Ramadurai 2013; Manjunatha et al. 2013; Seelam et al. 2017), SUMO has its own advantages over VISSIM. For example, (i) SUMO can use various car-following models in the urban areas whereas VISSIM restricts to Weidman model, (ii) In SUMO, each lane can be subdivided into favourable number of strips, making the number of vehicles shared in a lane known, which is not possible in VISSIM. This is especially important to address the parallel and lane-less movement, if traffic modification in the form values of the parameters and choice of models for car-following and lane-changing is required. SiMTraM is such a modification made in SUMO, where every lane is divided into number of strips and each vehicle can occupy one or more number of strips (Patel et al. 2016; Mathew and Bajpai 2012). Since same concept is incorporated in SUMO as sublane model, the present study used the Sublane model. Other variables that can be changed to match Indian traffic are spacing (both along the road and lateral), speed and acceleration rates, and the lane structure. This paper reports calibration of these variables in SUMO and a performance comparison with an already calibrated VISSIM. The methodology followed for this is discussed next.

3 Methodology

The main objectives of the present study are: (i) conduct analysis to find significant parameters of SUMO and (ii) to calibrate them and evaluate the performance. As a first step, selected network was developed in SUMO and detectors were placed to collect required output such as speed. Next, a sensitivity analysis was carried out to identify the significant parameters that can affect selected outputs in a significant way. In the third step, a GA model was developed to obtain the optimum values of each significant parameter. The performance of the calibrated model was checked using speed as a measurement of effectiveness. For this, speeds obtained from the calibrated SUMO model were compared with speeds obtained from VISSIM model that was already calibrated for the study site. A flow chart showing these major steps is shown in Fig. 1 and the steps are detailed in the following sections.

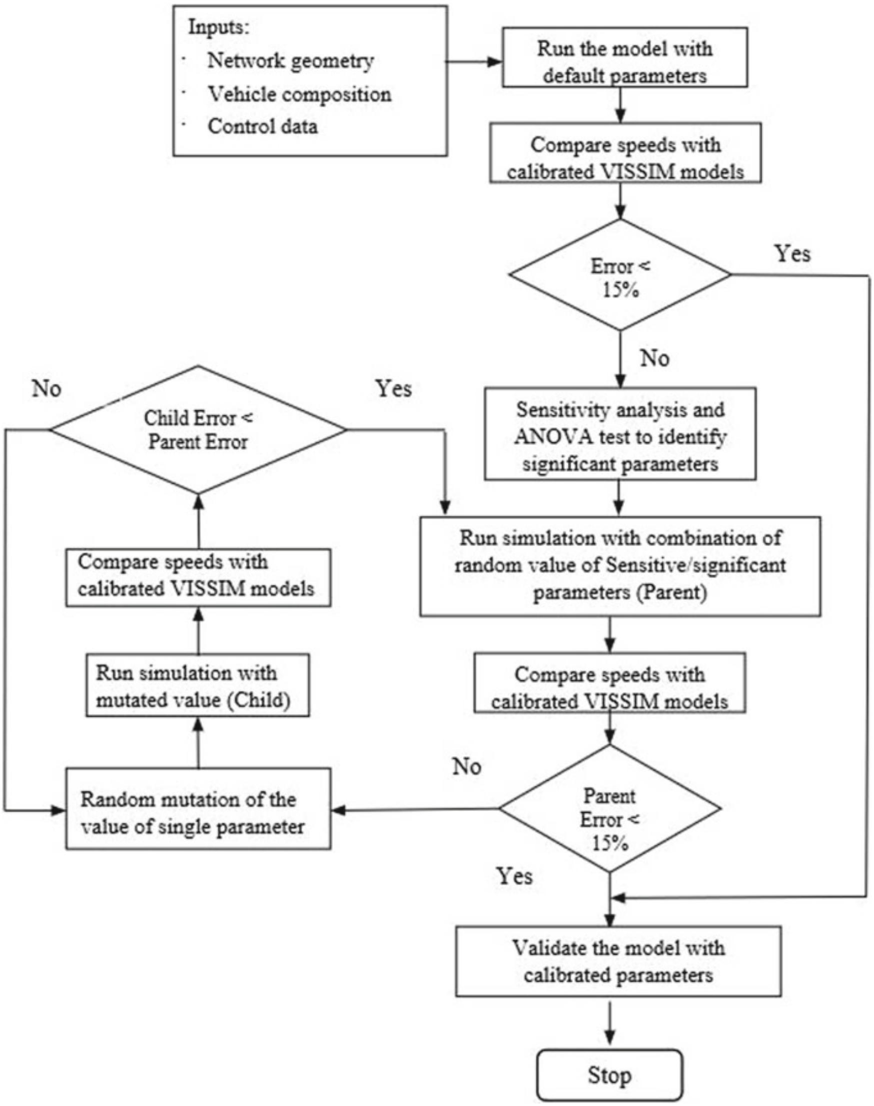


Fig. 1 Flowchart representing the proposed methodology

3.1 Data Collection

For the calibration, data were collected from a selected study stretch that can represent typical Indian traffic conditions. The selected study stretch was a 1.73 km section, starting from Madhya Kailash intersection and ending with Tidel Park intersection in Rajiv Gandhi Salai in the city of Chennai, India. It is a six lane urban arterial road

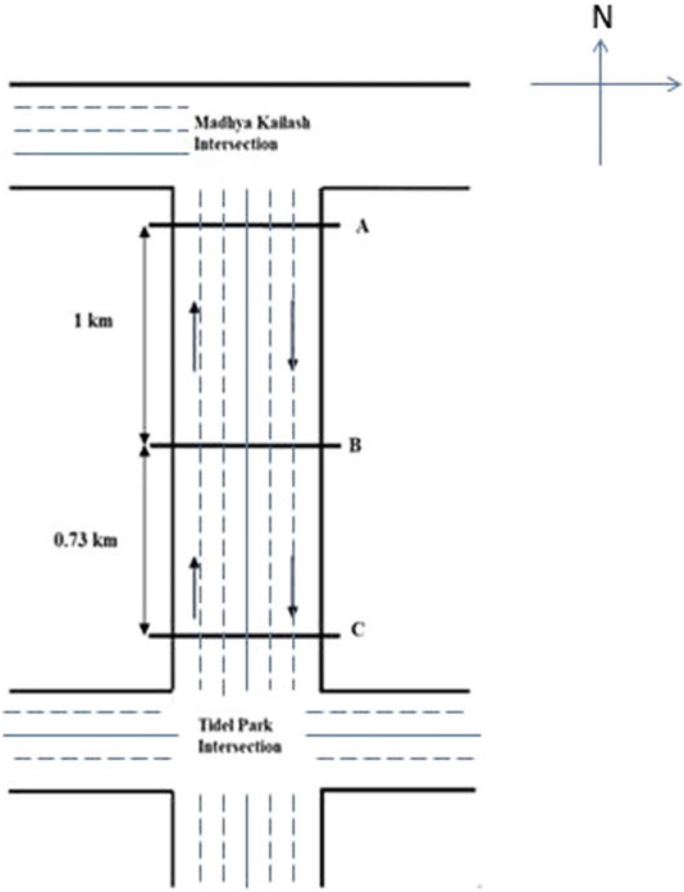


Fig. 2 Sketch showing the selected study stretch

having three lanes in each direction. The study considered only three lanes each of width 3.5 m in the south direction with three foot over bridges, designated A, B, and C as shown in Fig. 2. Flow data were collected from recorded videos for a period of one hour on a typical weekday. Data included classified counts of vehicles for every 1-min aggregation interval. The average flow (aggregated at 1-min intervals) was observed to be 4328 vehicles per hour with composition as presented in Table 1.

3.2 Network Generation in SUMO

The following inputs are required to calibrate SUMO.

Table 1 Vehicle composition in the study site

Type	Composition (%)
Bike	48.5
Auto	5.9
Car	33.6
LCV	7.1
Bus	4.9

Table 2 Vehicle characteristics used

Type	Length (m)	Width (m)	Acceleration		Deceleration	
			Min	Max	Min	Max
Bike	1.8	0.6	1.0	3.0	1.2	2.7
Auto	2.6	1.4	0.4	1.4	0.5	2.0
Car	4	1.6	1.0	2.5	0.7	2.2
LCV	5	1.9	1.0	2.5	0.7	2.2
Bus	10.3	2.5	0.8	1.8	0.8	2.3

1. **Network geometry**: This module includes characteristics of network, such as number and width of lanes (three lanes of each 3.5 m), and intersection geometry.
2. **Traffic data**: This includes extracted traffic flow and vehicle composition data that were used as inputs to SUMO. Outputs generated from existing calibrated VISSIM model were used to validate and calibrate the SUMO model.
3. **Vehicle characteristics**: This includes desired speed of vehicles, vehicle dimensions, deceleration and acceleration data rates as functions of speed. Standard acceleration and deceleration values for each mode were obtained from various motor companies specifications. Desired speed for each class of vehicle was assumed to be the speed limit that is imposed in the selected stretch and standard dimensions of vehicles as shown in Table 2 (IRC: 3-1983) were used.
4. **Control data**: Intersections that are considered in the study are controlled by fixed time signal with a cycle length of 180 s. At times, they will also be maintained by local traffic police by adjusting the cycle length to meet the real-time demand. On the day of data collection, it was observed that the cycle length varied from 150 to 300 s and the same were used in simulation.

3.3 SUMO Parameters

SUMO has several parameters that can be changed during calibration, and are listed below.

1. **Car-following model parameters**: Car-following determines the speed of a following vehicle in relation to the vehicle ahead of it (leader). These parameters include speed, and acceleration of leading vehicle, reaction time of the driver of the following vehicle, and driver characteristics.
2. **Lane-changing parameters**: Lane-changing determines speed adjustments related to lane-changing and lane choice on multi-lane roads. Associated parameters include speed gain during manoeuvre (speed gain probability, keep right probability, average waiting time), and front and rear gaps on the target lane.
3. **Sublane model parameters**: Division of lanes into sublanes try to address the lane-less movement of Indian traffic. Parameters?

Table 3 lists these parameters with associated range. However, all these parameters of SUMO shown in Table 3 may not affect the output of the model in a significant way. Therefore analysis was conducted to identify the significant parameters as discussed next.

To start with, a set of 100 samples were generated for each parameter within its range and in the next step, an ANOVA test was conducted to identify the significance of that parameter on the model. Being more conservative by evaluating with a least amount of type 1 error (NIST 2018), one-way ANOVA was conducted with a significance value (α) of 0.05 in this study. Speed values obtained from simulation were compared for varying values of the parameter, keeping all other parameters to default value. After conducting the test, parameters that had *p*-values less than 0.05 were regarded as significant. Table 4 lists the tested parameters and its corresponding *p*-values obtained from ANOVA test and Table 5 shows the parameters that were found to be significant for the present study i.e. the parameters that had *p*-value less than 0.05.

Once the significant parameters were identified, calibration of them to suit the traffic condition being studied is the next step and is discussed in the next section.

3.4 Calibration Using Trial and Error

To start with, a trial and error method was used to calibrate the parameters. Considering the difficulty of implementation procedure, the most significant parameters i.e. lateral resolution and speed were only changed around corresponding default values. On a whole, a total of 40 simulations were conducted. The results obtained from all these simulations were compared against the speed values that were obtained from the calibrated VISSIM model (Siddarth and Ramadurai 2013; Anand et al. 2014) and errors were quantified in terms of Mean Absolute Percentage Error (MAPE). Table 6 shows the calibrated values obtained from trial and error approach that induced least possible error. Figure 3 shows a sample comparison of speeds obtained using SUMO with calibrated values (shown in Table 6) and its comparison with VISSIM speed.

From Fig. 3, it can be observed that the simulated values obtained from SUMO are matching with VISSIM to a reasonable extent. Corresponding MAPE was found

Table 3 Parameters of SUMO

Parameter	Value	
	Minimum	Maximum
Driver imperfection		
Bike	0	1
Bus	0	1
Car	0	1
LMV	0	1
Auto	0	1
Minimum gap at jam conditions (m)		
Bike	0	5
Bus	0	5
Car	0	5
LMV	0	5
Auto	0	5
Speed dev from maximum lane		
Bike	0	0.5
Bus	0	0.5
Car	0	0.5
LMV	0	0.5
Auto	0	0.5
Lateral resolution—strip width (m)	0	3.6
Desired acceleration at 0 kmph		
Bike	2.0	3.0
Bus	0.8	1.8
Car	1.0	2.5
LMV	1.0	2
Auto	0.4	1.4
Desired deceleration at 60 kmph		
Bike	1.2	2.7
Bus	0.8	2.3
Car	0.7	2.2
LMV	0.7	2.2
Auto	0.5	2.0

to be close to 44.89%, which may not be sufficiently accurate for simulation studies. This is may be due to the large number of parameters that are varying at different ranges that can affect the accuracy of the SUMO model. In order to improve the accuracy, a possible better alternative that can handle multiple parameters at a time

Table 4 Results obtained from ANOVA test

Parameter	Value		ANOVA *P*-value
	Minimum	Maximum	
Driver imperfection			
Bike	0	1	0.834
Bus	0	1	0.993
Car	0	1	0.486
LMV	0	1	0.999
Auto	0	1	0.999
Minimum gap at jam conditions (m)			
Bike	0	5	**5.02E−26**
Bus	0	5	0.3857
Car	0	5	**1.03E−27**
LMV	0	5	0.99
Auto	0	5	0.99
Speed deviation from desired speed			
Bike	0	0.5	1.79E−01
Bus	0	0.5	0.999
Car	0	0.5	0.0002
LMV	0	0.5	0.239
Auto	0	0.5	0.708
Lateral resolution—strip width (m)	0	3.6	**1.14E−159**
Desired acceleration at 0 kmph			
Bike	2.0	3.0	**9.03E−38**
Bus	0.8	1.8	0.993
Car	1.0	2.5	**1.22E−07**
LMV	1.0	2	0.981
Auto	0.4	1.4	**9.73E−06**
Desired deceleration at 60 kmph			
Bike	1.2	2.7	9.97E−01
Bus	0.8	2.3	**1.04E−08**
Car	0.7	2.2	0.504
LMV	0.7	2.2	**9.11E−07**
Auto	0.5	2.0	**2.31E−05**
Desired speed (m/s)			
Bike	12.5	18	**0.023263266**
Car	13.89	20.83	**1.09298E−25**
LMV	13.89	20.83	**1.09298E−25**

Table 5 Significant parameters identified from ANOVA

S. No.	Parameter	Minimum	Maximum	ANOVA *P*-value
1	Bike acceleration (m/s^2)	2	3	90.02926E−39
2	Car acceleration (m/s^2)	1	2	12.1586E−08
3	Auto acceleration (m/s^2)	0.4	1.4	97.3212E−07
4	Bus deceleration (m/s^2)	0.8	2.3	10.3679E−09
5	LMV deceleration (m/s^2)	0.7	2.2	91.0602E−08
6	Auto deceleration (m/s^2)	0.5	2	23.0981E−06
7	Bike desired speed (m/s)	12.5	18	0.023263266
8	Car desired speed (m/s)	13.89	20.83	10.9298E−26
9	LMV desired speed (m/s)	13.89	20.83	10.9298E−26
10	Car speed dev (m/s)	0	0.5	0.000256565
11	Car mingap (m)	0	5	12.7699E−28
12	Bike mingap (m)	0	5	50.1514E−27
13	Lateral resolution (m)	0.6	3.6	11.364E−160

Table 6 Calibrated parameters obtained from trial and error approach

S. No.	Parameters	Calibrated value
1	Bike acceleration (m/s^2)	2.92
2	Car acceleration (m/s^2)	1.89
3	Auto acceleration (m/s^2)	1.39
4	Bus deceleration (m/s^2)	2.06
5	LMV deceleration (m/s^2)	2.01
6	Auto deceleration (m/s^2)	1.57
7	Bike desired speed (m/s)	12.82
8	Car desired speed (m/s)	15.17
9	LMV desired speed (m/s)	15.06
10	Car speed dev (m/s)	0.24
11	Car mingap (m)	2.88
12	Bike mingap (m)	2.06
13	Lateral resolution (m)	3.53

unlike trial and error approach was identified as Genetic Algorithm (GA). The details of GA and calibration procedure is detailed in next section.

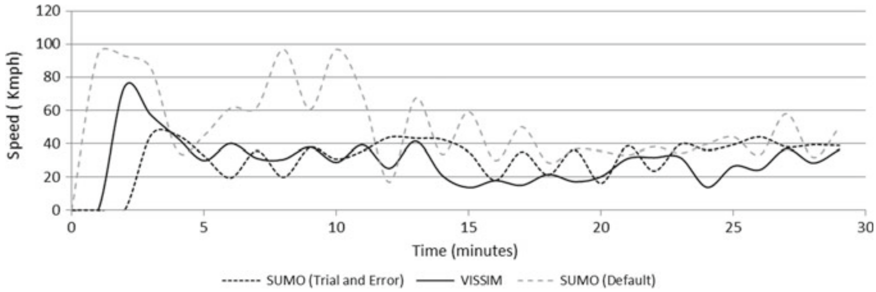


Fig. 3 Comparison of speeds obtained from calibrated SUMO (trial and error approach)

3.5 Calibration Using Genetic Algorithm

In the next phase, Genetic Algorithm (GA) was used for calibrating the parameters. This was done by varying only the values of identified sensitive parameters (shown in Table 5) while keeping other parameters values constant (default values). GA is a random search and optimization technique. In the present study, it was used to generate random sets for parameters within specified bounds and then calibration code was run till it finds the least Mean Absolute Percentage Error (MAPE) value between the simulated and actual measure.

The main difference between GA and other classic search algorithms is in the way in which the algorithm picks points. Classic search algorithms pick points more randomly and iterates until certain conditions are satisfied while GA first picks randomly and then mutates over that to generate a new set of values. In GA, a random parent would be selected from an initial population and given a fitness value based on the fitness function. This parent undergoes mutation and cross-over to form the child whose fitness value is found and compared with that of the parent. If the fitness value of child is more than parent, the child will become parent and the whole process repeats. If lower, the parent undergoes mutation again until the parents' fitness value is less than the child's fitness value (Whitley 1994).

In this study, a population of 100 random values for each sensitive parameter (Table 5) was created. Initial parent list of values were generated by taking a random value for each parameter. First, a simulation was run with this parent list and 1-min average speeds were measured to cover a distance of 100 m from Tidel park intersection. These values were then compared with values obtained using VISSIM and the errors, reported in terms of MAPE, is further considered as fitness value. Once the simulation is completed with the parent list, mutation occurs by changing a value of random sensitive parameter, which is called as child list. Further, the simulation will continue with the child list and the fitness value obtained from child list will be compared with the fitness value of parent list. If the fitness value of the child list is less than parent list, child will become the parent and this process will continue until the parent fitness value is less than the threshold selected, which is 3%. Finally, the parent list that satisfies the condition and the combination of such

Table 7 Calibrated values obtained from GA

S. No.	Significant parameters for GA (in SI Units)	Calibrated values
1	Bike acceleration (m/s^2)	2.46
2	Car acceleration (m/s^2)	1.49
3	Auto acceleration (m/s^2)	1.01
4	Bus deceleration (m/s^2)	2.08
5	LMV deceleration (m/s^2)	1.49
6	Auto deceleration (m/s^2)	0.86
7	Bike desired speed (m/s)	16.16
8	Car desired speed (m/s)	17.24
9	LMV desired speed (m/s)	17.24
10	Car speed dev (m/s)	0.059
11	Car mingap (m)	0.91
12	Bike mingap (m)	3.77
13	Lateral resolution (m)	3.37

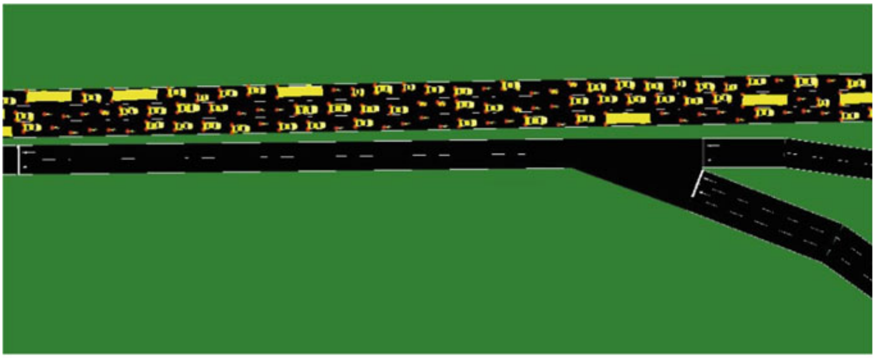
parameters was considered as calibrated parameters of the model and are listed in Table 7. Figure 4 shows the simulation run in SUMO before and after calibration showing the behaviour of lane-less and mixed traffic conditions. From Fig. 4, it can be observed that various types of vehicles are moving without following any lane discipline after calibration.

Figure 5 shows the comparison of speeds obtained from GA calibrated SUMO model and VISSIM model. From Fig. 5, it can be observed that the speeds obtained from SUMO model are closely following with the speeds obtained VISSIM model and the corresponding MAPE was 19.76%.

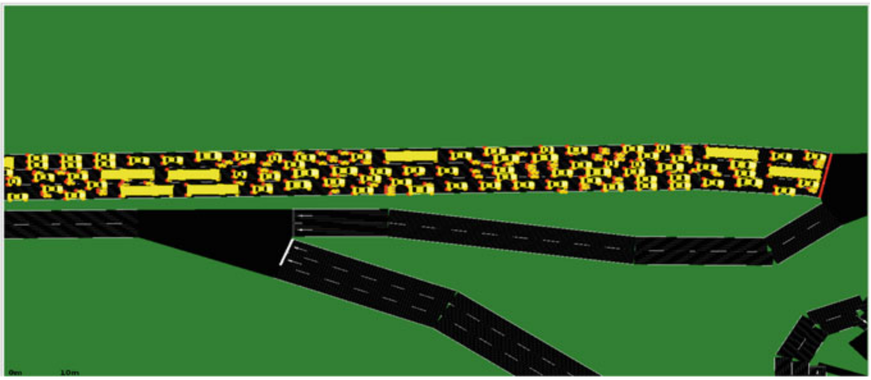
Finally, a comparison was made in terms of speed obtained from Trial and Error approach and GA approach with VISSIM and the corresponding results are presented in Fig. 6. From Fig. 6, it can be observed that the GA was able to produce speed values that are more comparable to the actual than trial and error. Error for Trial and Error approach was 44.89% whereas GA yielded around 20% MAPE showing the efficacy of the GA approach.

4 Summary and Conclusions

The main aim of this study is to understand SUMO software, which was originally developed for European traffic conditions, and to modify the same to represent the typical traffic conditions in India. To make it suitable for mixed traffic conditions, modification in the form values of the parameters and choice of models for car-following and lane-changing may be required. Other possible modifications may include spacing (both along the road and lateral), speed and acceleration rates, and



(a) Before calibration



(b) After calibration

Fig. 4 Traffic behaviour of SUMO model before calibration and after calibration

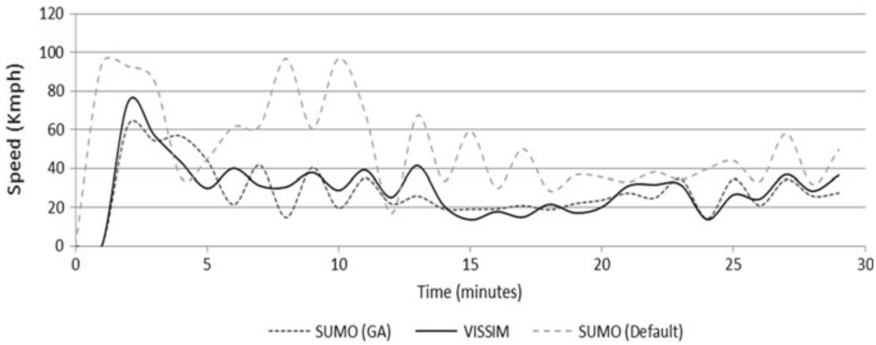


Fig. 5 Comparison of speeds obtained from calibrated SUMO (GA approach)

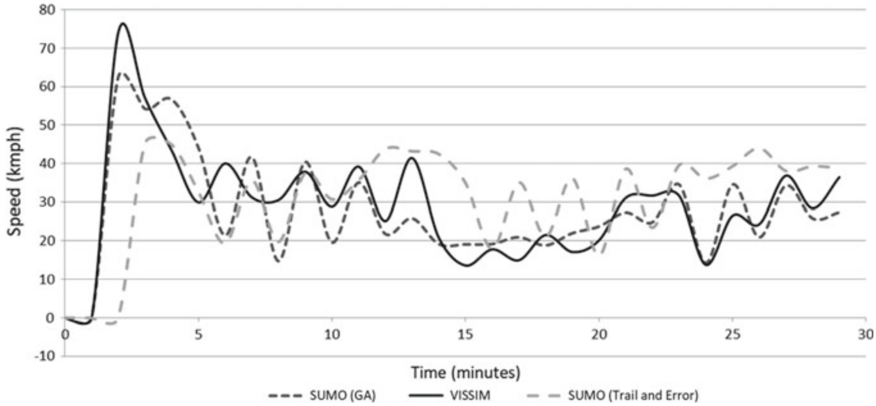


Fig. 6 Comparison of speeds obtained from calibrated SUMO model by GA and trail and error approach with VISSIM model

the lane structure. The present study mainly concentrated on identifying suitable values of the general parameters of SUMO. To start with, required road network was created in SUMO and corresponding inputs (volume, control, and vehicle proportions) collected from field were fed in. During calibration process, significant parameters that can affect the simulation model were identified using ANOVA test. Results showed that 14 parameters such as desired speed of car, desired speed of bike, lateral resolutions, etc. are significant in calibrating such model. Further, to identify the optimum values for these parameters, Trail and Error and Genetic Algorithm methods were used. The output data of average speed obtained from calibrated SUMO model was compared with the speeds obtained from an already calibrated VISSIM model in terms of MAPE. Results showed that calibrated SUMO speeds to be comparable with the speeds obtained from calibrated VISSIM.

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Data Collection in Countries with Extreme Vehicle Heterogeneity and Weak Lane Disciplined Traffic

Bhupali Dutta and Vinod Vasudevan

Abstract Understanding driver behavior is extremely important for the design and analysis of any transportation infrastructure. Several methodologies exist to collect information on driver behavior. However, most of these apply to homogeneous traffic with lane discipline. In India, like in most of the low- and middle-income countries, the traffic is highly heterogeneous and exhibits poor lane discipline. In this case, the vehicles interact not only longitudinally, but also laterally. Hence, these traditional methodologies may not work in such a scenario. The objective of this paper is to present a data collection method which will help to collect information about individual vehicles in highly heterogeneous traffic with poor lane discipline. Instrumented vehicles help to observe individual driver behavior accurately and precisely. Although such vehicles are present in various universities in the USA (such as University of Michigan, Southampton, Texas A&M University, to name a few), their purpose of such vehicles is different. However, the major challenges are associated with data processing and extraction. Since these sensors give large data, its processing is not easy, and it offers challenges. This paper discusses the opportunity such a vehicle offers to understand driving conditions and the challenges the researchers might face. This paper also presents some simple applications of the data.

Keywords Driver behavior · Data collection · Instrumented vehicle · Vehicle heterogeneity · Lane discipline · Accuracy

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1 Introduction

Data collection, especially on driver behavior, is an integral part of understanding driver responses under various scenarios. Information on driver behavior is the fundamental requirement to develop vehicle-following models, in the design and analysis of transportation infrastructure and road safety analysis. There are various traditional methods of data collection on driver behavior. These include moving observer method, videographic method, and GPS-based methods. However, most of these apply to homogeneous traffic with proper lane discipline. In India, like in most of the low- and middle-income countries, the traffic is highly heterogeneous and exhibits poor lane discipline. In this case, there exists both longitudinal and lateral interaction of traffic. Hence, accuracy on lateral spacing is equally important when compared to that of longitudinal spacing. Drivers are more sensitive to changes in spacing along the lateral direction rather than along the longitudinal direction. This is because the width of the road is limited and minor error in judgement in lateral spacing might lead to collisions and accidents. The traditional methods cannot give accurate information on the lateral spacing between the objects and vehicles. Therefore, innovative methods of data collection need to be explored to gather continuous data on lateral and longitudinal interactions of the vehicle with the driving environment. These environments are made up of both static (like roadway infrastructure, signs, vegetation, built environment) and dynamic (such as other moving vehicles and pedestrians) objects. This paper discusses how these data could be captured efficiently and related challenges in data extraction and processing.

2 A Brief Overview on Existing Methods of Data Collection

In order to study traffic, there are two broad classifications of studies, macroscopic and microscopic. Macroscopic studies consider traffic stream as a whole and try to understand its behavior, whereas microscopic studies try to understand the responses of an individual driver under various driving environments. Due to these fundamental differences, the data requirements and hence the data collection procedures are different. Data collection practices for each of these studies are discussed in this section. Also, a brief literature review of data collection practices on microscopic studies evolved over the years is explored in this section.

2.1 Data Collection for Macroscopic Studies

In order to analyze traffic macroscopically, data on stream characteristics, such as flow, speed, and vehicle type, are captured for the whole stream. Typically, these are captured only for small road sections. Such data provide much useful information

Table 1 Summary of data collection methods for macroscopic studies

Data collection technology	Type of data collected	Limitations
Magnetic loop detectors	Vehicle presence, volume, occupancy, speed	Limited lane coverage, short life, vulnerable to weather, temperature, and traffic conditions
Infrared sensors	Vehicle presence, vehicle class, speed	Limited lane coverage, vulnerable to bad weather
Micro-wave radar	Speed, vehicle count, vehicle class, lane assignment	Cannot capture longer road sections
Ultrasonic sensors	Vehicle count, vehicle class, speed	Vulnerable to bad weather and limited lane coverage
Videography method	Vehicle counts, speed, flow, and vehicle class	Sensitive to meteorological conditions and cannot capture longer road sections

on traffic stream; however, the data collection methods cannot capture minute details on driver–vehicle interaction with the surrounding traffic. Some of the traditional macroscopic methods of data collection along with their limitations are mentioned in Table 1 (Leduc 2008; Rouphail et al. 2018).

2.2 Data Collection for Microscopic Studies

In the microscopic method, individual driver behavior is captured in different traffic situations. Therefore, the details of the data collected and hence the technology are different. For microscopic studies, driver behavior needs to be captured under various driving conditions. Hence, the data collection is more challenging, and the details required are more demanding. The commonly used practices range from videography-based data collection procedures to driver simulators to instrumented vehicles. Table 2 describes some of the data collection technologies used for microscopic studies along with their limitations. However, as the number of sensors (instruments) used for data collection increases, data processing and storage become more complex (Leduc 2008; Rouphail et al. 2018).

As discussed in Table 2, each of the approaches to collect data for microscopic studies has certain limitations. An on-road observational survey with the help of videography captures driver behavior only for a stretch of road. It cannot capture driver behavior for longer duration and longer stretches. The choice of study location is determined primarily by the availability of high vantage point such as over-bridge in its vicinity. Besides, data extraction is labor intensive with a lower level of accuracy. In-vehicle devices like GPS and Bluetooth give information on travel time, speed, and route length. Such devices are cost-effective. Some of the applications of these in-vehicle devices are in origin–destination studies and trip length analysis. However,

Table 2 Summary of data collection methods for microscopic studies

Data collection technology	Type of data collected	Limitations
On-road observation method (videography)	Vehicle counts, speed, flow, and vehicle class	Vulnerable to bad weather conditions, limited temporal and spatial resolution, limited by proximity to a high vantage point (tall building, toll plaza), labor-intensive methods of data extraction, low accuracy
In-vehicle devices (GPS and Bluetooth)	Travel time, speed, route length, origin and destination points	Low travel time accuracy, low precision in GPS localization, vulnerable to loss in connection and connectivity error
Driving simulator	Driver behavior: risk-taking or risk-averse, driver distraction, driver fatigue, vehicle acceleration–deceleration, steering angle behavior	Biasdness in driver behavior, vulnerable to over-speeding by drivers, limited true replication of traffic scenarios of real roads, expensive
Instrumented vehicle (in developing countries: sensors used are V-Box, video cameras, and ultrasonic sensor)	Vehicle speed, acceleration, angular velocity, lateral gap, type of objects in the vicinity, route length	No information on longitudinal spacing, limited lateral gap information, and active data collection

accurate information may not be captured if there is either a loss in connection or connectivity error. Recently, driving simulators are used for data collection. The question about this approach, however, remains whether drivers behave the same way in a simulator as they do on the road because the actual traffic situations cannot be reproduced in the laboratory.

The traditional car-following models are developed with the help of data collected through videography for limited sections and duration. These models are calibrated and validated through simulation due to lack of microscopic data at that time. In 1950–1960, instrumented vehicles were developed for the first time to study traffic flow microscopically (Chandler et al. 1958; Herman and Potts 1959; Michaels and Cozan 1963). However, these studies had a limited number of observations and the data were not shared with the research community. In 2002, the Next Generation Simulation (NGSIM) program was started by the US Department of Transportation. Under this program, the first set of in situ microscopic driver–vehicle datasets was developed. In this dataset, many instrumented vehicles were used to collect information on driver and vehicle on two freeway sections for approximately 45 min and 0.5 miles. The NGSIM data have helped in many recent advances in microscopic traffic flow studies. However, one of the limitations of NGSIM data is that the data collection was limited to freeway sections only.

Recent advances in technology make instrumented vehicles feasible to collect detailed data to assess driving behavior. In such an experimental setup, a vehicle is equipped with various sensors like distance measuring sensor, speed measuring sensor, video cameras, etc. and drivers are asked to drive naturally. Instrumented vehicles help to observe instantaneous vehicle information and driver behavior accurately for longer road sections, for longer duration, and in different driving scenarios. Owing to the amount and extent of data instrumented vehicles can collect, in the recent past, institutions such as the University of Michigan, Southampton, Texas A&M University have developed such vehicles. The instrumented vehicles developed by these institutions are used in the study of road safety analysis (Dingus et al. 2006), insight into driver workload in different traffic scenarios (Young et al. 2017), differences in driving styles based on driver demographics (Ranjitkar et al. 2004). These studies are primarily conducted in predominantly homogeneous and lane-based traffic conditions.

In developing countries, the traffic is highly heterogeneous with weak lane discipline. In the recent past, studies are conducted to understand vehicle behavior in such traffic conditions (Bangarraju et al. 2016; Gunay 2003, 2007; Mahapatra and Maurya 2015; Mahapatra et al. 2016). Unlike under homogeneous traffic conditions, the vehicular interaction is both longitudinal and lateral. Although models are developed to take care of vehicle heterogeneity and weak lane discipline (Gundaliya et al. 2008; Mathew et al. 2015; Ravishankar and Mathew 2011), the models are validated through simulation. Besides, the data used in these studies are obtained through videography.

As mentioned previously, traffic on Indian roads is characterized by a wide variety of vehicular mix ranging from two-wheelers to passenger cars to light commercial vehicles to trucks and buses sharing the same road space. The vehicles differ in both static (vehicle size) and dynamic (vehicle operation) properties. Vehicles often do not follow lane discipline and occupy any suitable space on the road depending on their gap requirement and gap availability. Besides, different types of vehicles drive at different speeds. Hence, in such traffic conditions, lane change maneuvers occur frequently. In a given traffic scenario, to understand how vehicles behave in disorderly and heterogeneous traffic, it is crucial to acquire data on instantaneous vehicle trajectory, available gaps, speed, and traffic situation in the vicinity. Instrumented vehicle can be used in either active data collection (instrumented vehicle act as the study vehicle) or passive data collection (nearby vehicles act as study vehicles). To avoid observational bias and to include more observations, passive data collection is needed. Instrumented vehicles help to acquire instantaneous vehicle (speeds, steering angle, etc.) and surrounding information (gap, traffic density, etc.) in different traffic scenarios. In India, instrumented vehicle study has been undertaken to get insight into vehicle response in different traffic conditions (Mahapatra and Maurya 2015; Budhakar and Maurya 2017). The various sensors used in the vehicle are GPS-IMU units, video cameras, and ultrasonic sensors. There are certain limitations of the sensors used. The sensors cannot give some of the important information required in heterogeneous and weak lane disciplined traffic such as trajectory information of the vehicles, gaps available on the road, and other vehicle speeds. Although the ultrasonic sensor was used to get data on lateral spacing, the sensor cannot give information on the longitudinal spacing. The lateral extent of the ultrasonic sensor is limited, with a poor level of accuracy. Besides, only active data collection is possible. Hence, information of only selected drivers is possible.

3 Objective

There is a vast difference in the vehicle operating characteristics under homogeneous lane-based traffic and heterogeneous weak lane disciplined traffic. Examination of the available literature shows the need to develop a data collection method which will capture on-road vehicle behavior in heterogeneous disorderly traffic. Each of the existing methods exhibits certain limitations which makes efficient data collection of continuous streams difficult. To gather accurate and instantaneous driver behavior in different traffic scenarios, an instrumented vehicle is an alternative. Hence, the objective of this paper is to present the details of an instrumented vehicle which will help collect data on various vehicle interactions instantaneously and accurately in heterogeneous traffic with weak lane discipline.

4 Development of the Instrumented Vehicle

As has already been mentioned, there are several advantages of instrumented vehicle studies. Recent advances in technology make instrumented vehicle studies feasible to collect detailed data to assess driving behavior. The objective of this paper is to present a data collection method which will help to collect data of individual drivers under various driving environments. The basic requirements of the instrumented vehicle are as follows: The instruments in the vehicle should be able to (1) identify the type of objects, (2) capture distances of these objects, (3) capture speed, (4) capture vehicle behavior (such as accelerating, decelerating, and changing direction), and (5) capture location-specific information. Keeping these requirements in mind, instruments need to be identified.

There are a few options to address the first couple of requirements. Traditionally, radar is used. However, one of the major drawbacks of the radar is its accuracy and object detection. While radar provides a reasonable estimate of distances of objects around the radar, identifying them and tracking becomes difficult. Another option is the use of video cameras. Since image processing techniques have improved significantly over the years, it is another option. However, estimating accurate distances is a challenge from video data. Also, videography faces challenges associated with ambient light conditions. Recently, LiDAR is used in autonomous vehicle navigation and object detection. LiDAR stands for light detection and ranging. The sensor emits laser beams to determine the range and objects. The X, Y, and Z coordinates of an object near the LiDAR are calculated from the difference between the time of release and time of return of the laser pulse and the angle at which the laser pulse hits the object. Since LiDAR data are not affected by light conditions and since the distances are directly captured, LiDAR provides a more accurate estimate of the distances than other methods such as the use of radar or videography. Video cameras can be used to verify the objects detected by LiDAR.

GPS has been used extensively to capture locations of vehicles. GPS can also be used to estimate speeds. However, as the location data itself have an error associated, depending on the accuracy of the GPS, estimates of speed and acceleration would amplify these errors. Besides, bad weather condition, dense surrounding landscape, and forested areas can limit the use of GPS further because in such conditions satellites fail to provide location information. On-board diagnostic (OBD) port of the vehicles provides access to a few operating data of vehicles. These also include speed. Since these data are directly captured from vehicles, they are accurate. Speed can also be estimated from LiDAR data by checking the relative position of the instrumented vehicle from a stationary object. The accuracy of speed measurement with OBD is ± 2 kmph.

Location of the vehicle is important to understand the land use and other related information of the neighborhood. GPS can be used to capture this important information also.

The driver attributes that are important in a traffic study are changes in acceleration, and steering wheel angles. Acceleration-related data can be estimated from speed data captured using with either a GPS unit or OBD sensor. Steering wheel angle, which indicates whether the driver changes lane, can be captured using steering angle sensors.

Since each of the sensors is independent, it is important to synchronize them so that the data captured are on the same time scale. This will help to link data from various sources. Hence, data integrator is important. Also, as these sensors require energy to operate, the power supply unit is also essential.

Figure 1a, b show pictures of the sensors and the vehicle, respectively. Most of these sensors listed in this section give data in the required format directly. However, LiDAR is one of the sensors which need some detailed discussion as it does not directly provide the required details. The data need to be post-processed to detect and track objects from point cloud data.

4.1 VLP-16 LiDAR

As discussed previously, due to its advantages like the 3D representation of the surrounding and feasibility of data collection at night, the technology is used in object detection and tracking, control, and navigation for autonomous cars. LiDAR technology helps to capture accurate distance information of the objects in the surrounding. Since it provides 360° field of view (FOV) of the surrounding, objects present laterally can also be detected. One of the low-cost ones is model VLP-16 from Velodyne. The horizontal range of VLP-16 is 100 m, and the vertical FOV is $\pm 15°$. The accuracy level is up to 15–30 cm on the horizontal plane. Data can be captured at a frequency of 5–20 Hz. The LiDAR sensor can be mounted on the roof of the vehicle so that it has an uninterrupted view.

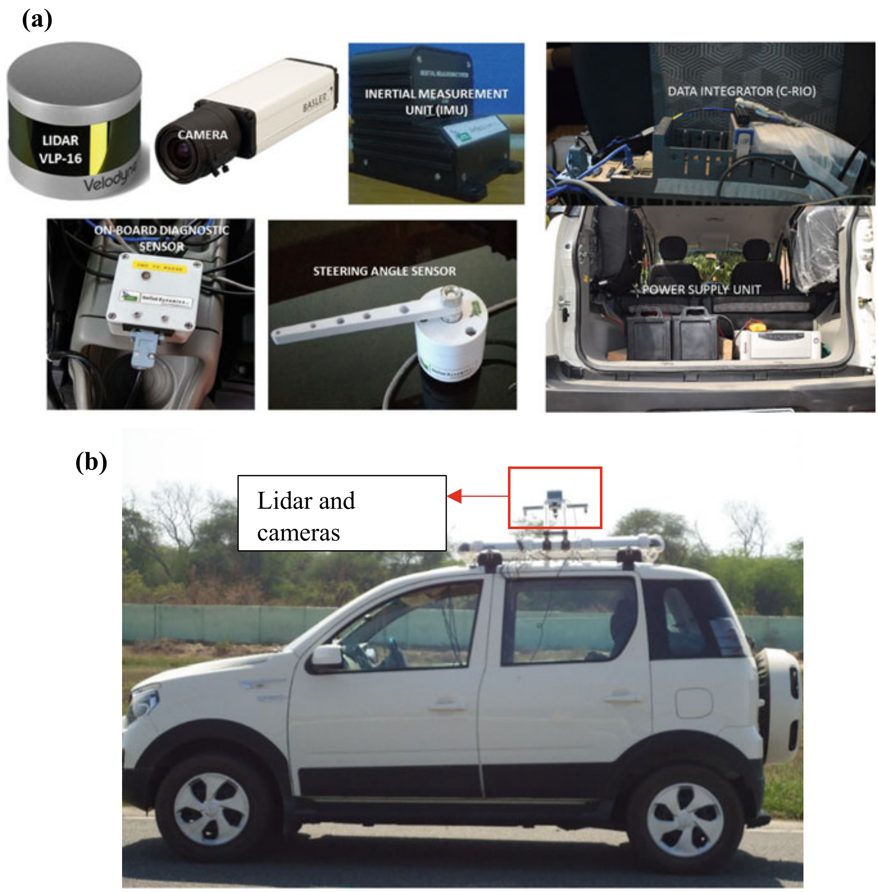

Fig. 1 Instrumented vehicle, available sensors, and power distribution unit. **a** Available sensors, data integrator and power supply unit. **b** Vehicle with the Lidar and video cameras

To test the accuracy of VLP-16 LiDAR, certain experiments were conducted, in static conditions. Two types of objects—board and flex—were placed at different locations on the road (center, left, and right) and *X* and *Y* coordinates of the center of the objects were measured with the help of LiDAR and measuring tape from the LiDAR. Results of static data validation are shown in Table 3. From the table, it can be observed that distance measurement with LiDAR is reasonably accurate. In the dynamic method of data collection, accuracy of LiDAR lies between 5 to 10 cm.

Although LiDAR offers a lot, there are quite a few challenges concerning data processing. It is explained in the following sub-sections.

Table 3 Data validation for the accuracy of VLP-16 LiDAR in the static condition

Object	Location of the object on the road	*Y* coordinate of the center (measured with tape) (m)	*Y* coordinate (measured with LiDAR) (m)	*X* coordinate of the center (measured with tape) (m)	*X* coordinate (measured with LiDAR) (m)	Difference in measurement in *Y* coordinate (m)	Difference in measurement in *X* coordinate (m)
Board	Center	10	10.014	0.63	0.623	−0.014	−0.007
	Left	20	20.036	0.46	0.456	−0.036	−0.004
	Center	30	30.08	0.46	0.452	−0.08	−0.008
	Right	40	40.065	0.63	0.614	−0.065	−0.016
Average difference in *X* and *Y* coordinates						**−0.049**	**−0.009**
Flex	Center	10	10.016	0.45	0.452	0.016	0.002
	Left	20	20.021	0.98	0.985	0.021	0.005
	Right	30	30.029	0.45	0.529	0.029	0.079
	Center	40	40.069	0.45	0.450	0.069	0
Average difference in *X* and *Y* coordinates						**0.034**	**0.022**

4.1.1 Challenges in Data Processing

As discussed before, LiDAR generates 3D point cloud data (PCD) of the objects in the surrounding. GPS gives unique location information of an object/vehicle. Unlike GPS, LiDAR does not generate unique ID for the objects in the surrounding. Since the points in different time frames are not linked, it generates different IDs for the same object in different frames/scans. In one frame, approximately 30,000 points are generated. Tracking the same point/object in different frames becomes challenging. 3D PCD of the surrounding as obtained from one of the frames of the LiDAR is shown in Fig. 2. Figure 3a, b show how the ID of the same corner point of a vehicle differs from one scan to the next. PCD of objects is lost when the sensor moves at high speed. In the case of occlusion, no information can be derived from the target object. This directly affects detecting and tracking performance.

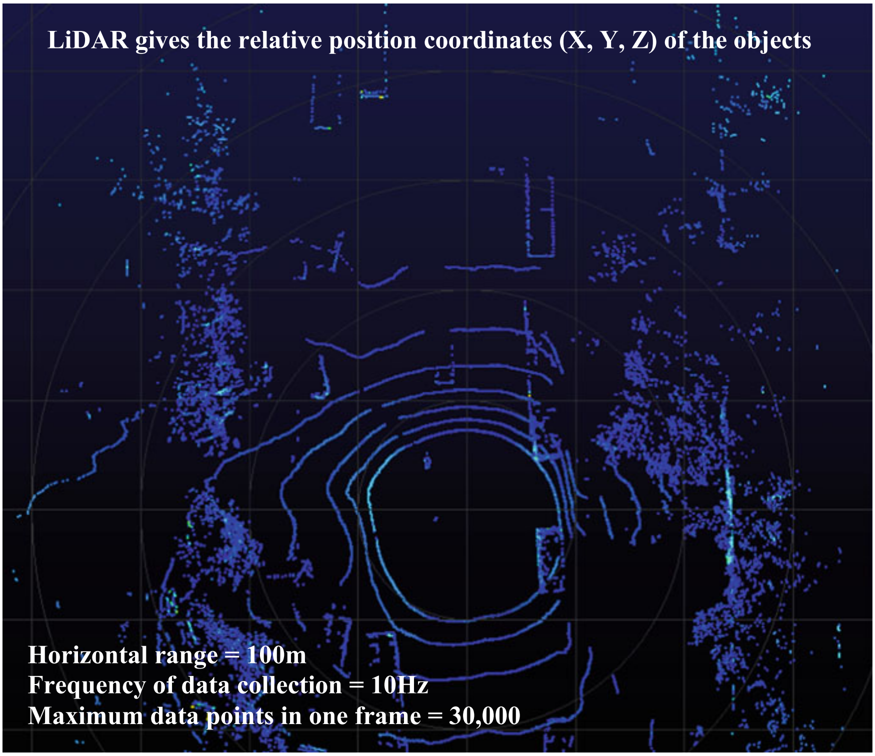

Fig. 2 LiDAR scan data

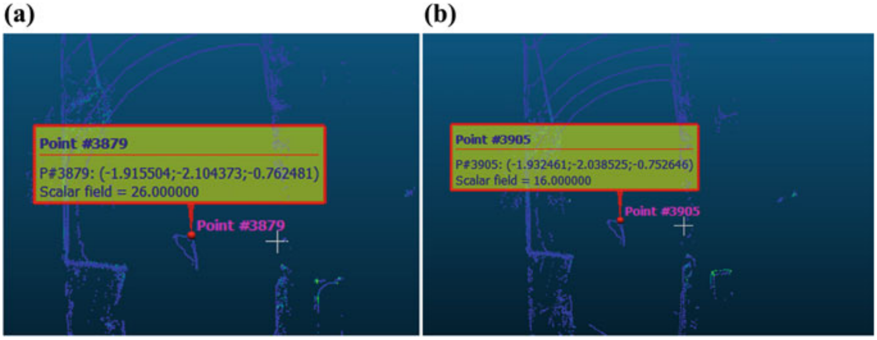

Fig. 3 **a** PCD of one corner of a vehicle at time t_1. **b** PCD of one corner of a vehicle at time t_2

4.1.2 Data Extraction

LiDAR eliminates the positioning errors that arise in image processing; however, object detection and tracking from LiDAR data is a challenge. Different algorithms such as DBSCAN and bounding box concept are available in the literature to track objects detected with the LiDAR sensor (Dewan et al. 2016). Most of the algorithms are based on object tracking using 3D LiDAR technology in conjunction with 2D RGB camera images (Asvadi et al. 2016). Most of the tracking algorithms are applicable to detect objects in a static environment. There are a few algorithms to track objects detected with LiDAR in a dynamic environment also. Such algorithms use Bayesian approach, octree-based approach, etc., to detect objects. These algorithms give very accurate location information of the objects; however, identification of the object type is difficult with these algorithms and the algorithms are computationally exhaustive. As already mentioned, four video cameras are installed on four sides of the vehicle to identify feature types in the surrounding. With the help of these video cameras, objects of interest around the instrumented vehicle can be manually identified. Programs need to be written to detect and track objects from frame to frame. This is one of the major challenges of using LiDAR.

5 Potential Applications of the Instrumented Vehicle

There are several applications of instrumented vehicle. These include studying vehicle behavior under various scenarios including different road categories, different levels of services, overtaking, in the presence of other vehicles, in the presence of non-motorized road users, etc., to name a few. Once data are collected, these information can be used to develop models which describe driver behavior. Figure 4a, b show one LiDAR scan and corresponding camera images in one particular time stamp. Such an instrumented vehicle can also be used to capture the behavior of all road users, including pedestrian.

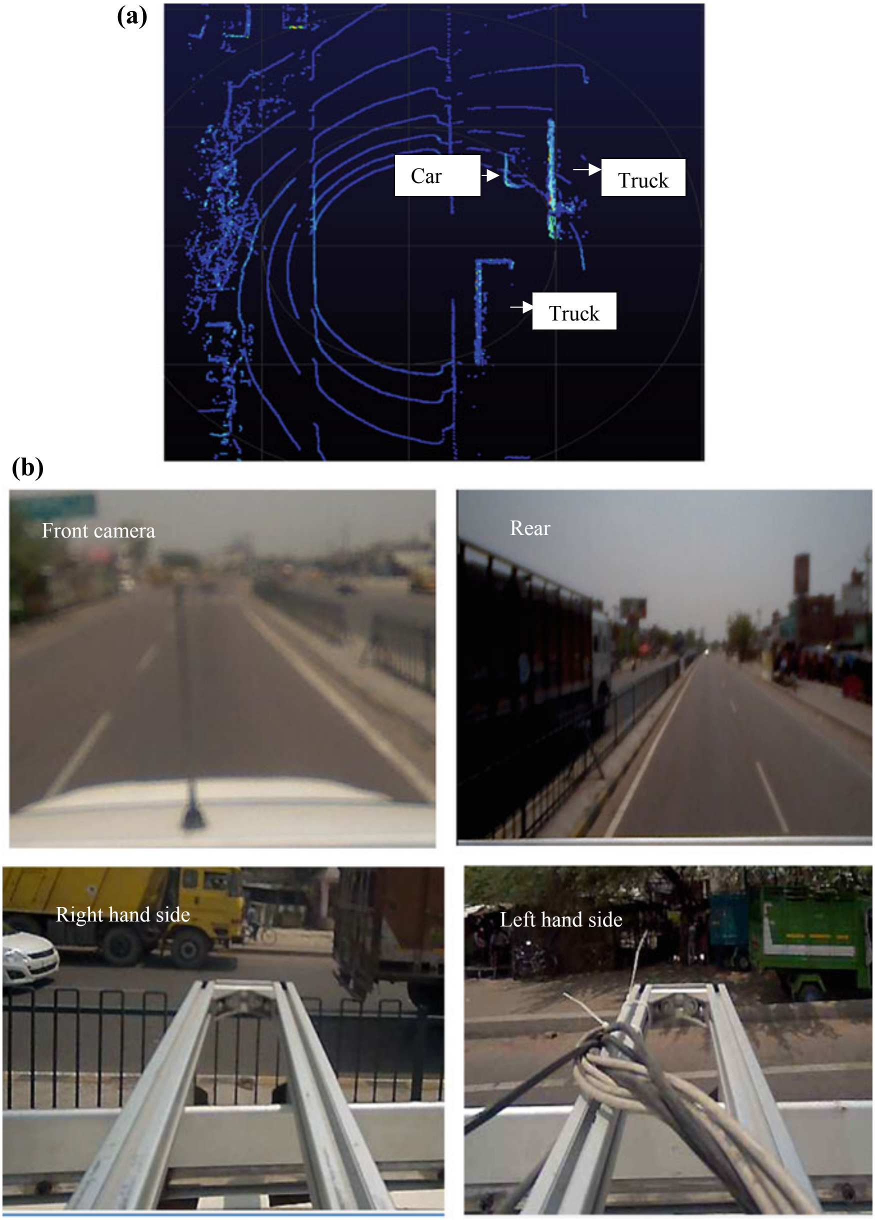

Fig. 4 Concurrent view of the surrounding from the video cameras and the LiDAR. **a** LiDAR scan. **b** Video-camera images

6 Conclusion

In low- and middle-income countries like India, due to vehicle heterogeneity on the road and poor lane discipline, vehicular interaction occurs along both lateral and longitudinal directions. Collecting and understanding vehicle information in such traffic conditions is important for a better understanding of traffic. The conventional methods of data collection on driver behavior cannot capture accurate information in such traffic conditions. Instrumented vehicles help to observe individual driver behavior accurately and precisely. The paper discusses the types of sensors used in building an instrumented vehicle to capture vehicle behavior in heterogeneous and poor lane disciplined traffic conditions. Since these sensors give a large quantity of data, it is important to budget time and resources for data processing.

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Simulation of Classified Lane-Wise Vehicle Count at Toll Plazas Using Monte Carlo Simulation and Probability-Based Discrete Random Number Generation

V. B. Soorya, T. M. Rahul, and Shriniwas S. Arkatkar

Abstract The simulation-based prediction of traffic conditions based on current and past traffic observations is an important component in the intelligent transportation system (ITS) applications. Infrastructure, in the form of toll plazas, is inevitable for collection of revenue after the development of National Highways in India. Intelligent transportation systems utilize the advanced technologies and employ them in the field of transportation. The implementation of advanced traffic management systems (ATMS) at toll plazas will improve the toll plaza operations. A simulation model can help in the evaluation and optimization of toll operations of existing toll plazas as well as in the planning and design of similar systems. With this motivation, a lane-wise classified vehicle count prediction algorithm, which can simulate traffic conditions at any time interval, has been developed in this study based on Monte Carlo simulation (MCS). Vehicle arrival was modeled by assuming Poisson's distribution, followed by classification. Lane selection was done using the probability-based discrete random number generation. Radio-frequency identification (RFID)-based electronic toll collection (ETC) system gives timely varying traffic counts observed at the toll plaza, which has been utilized to develop and validate the simulation model. The flexibility with respect to the probabilities of the proposed algorithm makes it more applicable in the area of ITS. The observed vehicle count for each lane has been compared with the simulated values. The results of statistical tests show that there is no significant difference between actual and simulated traffic for each lane.

Keywords Toll lane choice · Monte carlo simulation (MCS) · Intelligent transportation system (ITS)

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1 Introduction

Intelligent transportation systems (ITS) apply advanced technologies to improve the efficiency and safety of transportation systems (Lelitha Vanajakshi et al. 2010). The ability to accurately predict current traffic conditions, which is a critical component of advanced traffic management systems (ATMS) applications, is a fundamental part of ITS (Ben-Akiva et al. 1998). The provision of timely and accurate traffic information is valuable for both operators and users of the infrastructure. The installation of traffic management centers that imparts this information would improve the efficiency, performance, and coordination of all traffic operations. However, successful implementation of such advanced systems requires a thorough understanding of historical, present, and future traffic conditions of the system. Development of models and algorithms which are suitable for implementation of ITS is an important task for the researchers in this area. Aptly, the importance of short-term traffic prediction module in ATMS architecture was highlighted. Heterogenous non-lane-based traffic conditions in India make the simulation of traffic conditions somewhat challenging (Arasan and Koshy 2005).

The current study focuses on the application of these advanced technologies in toll plaza operations. Toll plazas are inevitable for revenue generation to recover the cost of construction, operation, and maintenance of transportation systems (Prozzi et al. 2009). There are different toll collection technologies available, and further advancements are possible only when it is tied in with ITS (Persad et al. 2007). Electronic toll collection (ETC) systems that reduce delay in toll collection is an important application of ITS. The main functional components of the ETC are automatic vehicle identification (AVI), automatic vehicle classification (AVC), and vehicle enforcement system (VES) (Muthulakshmi et al. 2015). These components require to perform various analysis and simulation-based predictions and for this, the type of vehicles, date and time, lane number, etc., need to be recorded.

Generally, the traffic count has been found to vary with time at toll plazas (Chao 2000). Effective toll plaza operation is very important during both the peak and the off-peak hours. Suboptimal operation during peak hours adversely affects the throughput of the facilities, whereas during hours with low traffic, it may result in high operating costs. A stochastic queueing can be undertaken to understand this impact during peak and off-peak periods. Stochastic queueing analysis at toll plazas can be performed by two approaches, one is an analytical approach based on sets of mathematical equations, and the other is a simulation technique done on a microscopic scale (May 1990). Among these methods, a simulation model has been found to give a more in-depth analysis and comprehensive understanding of the toll plaza operations (Ceballos and Curtis 2004). Here, the lane configurations can change electronically over time using some electronic display technologies (Hassett 2003) that utilize information of optimal lane configurations (Kim 2009).

With this motivation, a lane-wise classified vehicle count prediction algorithm, which can simulate traffic conditions at any time interval, has been developed based on Monte Carlo simulation (MCS) and probability-based discrete random number generation. The study has been carried out using hourly vehicle count data collected from a toll plaza in India. The main objective of the study was to develop a computer-based algorithm to simulate the number of vehicles arrived at the toll plaza followed by a prediction of lane-wise classified vehicle count. Along with this, a microscopic analysis of lane-wise distribution of vehicles over time and its comparison with the service level standards at toll plazas, in an Indian scenario, have been accomplished. The present study may be used to understand the changing lane configuration at toll plazas by predicting lane-wise traffic count, and subsequently to assess the alternate treatments, and safety and efficiency. A detailed literature review, which explains the available studies regarding toll plaza simulations and research gap in this area, is presented in the following section.

2 Literature Review

Current literature review elicits different studies related to toll plaza optimization using simulation and analytical approaches. Microscopic simulation can be done using simulation models developed using commercially available software. Other simulation models include Corridor Simulation (CORSIM) (Traffic Software Integrated System 2000), Visual Solutions Simulation (VISSIM) (PTV AG Corporation 2000), and Parallel Microscopic Simulation (PARAMICS) (Quadstone Limited 2003). However, all these systems were restricted in their comprehensive output to carry out the toll plaza optimization operations and most of them are for homogenous lane-based traffic conditions (Al-Deek et al. 2005).

One of the initial studies carried out by Gulewicz and Danko 1995 evaluated optimum lane staffing requirement of a toll plaza based on simulation results. The simulation model was developed using a software interface General Purpose Simulation System (GPSS) world, by means of input as vehicle inter-arrival data for each arrival lane and by vehicle type.

After the advancement of ETC systems, considerable studies have been made to evaluate the effect of the introduction of fully automated ETC lanes in toll plazas. Al-Deek (2001), developed a stochastic discrete event microsimulation model called toll plaza simulation model (TPSIM) and applied it to a toll plaza in Florida. Al-Deek et al. (2005), improved TPSIM by integration of a car-following model with lane selection algorithm and evaluated the lane throughput, vehicle delay and queue length.

Astaria et al. (2001) presented a much more improved microsimulation model for toll plazas. Along with car-following and lane-changing model, they had introduced a booth selection model in which each driver was assigned a utility function that depended on the queue length. Subsequently, another study by Mudigonda et al. (2009) found that drivers' lane selection comprises the geometry of entrance and exit way, and the driver's wait time. They had used origin and destination demand matrix created from ETC dataset as an input for simulation in Paramics software. Different from other studies, they had considered lane selection probabilities also. Driver's lane selection was based on utility maximization of lanes. Before that, optimal toll plaza configuration was found using Paramics model by Chien et al. (2005).

Ceballos and Curtis (2004), conducted a comparative study of simulation models and analytical queuing models for queue analysis at toll plazas. They found that simulation offers a more inclusive understanding of the toll plaza operation and performance, and that analytical queuing models could be used for the analysis in early stages of planning.

Using AIMSUN simulation package, specific to European vehicles, Poon and Dia (2005) discussed the toll plaza operations and effect of fully automated toll lanes. They also concluded that the output of these simulation results depends heavily on the accuracy of input parameters. Shitama et al. (2006), simulated the traffic conditions at a toll plaza in Japan by collecting successive vehicle tracking data using extensive video graphic survey. Hamid (2011) mentioned that toll operators decide the number of toll booths required, based on a trial and error exercise, which causes congestion in the long run. Hence, they proposed new toll booth configurations for a toll plaza at Malesia based on simulation results from VISSIM software. Recently, Vidanapathirana and Pasindu (2017) found that installation of ETC systems can increase the lane capacity by up to 50%, using the simulation results from VISSIM software.

Munawar and Andriyanto (2013) proposed a computer simulation model to predict queues and delays at a toll plaza in Jakarta. Traffic count data, classified vehicle percentage, and service time at toll plaza were collected using video recording technique and considered as the data input.

In an Indian context, there are only a few studies, like the one by Parmar et al. (2013), which modeled the drivers' toll lane choice behavior using random coefficients-based mixed logit model. The analysis was done using 450 automobiles user's lane choice data. The model was not validated.

From the extensive literature review, it can be observed that most of the studies were carried out using commercial microscopic simulation software. Development of such simulation models requires the estimation of accurate microscopic parameters that necessitate more experimental observations. Most of the studies evaluated the effect of the introduction of ETC lanes. However, it can be seen that the majority of these studies were from homogeneous and lane-based traffic conditions. In developing countries like India, the traffic is composed of different categories of vehicles, making it highly heterogeneous in nature and difficult to account for. In addition, lane-wise prediction over time at toll plazas has not been explored in detail. The contribution of this paper, to the literature, is as follows. It postulates a computer-based algorithm to predict the time-dependent changes in traffic conditions in toll plazas. The model developed using the data collected from an automatic vehicle identification system in a toll plaza does not require an extensive survey. Moreover, the model is validated using the actual data collected for one day. By using historical hourly traffic data, traffic conditions have been simulated for lower time intervals for a better understanding of queue formations and utilization of lanes. The flexibility with respect to the probabilities of the proposed algorithm makes it more applicable in the area of ITS.

Table 1 Descriptive statistics of data collected

Direction of flow and lane number		Off-peak hour (Veh/hour)		Peak hour (Veh/hour)	
		MEAN	STD	MEAN	STD
Krishnagiri to Thopur	LANE 1	111	29	196	15
	LANE 2	32	10	49	4
	LANE 3	81	25	92	18
	LANE 4	83	17	98	8
	LANE 5	95	11	96	9
	LANE 6	72	21	71	11
Thopur to Krishnagiri	LANE 7	63	24	81	23
	LANE 8	96	9	93	11
	LANE 9	80	9	98	10
	LANE 10	77	22	84	24
	LANE 11	33	12	49	13
	LANE 12	114	30	216	20

3 Data Collection

The data collected using the electronic entry system at a semi-automated toll plaza in India, Krishnagiri-Thoppur toll plaza, which is located on NH-44, Krishnagiri-Hosur highway in Tamil Nadu, was used in this study. The road network is four-lane divided carriageway and the toll plaza section consists of 12-lanes, each with toll booths, in which 1–6 are in the Krishnagiri to Thopur direction and 7–12 are in the Thopur to Krishnagiri direction. Hourly classified lane-wise data was collected from June 20, 2016 to June 26, 2016. Table 1 shows the descriptive statistics of total data collected for both directions. The lane-wise distribution of vehicles shows that, among all lanes, lane 1 and lane 12 have the maximum number of vehicles in off-peak and peak hour. Similarly, lane 2 and lane 11 possess the lowest number of vehicles per hour. Since traffic conditions in both the directions are not significantly different, the developed simulation model is validated using data collected from Krishnagiri to Thopur direction.

4 Methodology

The methodology followed for the simulation is as shown in Fig. 1. The basic steps involved in the development of the simulation model consist of the formulation of a data inputting system and the random number generation. The first step in the simulation process is vehicle arrival generation at a specified time interval using Monte Carlo simulation (MCS). MCS is a type of simulation that depends on repeated

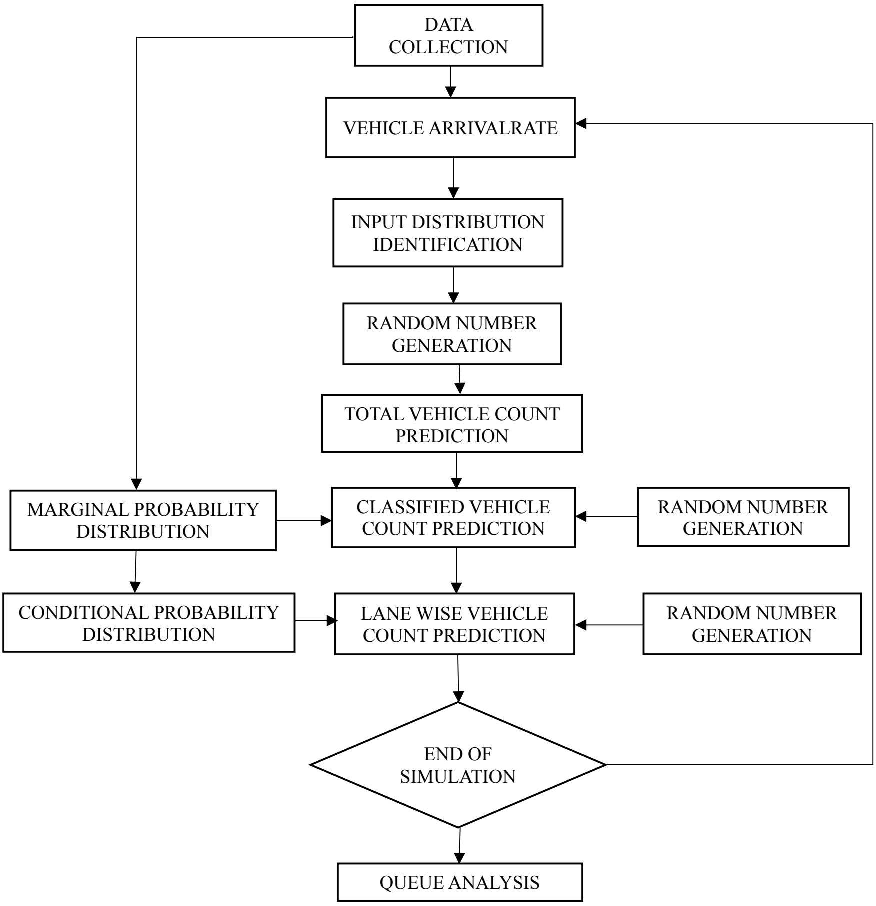

Fig. 1 Simulation flowchart

random sampling and statistical analysis to compute the results. Input distribution identification (distribution fitting) and random number generation are the important steps in this process (Mishra et al. 2015). In MCS, the first step is to identify the best fitting statistical distribution for the random variables, which can be done using a maximum likelihood estimator for the distribution parameters. Next step is to generate random numbers between 0 and 1 (Raychaudhuri 2008). The prediction can be done using the random numbers generated and the parameters of the fitted statistical distributions. In the present study, the best fitting distribution for the vehicle arrival count from the historical data was determined using Poisson's distribution. The probability mass function of Poisson's distribution is given in Eq. 1:

$$p(x) = \frac{\lambda^x e^{-\lambda}}{x!} \tag{1}$$

where $p(x)$ is the probability of 'x' vehicles arriving in an interval 't,' and 'λ' is the mean arrival rate of vehicles. The current study uses mean arrival rate of vehicles at each time interval (every minute), determined from existing data, to generate the vehicle arrivals.

Once the total number of vehicles is generated, the algorithm generates a classified count of these vehicles using the marginal discrete probability distribution, attached with different vehicle types, obtained from historical data. The next step utilizes the discrete probability distribution of the historical lane-wise vehicle count conditional on the total classified vehicle count to predict the lane selected by each vehicle type. Both the above-mentioned steps can be achieved by knowing the joint probability distributions between the vehicle type and the lane selection of vehicles entering the toll plaza. The joint probability mass function (PMF), $p_{x,y}$ (x, y) of two discrete random variables X and Y, is defined as given in Eq. 2.

$$p_{x,y}(x, y) \equiv P[(X = x) \cap (Y = y)] \tag{2}$$

The joint cumulative distribution function (CDF) is defined in Eq. 3.

$$F_{x,y}(x, y) \equiv P[(X \leq x) \cap (Y \leq y)] = \sum_{x_i \leq x} \sum_{y_i \leq y} p_{x,y}(x_i, y_i) \tag{3}$$

First type of distribution that can be obtained from joint distribution is the marginal probability distribution and the next is a conditional probability distribution. The behavior of a particular variable irrespective of the other is described by marginal probability mass function. Here, the marginal probability distribution of classified count of vehicles (Y) considering first, PMF ($p_y(y)$) and CDF ($F_y(y)$), is given in Eqs. 4 and 5, respectively.

$$p_y(y) \equiv P[Y = y] = \sum_{\text{all } x_i} p(y, x_i) \tag{4}$$

$$F_y(y) \equiv P[Y \leq y] = \sum_{y_i \leq y} p_y(y_i) \tag{5}$$

Then, if the value of one of the variables is known, say $Y = y_{o,}$ the relative likelihoods of the various values of the other variable are given by $p_{x,y}(x, y_o)$. If these values are normalized so that their sum is unity, they will form a proper distribution function. This distribution is called the conditional probability mass function of 'X' given Y, $p_x|_y(x, y)$, and is given in Eq. (6).

$$p_{x|y}(x, y) = P[X = x|Y = y] = \frac{p_{x,y(x,y)}}{\sum_{\text{all } x_i} p_{x,y(x_i,y)}} \tag{6}$$

Hence, by knowing the historical classified vehicle count in each lane and number of total arrived vehicles in a particular time interval, the marginal probabilities corresponding to each class of vehicle, represented by variable 'Y,' can be found out. Similarly, one can find the probabilities of each lane, represented by variable 'X,' conditional on the classified vehicle count. In the current study, if $F_i(Y)$ represents the marginal distribution of Y_i (vehicle type) with 6 categories of vehicles ($i = 1, 2, \ldots 6$) and $F_k(X|Y)$ be the conditional distribution of $X_{k,}$ (vehicle lane) with 6 lanes ($k = 1, 2, 3 \ldots 6$), the algorithm would first generate 'Y' marginally from $F_i(Y)$ and next generate 'X' from $F_k(X \mid Y)$.

Once the probabilities are known, to generate a realization of random variable, discrete version of inverse transform method can be used. That is, let 'X' be discrete random variable with probabilities $P(X = x_i) = p_i$, here, $i = 0, 1, 2 \ldots,$ $\sum_{i=0}^{\infty} p_i = 1$. To generate a realization of X, first generate a random number U from U (0, 1) and then set $X = x_i$, if $\sum_{j=0}^{i-1} p_i \leq U < \sum_{j=0}^{i} p_j$.

Lastly, the procedure starting with the generation of vehicles at a specific time interval may be repeated for each time interval. Based on the above methodology, the simulation was carried out using MATLAB software, for each minute of the entire day. Validation of the model and relevant results are explained in the next section.

5 Validation of Simulation Results

The results from the simulation model and its validation and analysis are presented in this section. Simulation has been carried out for the entire 24 h of the day. Results from the one simulation run have been validated using the observed hourly vehicle count at the toll plaza. The validation of the prediction models can be performed using mean absolute percentage error (MAPE) using Eq. (7) and mean absolute error (MAE) using Eq. (8), between the predicted and observed values.

$$\text{MAPE} = \frac{1}{n}\sum_{i=1}^{n}\frac{(A-B)}{A}100 \tag{7}$$

$$\text{MAE} = \frac{1}{n}\sum_{i=1}^{n}|A-B| \tag{8}$$

where A is the actual vehicle count observed in the field, B is the simulated vehicle count, and n is the number of time intervals considered.

Tables 2 and 3 show the distribution of actual and predicted vehicle count and MAPE and MAE for lane-wise prediction and classification for peak and off-peak hour separately. Overall MAPE of 20.7% is obtained for lane-wise simulation, and 20.29% is obtained for classification. Also, the measured average MAE of 20 vehicles per hour is obtained for lane-wise simulation, and 13 vehicles per hour are obtained for classification, where the average observed vehicle count for that day from the

Table 2 Validation of simulation results for lane-wise prediction

Lane number	Actual (Veh/hr)	Predicted (Veh/hr)	Actual (Veh/hr)	Predicted (Veh/hr)	MAPE (%)		MAE (Veh/hr)	
	Off-peak	Off-peak	Peak	Peak	Off-peak	Peak	Off-peak	Peak
LANE 1	126	93	182	121	25.31	32.01	34	61
LANE 2	33	33	54	56	22.72	22.72	7	13
LANE 3	84	84	123	118	15.84	15.84	19	18
LANE 4	91	91	101	114	17.94	17.94	12	18
LANE 5	103	97	105	116	15.76	15.76	16	18
LANE 6	66	70	101	98	19.16	19.16	18	19

Table 3 Validation of simulation results for vehicle classification

Vehicle category	Actual (Veh/hr)	Predicted (Veh/hr)	Actual (Veh/hr)	Predicted (Veh/hr)	MAPE (%)		MAE (Veh/hr)	
	Off-peak	Off-peak	Peak	Peak	Off-peak	Peak	Off-peak	Peak
BUS	35	40	39	47	26.08	25.62	8	9
CJV	161	170	251	313	10.41	24.73	15	63
LCV	54	60	46	55	15.87	23.48	7	10
MAV	11	12	11	14	28.23	25.93	3	2
Truck 2 axle	60	66	49	63	13.78	30.68	8	14
Truck 3/4 axle	117	120	108	131	8.36	24.97	9	26

field was 100 vehicles per hour. The obtained accuracy is adequate in most of the ITS applications. Due to high variation in traffic during peak hours, prediction accuracy during peak hours is less than off-peak hours. This can be improved by more simulation runs for each time interval. During off-peak hours, for lanes 2, 3 & 4 show more accurate prediction, but it shows higher MAPE because MAPE produces much higher values when the actual values are small.

An increased MAPE is observed during some hours of the day, and these hours show some variation in simulated and actual vehicles. A comparison of lane-wise prediction accuracy of 24-hour vehicle count in terms of MAPE and MAE are shown in Figs. 2 and 3, respectively. Maximum MAPE and MAE are observed in lane 1, and all the other lanes MAPE and MAE are less than 20 vehicles per hour. This is may be due to the abnormal field conditions such as temporary lane closure, or high variation in traffic, on that particular day. Similarly, Figs. 4 and 5 show the accuracy in classification of vehicles in each hour of the day. It is clear from these analyses that at peak hours, MAPE is more as compared with off-peak hours. This might be improved by more simulation runs for each time interval.

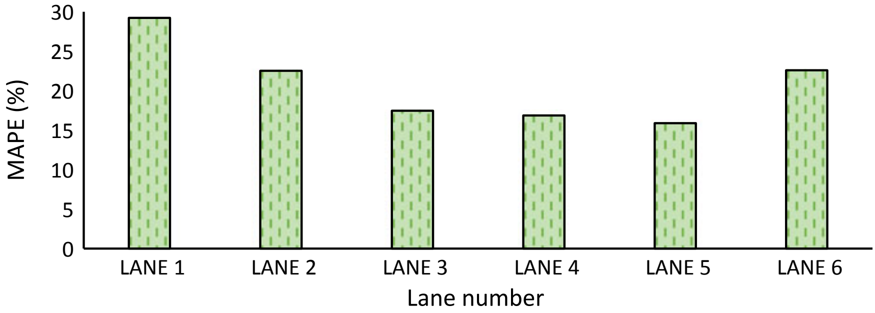

Fig. 2 MAPE (%) for 24-h lane-wise vehicle count prediction

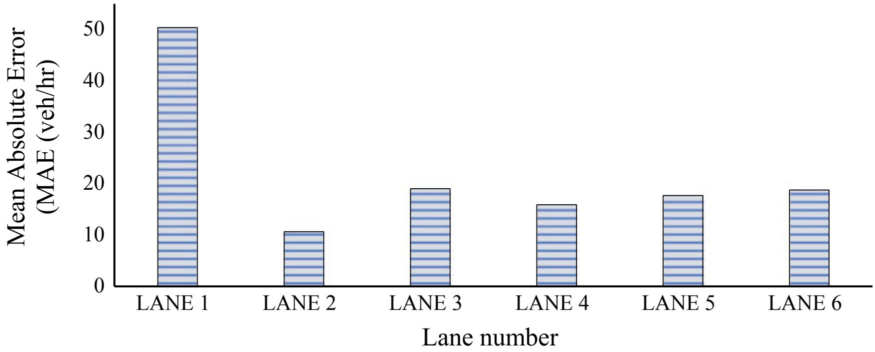

Fig. 3 MAE for 24-h lane-wise vehicle count prediction

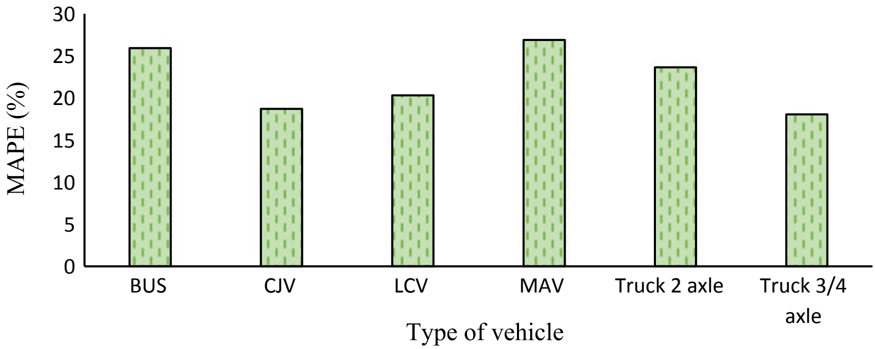

Fig. 4 MAPE (%) for 24-h classified vehicle count prediction

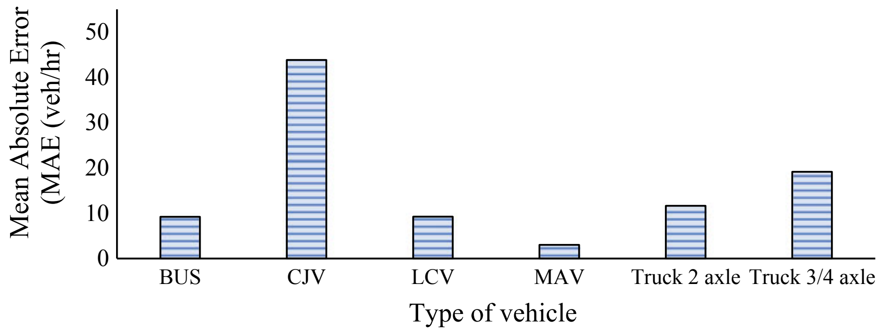

Fig. 5 MAE for 24-h classified vehicle count prediction

5.1 Capacity Analysis at Toll Plaza

A capacity analysis of the toll plaza obtained using the simulation results is discussed in this section. There is essentially no sound theoretical basis for capacity and level of service at toll plazas. Nevertheless, according to the Manual of specifications & standards (2010) of Indian government, the capacity of a semi-automatic lane is suggested as 240 vehicles per hour. When the number of vehicles in the lanes exceeds this limit, the queue of vehicles becomes so large that it will result in unnecessary delay for the users.

The present study considers four vehicles per minute per lane as the capacity exceeding point. Figure 6 shows the number of times the vehicle arrival count exceeds four vehicles per minute in lane 1, over a day. The maximum number—six—exists for the seventeenth hour of the day. Approximately similar observations are made for all the lanes. It is observed that during peak hours, that are 10–14 and 17th, 18th hours of the day, the capacity is exceeded 4–6 times in all the lanes.

Furthermore, the distribution of the number of vehicles in lane 1 for each minute of these hours is examined. Figure 7 shows the number of vehicles present in each minute of the seventeenth hour (peak hour) and fourth hour (off-peak hour) in lane 1. In the off-peak hour, during almost all the time interval, vehicle arrival is less

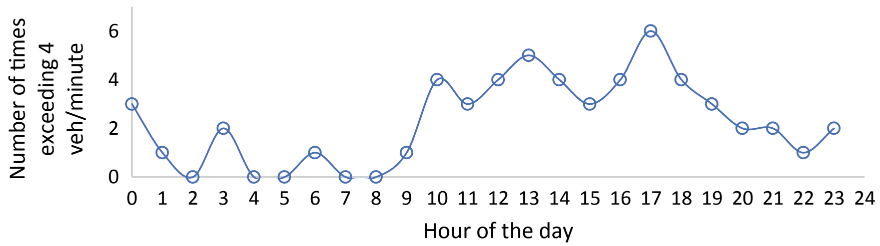

Fig. 6 Number of times exceeding four vehicles per minute over a day (lane 1)

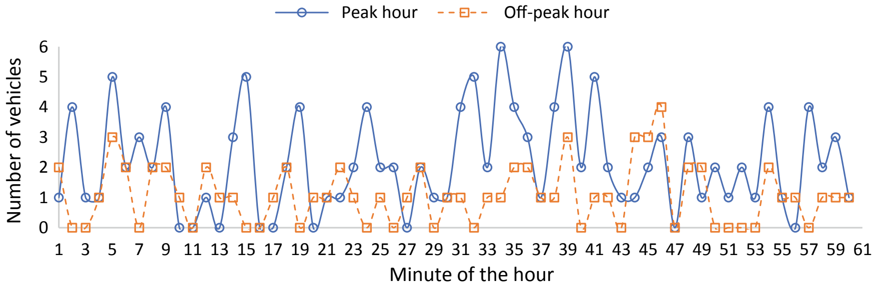

Fig. 7 Distribution of number of vehicles present in selected peak hour and off-peak hour (lane 1)

than four vehicles per minute. A vehicle arrival rate of 0–1 vehicles per minute is observed 42 times. A similar observation is made for all the lanes. Thus, during off-peak hours, alternate treatments can be evaluated by closing one or two lanes to reduce the operating cost. In the peak hours, most of the time there is an existence of flow greater than two vehicles per minute, which may cause a longer queue length, wait time, and thus delay.

6 Summary and Conclusions

The present study focuses on the simulation of lane-wise classified vehicle count based on MCS and probability-based random number generation. This is going to be an imperative step in optimizing tollway operations as simulation-based predicted tollway count is a mandatory input variable for any policy decisions on tollway operations, particularly for ATMS component deployment at tollway. Further, such models would also aid in optimizing or planning the operations at toll plazas with a scientific base.

With this motivation, using the data collected from a toll plaza in India, the present study develops a toll plaza simulation model. The performance evaluation of the algorithm showed a good predictive capability for the model. The present study also undertook a capacity analysis at a toll plaza during specific time intervals using the simulation results. The results from the current study show that during peak hours (10–14 and 17 and 18th hours of the day) the capacity is exceeded 4–6 times in all the lanes. On the contrary, the vehicle flow never exceeded the capacity during off-peak hours. On analyzing the off-peak hours, 42 times, 0–1 vehicle per minute was observed in lane 1.

A major policy decision that may be adopted to manage the vehicle delay during peak hour is diverting vehicles from one lane to other in peak hours. Further, operational costs may be reduced during off-peak hours by closing one or two lanes. These decisions can be taken on the basis of evaluating each alternative treatment by modifying the algorithm inputs, and this could form a scope for a future study. Likewise, tollway operations and other relevant logistic such as manpower to be deployed for specific time period/hour of the day can be planned more precisely using a detailed estimate of toll count in real time. Further improvements in the analysis can be done by means of changes in the service times at each tollbooth. Using the service times in each lane and simulation output, a microscopic queuing analysis can be carried out in future.

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Performance Evaluation of Urban Roadway Links Using V-Box

G. Yadav and A. Dhamaniya

Abstract Present study attempts to evaluate the operational efficiency of urban roadway links using V-Box. An urban arterial of 5.8 km length in Surat city of Gujarat is selected and divided into six links based on the number of intersections in the entire facility. All links are identical in geometry (six-lane divided) with different land use patterns. In order to evaluate the operational efficiency of these links, the number of runs of different categories of vehicles including motorized two-wheelers (2W), auto-rickshaw (3W), big car and small car (4W) and heavy vehicle (bus) has been taken at different time periods. A number of 30 runs of each vehicle category were taken on the entire facility for peak hours and non-peak hours. The collected data has been analysed for spatial and temporal variation in speed for different vehicle categories. The spatial variation has been checked by analysing the inter-segmental variation of speed. Also, within each segment, the variation of speed for different category of vehicles for peak and non-peak hours has been studied. Further, cumulative speed plots have been plotted to check the variability of speed for different segments for different vehicle categories. Furthermore, the excess speed over posted speed limit for different mid-block sections has also been checked.

Keywords V-Box · Temporal variation · Spatial variation

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1 Introduction

In a developing country like India, the traffic in urban streets is seeing a boost which affects the quality or the level of facility being provided by the road. Primary objective of this study is to carry out the performance evaluation of the urban roadway links using V-Box survey. This study aims to cover different aspects of traffic including average speed of different vehicles, travel time, controlled and uncontrolled delays, congestion factors and spatial and temporal variation of speed for the sections using V-Box analysis and checking its effectiveness in data collection and analysis. Variation in speed of vehicles on different mid-block segments of the same road is also an important aspect of analysing the performance of the roadway. How speed of stream or the speed of different vehicle categories changes on the same stretch with for different time periods is also needed to be determined for a given segment. This is known as spatial variation of speed. The level of service (LOS) term has been coined by the Highway Capacity Manual (HCM) which represents the level or standard of facility a user can derive from a road under various functional characteristics and traffic volumes. The term level of service is defined as a qualitative measure which describes the operational conditions within traffic stream, and their experience by motorists and travellers. This study checks this effect on six different mid-block sections of Gaurav Path in Surat city. In urban areas, with change in land use pattern along the roadway, the level of service changes. It leads to variation in travel time, average speed, delays and other parameters. The factor 'land use' has not been effectively considered for the determination of LOS till date. Hence, this study attempts to carry out the LOS determination with land use as a measure of effectiveness. The study shall aim to find the existing level of service on the selected road section and determine the present traffic conditions, so as to state if the present conditions full-fill the present demand or any measures are to be taken to overcome the problem eventually to be faced in the future. The outcomes of the study will be helpful for the analysis of the roads sharing similar geometric characteristics, land use and traffic conditions (composition, control parameters). Below curve shows the variation of speed with volume (Fig. 1).

2 Literature Review

In past years, many researchers have tried to assess level of service on the basis of different parameters. Maitra et al. (1999) have presented a unified methodology for the quantification of congestion at urban mid-block sections, relating the level of congestion to the casual influences of traffic movement by modelling, and demonstrate the potential use of modelled congestion as a measure of effectiveness for obtaining the LOS. However, the scope of this paper is restricted to the application

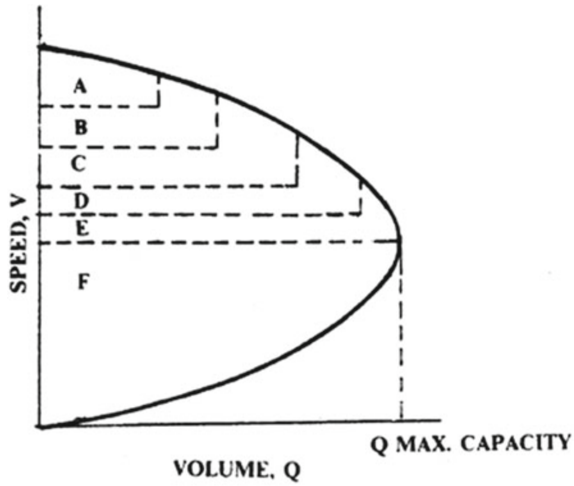

Fig. 1 Speed–volume curve showing level of service (IRC 106-1990: Guidelines For Capacity of Urban Roads in Plain Areas)

of the model on roadway condition in terms of traffic lanes only. Marwah and Singh (2000) has developed a traffic simulation model, which can imitate the movement of heterogeneous traffic, to analyse the various environment of the road system. The limitation of this study was that only cars, 2W and NMVs were considered during simulation.

Maitra et al. (2003) have captured the mixed traffic operations on roads where partial widening has been done, by modelling of congestion. Using the congestion models, the benefits, if any, from such partial widening have been explored by comparing the congestion and level of service characteristics on selected study roads. But, the limitation of this study was that surface conditions are not considered for the development of congestion models. Bhuyan and Rao (2011) have used average travel speed as the (MOE), which in this case has been derived from second by second speed data obtained from GPS receiver fitted on mobile vehicles. Hierarchical agglomerative clustering (HAC) is implemented on average travel speeds to define the speed ranges of urban street and LOS categories which are valid in Indian context are different from that values specified in HCM (2000). Under limitation of this study is that we need large number of speed data points in order to get better results in classifications.

Although various parameters have been exercized to assess the level of service on urban arterial roads, but the factor ‘land use’ is still untouched for the determination of LOS till date. Hence this study will carry out the LOS determination with land use as a measure of effectiveness. The study will also find the existing level of service on the selected road section and determine the present traffic conditions, so as to state if the present conditions full-fill the present demand or any measures are to be taken to overcome the problem eventually to be faced in the future.

3 Data Analysis

3.1 Study Area

The total stretch of 5.7 km has been selected to carry out the study starting from Athwa Gate Circle and ending at Rahul Raj Mall in the city of Surat, Gujarat (Fig. 2; Table 1).

The whole stretch has been divided into six segments as detailed in the above table and google map. AutoCAD drawing of full stretch has also been drawn showing the land use pattern and the section of Athwa Gate to Police Parade Ground has been shown in Fig. 3.

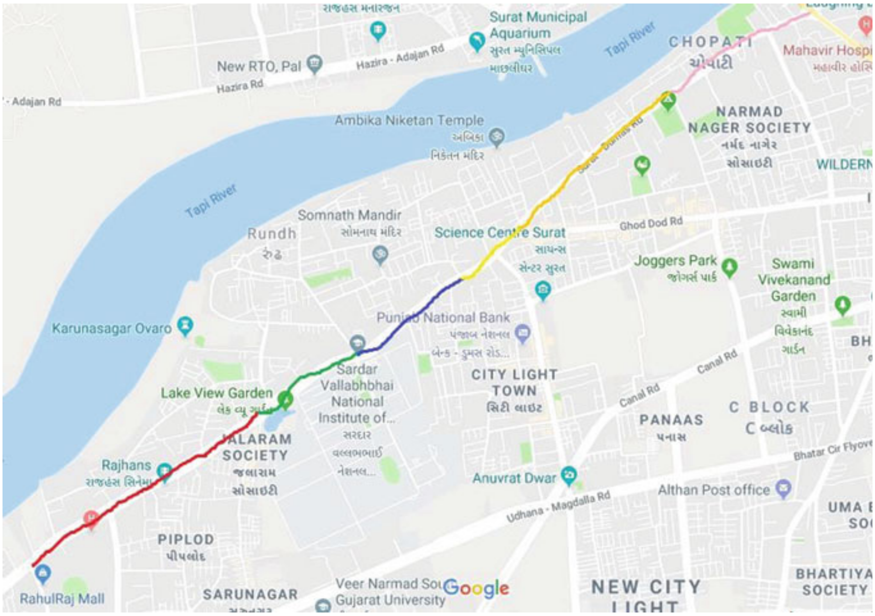

Fig. 2 Google Map of whole stretch from Athwa Gate to Rahul Raj Mall

Table 1 Details of various mid-block sections

Colour	Mid-block section	Distance (km)
	Rahul Raj-Kargil Chowk (RR-KC)	1.59
	Kargil Chowk-SVNIT (KC-SV)	0.67
	SVNIT-Sargam Shopping Centre (SV-SR)	0.76
	Sargam Shopping Centre-Parle Point (SR-PR)	0.42
	Parle Point-Police Parade Ground (PR-PP)	1.22
	Police Parade Ground-Athwa Gate (PP-AG)	0.91

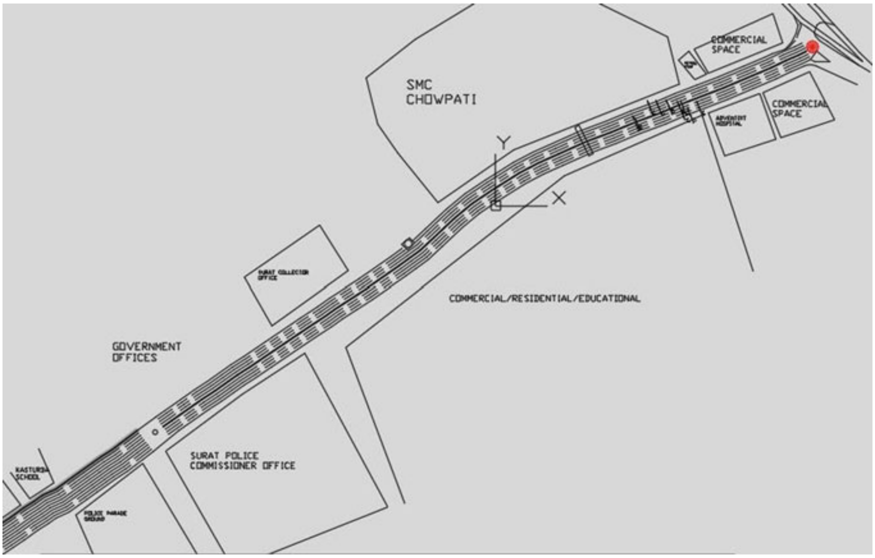

Fig. 3 AutoCAD drawing of Athwa Gate–Police Parade Ground Section

3.2 Data Collection

Data collection mainly comprised of two stages:

1. **Speed Data Collection using V-Box**

Speed data of different categories of vehicles, i.e., two-wheelers, three-wheelers, big and small cars and buses, was collected during this stage. These data were collected during different days and during different periods of time and were categorized into morning peak, off-peak and evening peak hours. In total, 30 samples, in both up and down directions for each vehicle category, were taken for the study (Fig. 4).

2. **Vehicular Volume Count**

Finally, for relating vehicular volume with speed obtained from V-Box, volume count was also done. This count was done for 1 h for each mid-block section for all three time periods, i.e., morning peak, off-peak and evening peak (Fig. 5).

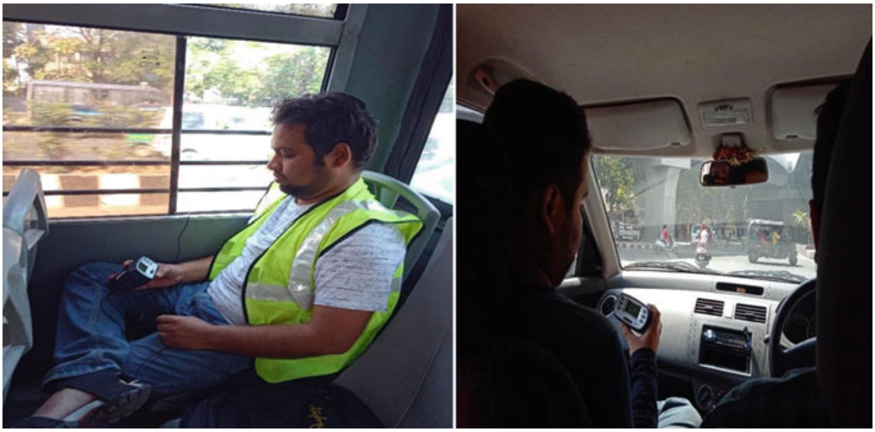

Fig. 4 V-Box speed data collection

Fig. 5 Volume count survey

3.3 V-Box Data Collection

From the speed data collected from V-Box, speed-distance plots were plotted using performance box software for different time period and for different stretches. Below are some sample speed–distance plots of different vehicle categories for the stretch Athwa Gate to Police Parade Ground (Figs. 6, 7 and 8).

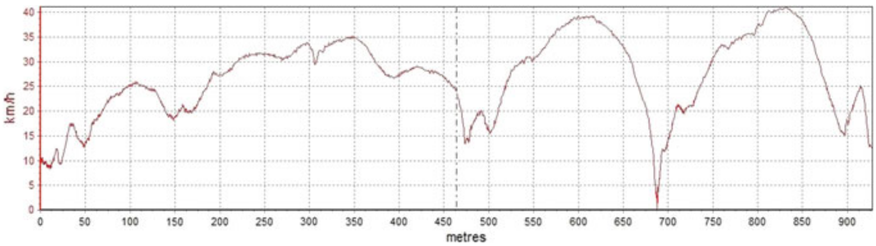

Fig. 6 Variation in speed with space for 2W

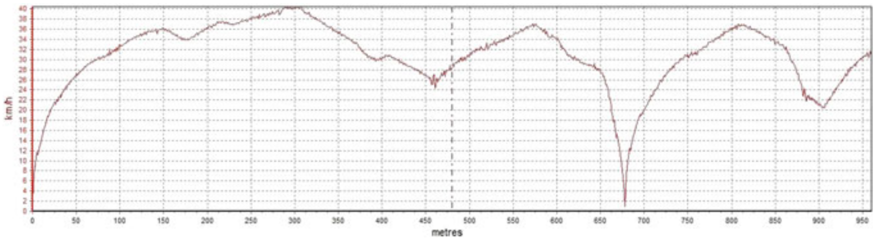

Fig. 7 Variation in speed with space for 3W

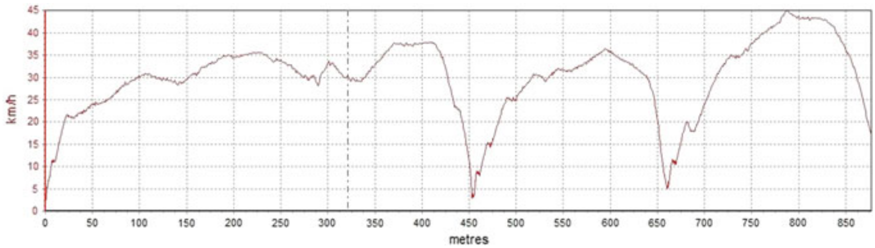

Fig. 8 Variation in speed with space for small car

3.4 Data Analysis

3.4.1 Spatial Variation of Speed

Spatial variation of speed was analysed for each category of vehicle separately for different mid-block sections and cumulative frequency versus mean speed plot has been plotted for each category for different time periods (Figs. 9, 10 and 11).

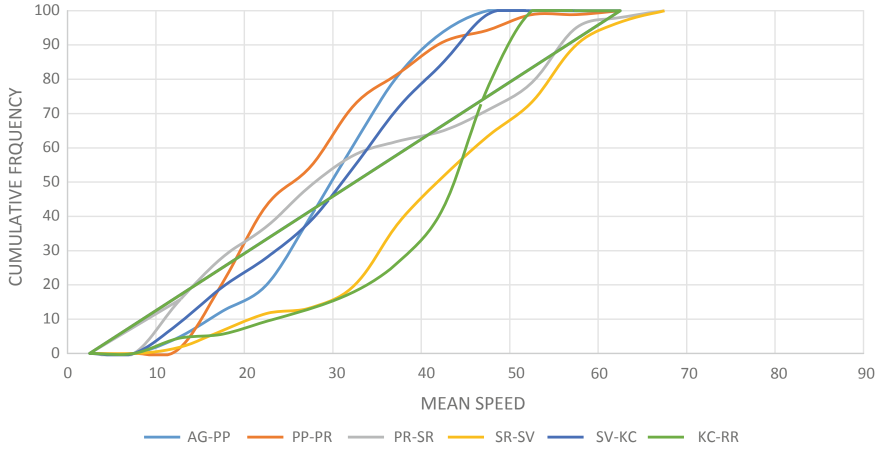

Fig. 9 2 wheeler, Evening Peak

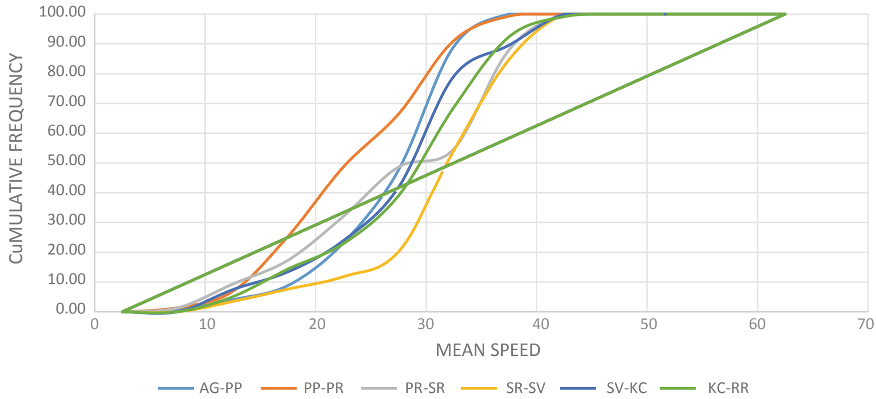

Fig. 10 3W, evening peak

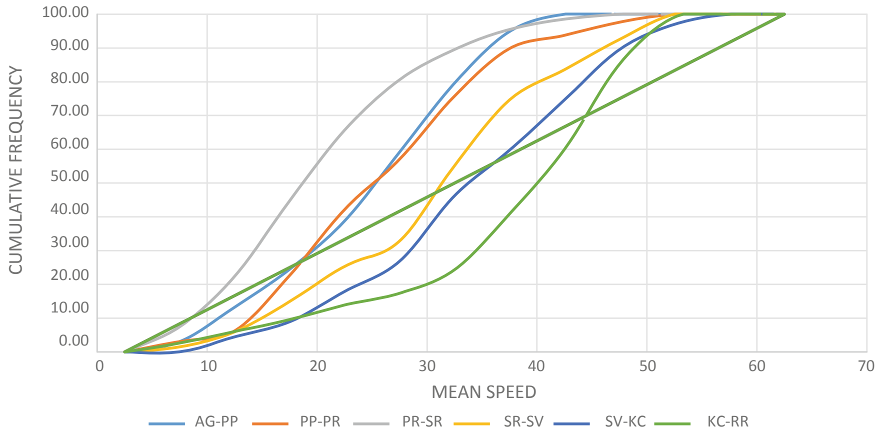

Fig. 11 Small car, evening peak

3.4.2 Temporal Variation

Here, the variation in speed of vehicles was analysed for different period of time, i.e., morning peak, off-peak and evening peak within a particular stretch. Mean speed versus cumulative frequency plots of some samples have been shown here (Figs. 12, 13, 14 and 15).

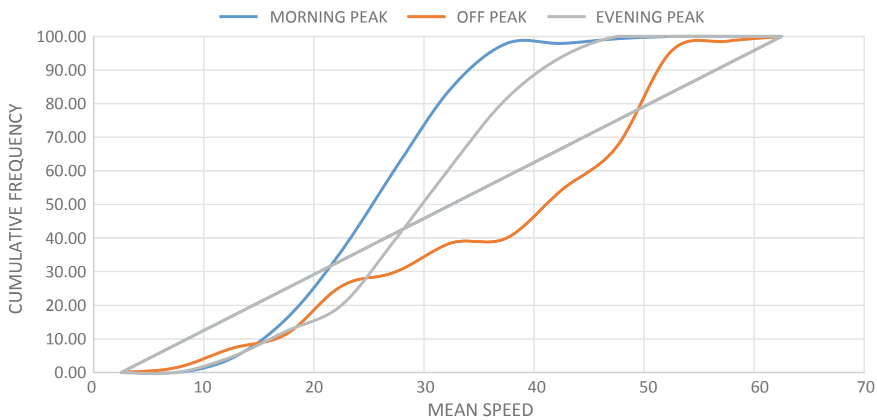

Fig. 12 2W: Athwa Gate–Police Parade Ground

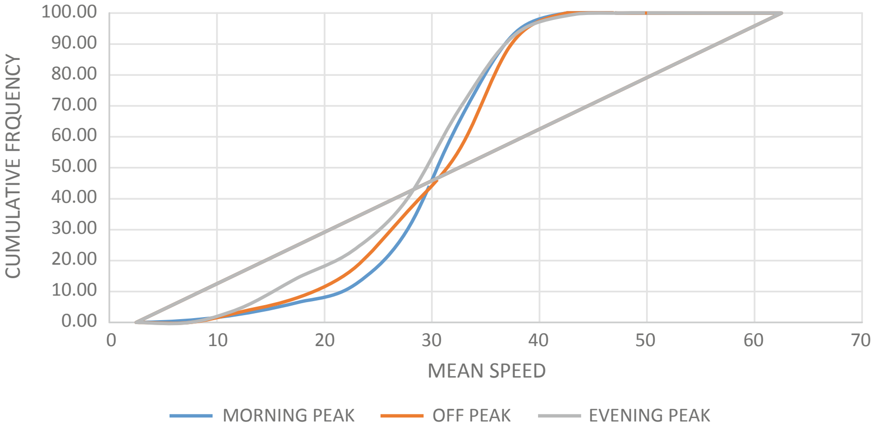

Fig. 13 3W: Kargil Chowk-Rahul Raj

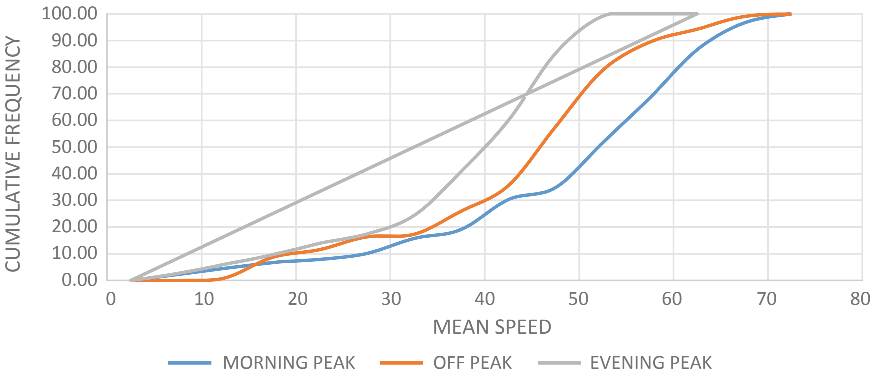

Fig. 14 Small Car: Kargil Chowk-Rahul Raj

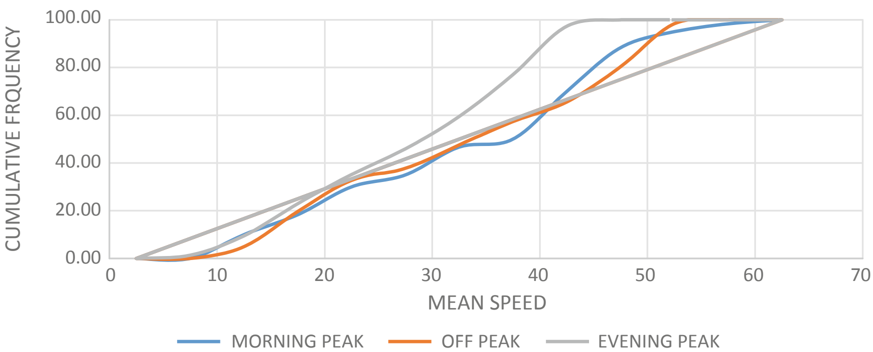

Fig. 15 Big Car: Sargam-SVNIT

3.4.3 Analysis of Speed Data

The speed of different vehicle categories over different mid-block sections was analysed and the excess speed as compared to the posted speed limit was also determined.

Here, excess speed in terms of variation of 85th percentile speed from speed limit is determined and it was found that for most of the stretches and for most of the time, the 85th percentile speed was way below the posted speed limit. Only Rahul Raj-Kargil Cowk and Kargil Chowk-SVNIT have shown some examples of over speeding because of better LOS conditions over there. For remaining stretches, less road width, more frontage access has led to vehicle speed less than posted speed limits.

Such low speeds are not called as good if we talk about operational point of view. But from safety point of view, such values are considered best (Tables 2, 3, 4).

3.4.4 Graphical Comparison of V-Box Speed Data for Different Stretches for Different Time Periods

Some representative samples of different categories of vehicles for a given stretch, for a given period of time, were also plotted on the same graph to analyse the trend of variation of speed among different vehicle categories. Some of them are shown below (Figs. 16, 17 and 18).

3.4.5 V-Box Data Versus Data from Volume Count

The speed data obtained from V-Box survey was compared with the speed data observed in the field through video graphic survey. In total, 30 samples for each category of vehicles were taken for collecting GPS speed data through V-Box. Ten samples each were taken for each duration.

Also, speed data was collected by marking 100 m stretch within each mid-block section for each category. Now, average of both speed data has been taken and statistical test ANOVA has been applied between the data to check if there is any significant difference between the data or not. Finally, it has been found that the speed data obtained from GPS or V-Box and the field data of speed obtained through video graphic survey are almost the same.

Table 2 Morning peak

Segment	Category	Morning peak				
		Speed limit	Mean speed	S D	85th percentile speed	Excess speed
AG-PP	2W	50	27.52	7.22	34.88	−15.12
	3W	35	29.08	5.67	34.36	−0.64
	SC	60	28.91	9.03	38.31	−21.69
	BC	60	28.33	9.24	38.69	−21.31
	HV	40	20.96	7.48	28.88	−11.12
PP-PR	2W	50	25.64	9.47	33.63	−16.37
	3W	35	24.94	6.92	32.65	−2.35
	SC	60	25.65	8.15	34.89	−25.11
	BC	60	26.98	9.03	36.5	−23.5
	HV	40	20.28	7.41	28.64	−11.36
PR-SR	2W	50	32.29	16.8	52.57	2.57
	3W	35	26.72	8.86	37.8	2.8
	SC	60	20.05	7.75	28.7	−31.3
	BC	60	23.42	7.52	32.22	−27.78
	HV	40	16.36	6.64	22.79	−17.21
SR-SV	2W	50	40.06	12.8	53.33	3.33
	3W	35	35.19	8.1	41.46	6.46
	SC	60	35.57	14.3	52.59	−7.41
	BC	60	35.34	13.8	46.79	−13.21
	HV	40	25.61	9.89	35.48	−4.52
SV-KC	2W	50	33.62	11.8	46.74	−3.26
	3W	35	29.7	9.44	39.95	4.95
	SC	60	40.6	15.6	58.45	−1.55
	BC	60	40.13	14	50.03	−9.97
KC-RR	2W	50	42.03	10.3	50.85	0.85
	3W	35	31.95	6.65	38.49	3.49
	SC	60	50.75	15.3	64.29	4.29
	BC	60	49.59	15.7	65.49	5.49

Table 3 Off-peak

Segment	Category	Off-peak				
		Speed limit	Mean speed	S D	85th percentile speed	Excess speed
AG-PP	2W	50	39.34	14.1	53.38	3.38
	3W	35	28.61	6.1	35.26	0.26
	SC	60	26.6	9.75	37.62	−22.38
	BC	60	28.68	7.92	36.36	−23.64
	HV	40	20.29	7.14	28.1	−11.9
PP-PR	2W	50	27.23	12	37.61	−12.39
	3W	35	27.03	7.12	33.66	−1.34
	SC	60	29.92	10.5	40.58	−19.42
	BC	60	29.83	8.35	39.28	−20.72
	HV	40	19.32	6.58	26.42	−13.58
PR-SR	2W	50	30.66	6.54	37.43	−12.57
	3W	35	28.99	5.84	35.19	0.19
	SC	60	33.97	16.6	50.26	−9.74
	BC	60	24.45	6.95	31.05	−28.95
	HV	40	20.14	7.77	30.57	−9.43
SR-SV	2W	50	30.81	9.69	41.24	−8.76
	3W	35	30.77	5.66	35.69	0.69
	SC	60	45.52	10.7	53.75	−6.25
	BC	60	34.9	13.6	50.6	−9.4
	HV	40	24.9	11.5	37.5	−2.5

(continued)

Table 3 (continued)

Segment	Category	Off-peak				
		Speed limit	Mean speed	S D	85th percentile speed	Excess speed
SV-KC	2W	50	30.64	14	45.22	−4.78
	3W	35	30.21	8.66	39.22	4.22
	SC	60	41.76	13.5	51.49	−8.51
	BC	60	37.31	13.7	52.18	−7.82
KC-RR	2W	50	40.68	8.75	49.43	−0.57
	3W	35	32.12	7.43	39.25	4.25
	SC	60	45.47	13.3	56.86	−3.14
	BC	60	44.11	15	57.18	−2.82

Table 4 Evening peak

Segment	Category	Evening peak				
		Speed limit	Mean speed	S D	85th percentile speed	Excess speed
AG-PP	2W	50	31.84	8.91	40.5	−9.5
	3W	35	28.92	6.14	34.37	−0.63
	SC	60	26.82	8.83	35.89	−24.11
	BC	60	28.3	8.65	36.67	−23.33
	HV	40	23.97	8.84	33.21	−6.79
PP-PR	2W	50	29.81	10.5	41.33	−8.67
	3W	35	25.67	7.5	34.05	−0.95
	SC	60	28.03	9.76	37.44	−22.56
	BC	60	27.08	8.62	36.55	−23.45
	HV	40	22.07	7.69	29.12	−10.88
PR-SR	2W	50	34.43	17.5	55.85	5.85
	3W	35	29.97	9.33	39.65	4.65
	SC	60	22.4	9.12	32.61	−27.39
	BC	60	22.04	7.36	30.46	−29.54
	HV	40	21.37	8.43	29.44	−10.56
SR-SV	2W	50	44.34	13.3	58.6	8.6
	3W	35	33.18	6.97	39.99	4.99
	SC	60	33.26	11	45.36	−14.64
	BC	60	30.26	10.7	41.78	−18.22
	HV	40	26.18	10.9	37.79	−2.21

(continued)

Table 4 (continued)

Segment	Category	Evening peak				
		Speed limit	Mean speed	S D	85th percentile speed	Excess speed
SV-KC	2W	50	31.98	11	44.98	−5.02
	3W	35	29.74	7.96	37.48	2.48
	SC	60	36.01	10.9	46.22	−13.78
	BC	60	36.3	12.2	47.85	−12.15
KC-RR	2W	50	42.79	10.4	50.77	0.77
	3W	35	30.5	7.95	37.75	2.75
	SC	60	39.68	11.5	49.65	−10.35
	BC	60	41.63	10.2	50.93	−9.07

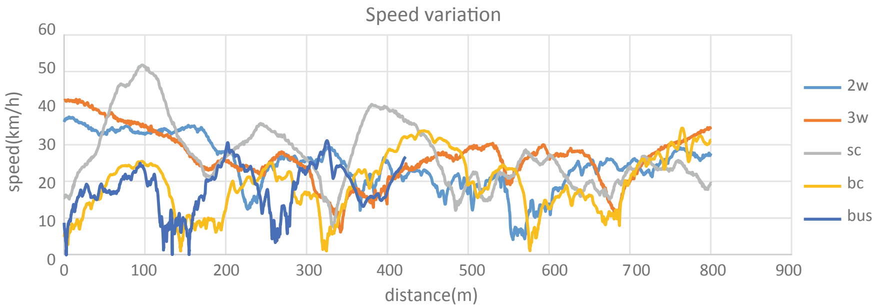

Fig. 16 Police Parade–Parle Point, Morning Peak

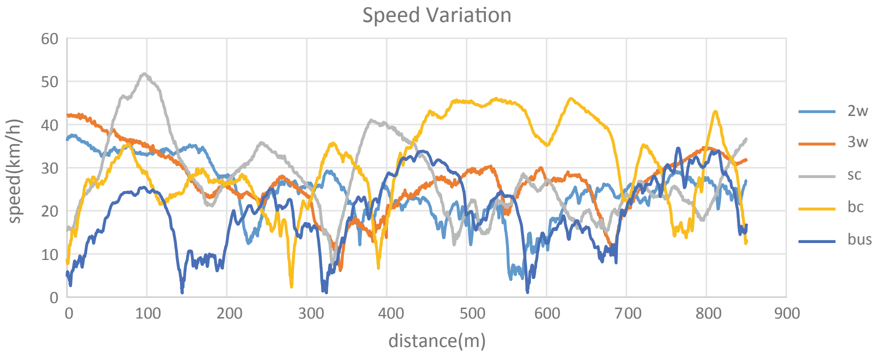

Fig. 17 Athwa Gate–Police Parade Ground, Evening Peak

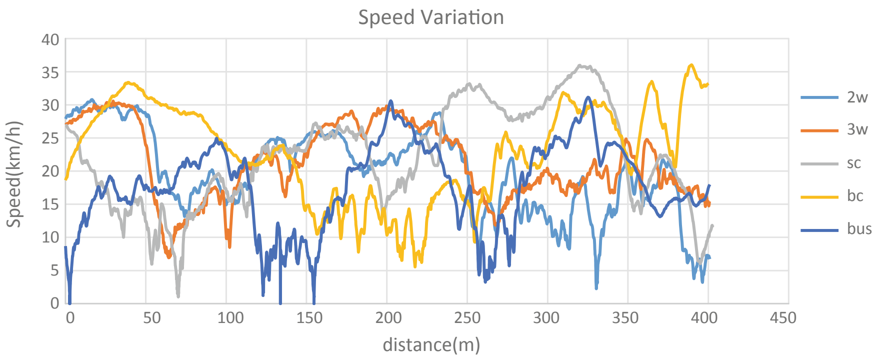

Fig. 18 Parle Point–Sargam, Morning Peak

	Time						
	Duration	Morning-peak		Evening-peak		Off-peak mode	
		V-Box	V field	V-Box	V field	V-Box	V field
2W	AG-PP	30.15	32.1	28.25	29.3	38.33	40.2
	PP-PR	33.42	35.02	28.65	29.66	33.62	36.6
	PR-SR	31.09	30.22	32.03	30.19	36.85	35.15
	SR-SV	38.58	40.12	40.41	38.22	49.18	53.7
	SV-KC	41.23	43.71	38.69	40.61	48.26	50.01
	KC-RR	46.79	47.87	48.7	47.2	45.52	45.57
	Average	36.87667	38.17333	36.12167	35.86333	41.96	43.53833
	Variance	42.16183	47.71079	63.54626	54.07003	42.66732	55.78374
	p-value	0.744520481		0.954621309		0.704971773	
3W	AG-PP	28.63	30.23	23.56	25.6	30.11	30.32
	PP-PR	30.88	29.45	29.01	30.22	31.21	32.1
	PR-SR	24.8	23.22	22.15	23.06	28.28	30.22
	SR-SV	35.75	36.17	34.2	33.21	37.85	39.6
	SV-KC	35.04	36.22	34.32	35.23	39.05	38.56
	KC-RR	28.88	29.32	27.03	27.77	28.22	27.46
	Average	30.66333	30.76833	28.37833	29.18167	32.45333	33.04333
	Variance	17.35395	24.0003	26.68414	21.23678	23.00299	24.17495
	p-value	0.968884404		0.782013651		0.837576091	
SC	AG-PP	30.66	32.53	28.29	30.36	34.65	34.19
	PP-PR	31.26	31.92	30.35	32.26	35.56	36.98

(continued)

(continued)

	Time						
	Duration	Morning-peak		Evening-peak		Off-peak mode	
		V-Box	V field	V-Box	V field	V-Box	V field
	PR-SR	30.25	30.45	29.35	30.22	34.56	35.62
	SR-SV	38.87	39.65	36.78	37.88	47.69	49.16
	SV-KC	47.29	50.72	39.59	40.73	51.12	52.42
	KC-RR	58.56	58.41	45.53	45.69	48.69	48.28
	Average	39.48167	40.61333	34.98167	36.19	42.045	42.775
	Variance	131.1937	132.8003	46.7369	39.75848	62.22883	64.51551
	p-value	0.867935178		0.756843467		0.876964227	
BC	AG-PP	30.72	32.79	28.52	31	38.62	39.12
	PP-PR	25.79	25.42	25.9	27.21	30.36	31.76
	PR-SR	22.51	24.71	24.12	25.44	30.25	31.33
	SR-SV	41.35	43.22	39.56	40.23	43.55	43.21
	SV-KC	45.79	48.73	37.97	40.33	44.34	46.22
	KC-RR	55.55	55.64	45.37	45.37	46.28	47.16
	Average	38.198	39.544	33.57333	34.93	38.908	39.81943
	Variance	192.1843	194.1279	73.62279	66.2694	42.25261	40.64581
	p-value	0.882087603		0.784464828		0.795620151	
BUS	AG-PP	26.98	29.72	25.19	26.57	32.33	34.61
	PP-PR	28.65	30.26	28.21	29.31	32.56	33.12
	PR-SR	25.16	25.2	22.12	22.96	30.26	30.99
	SR-SV	26.63	28.95	25.56	25.99	35.56	34.09

(continued)

(continued)

	Time						
	Duration	Morning-peak		Evening-peak		Off-peak mode	
		V-Box	V field	V-Box	V field	V-Box	V field
	Average	26.855	28.5325	25.27	26.2075	32.6775	33.2025
	Variance	2.053767	5.224758	6.218867	6.783492	4.762558	2.556892
	p-value	0.260040817		0.621694222		0.711333977	

4 Results and Conclusions

It was found that among all six mid-block sections, the Rahul Raj-Kargil Chowk, Kargil Chowk-SVNIT and SVNIT-Sargam possess comparatively better level of service with respect to other stretches. It is so because the land use pattern along these stretches is not very much congested and commercialise. Also, availability of service lane, cycle lane, six-lane roads which are not available in other stretch enhances user experience on these stretches.

If we talk about temporal variation, then speed variations were more in morning peak and evening peak hours as compared to off-peak hours. But it was also observed that since the speed of three-wheelers was restricted by administration, there was not very much difference in the speed values for different times of the day. Although the cumulative speed plots for small cars and big cars clearly represent the effect of peak and non-peak hours on vehicle speed. Two-wheelers also can manoeuvre easily in any time of the day owing to their small size and easy navigation.

Excess speed in terms of variation of 85th percentile speed from speed limit was also determined and it was found that for most of the stretches and for most of the time, the 85th percentile speed was way below the posted speed limit. Only Rahul Raj-Kargil Chowk and Kargil Chowk-SVNIT has shown some examples of over speeding because of better LOS conditions over there. For remaining stretches, less road width, more frontage access has led to vehicle speed less than posted speed limits.

On plotting the speed variation of different vehicles on same speed-distance graph, it was found that the total variation was minimum for two-wheeler category owing to their easy manoeuverability even in more traffic. On the other hand, the speed variation of bus was maximum.

ANOVA of data obtained from V-Box and data obtained from videographic survey indicate that there is not much difference between the two data.

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Transportation Planning

Evaluation of Noise Level in and Around Railway Platform in Surat City

Minakshi Vaghani, Misaq Ahmad Muradi, and Punit Limbani

Abstract The constantly increasing market demand in transport leads to a less peaceful transport system affecting many citizens both during the day and overnight. Noise is seen as the most important environment issue for those living in the neighbourhood of a railway line. According to data obtained from Ministry of Indian Railways, 11,000 trains run every day, of which 7,000 are passenger trains (Graphical representation of Noise levels in and around Mumbai during 2011). These trains have the potential to generate noise especially while stoppage in or nearby railway stations. Attempts were made to evaluate the noise levels in and around the railway platform in the Surat city. Based on levels of exposure to noise sources, total of 06 study locations were identified. Noise levels on weekday and weekend were measured in three time shifts day, evening and night at the predefined study spots. It is observed from study that the majority of the noise levels exceed the limit stipulated by Central Pollution Control Board at all locations. For better representation of results, box plots were illustrated. The research was carried out to quantify the noise potential nearby Surat railway station and the same can be utilised in order to mitigate the issue. Attempts should be made to reduce noise levels for humans exposed to area nearby railway station. Noise barriers should be provided to absorb noise coming from movement of trains, and some safety aids like earplugs have to be used by those people who are continuously exposed to higher and noisy area. Regular maintenance of railway track solves the issue up to certain extent.

Keywords Noise · Train · Peak · Horn · Passengers

1 Introduction

Noise is derived from the Latin word 'nausea' implying 'unwanted sound' or 'sound that is loud, unpleasant or unexpected'. The noise originates from human activities, especially the urbanisation and the development of transport and industry. The

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urban population is much more affected by this noise nuisance. However, small town/villages along side roads or industries are also victim of this problem. The definition given by EPA of noise is “unwanted or disturbing sound”. Sound becomes unwanted when it either interferes with normal activities such as sleeping, conversation, or disrupts or diminishes one’s quality of life (Garg et al. 2016). The fact that you cannot see, taste or smell it may help explain why it has not received as much attention as other types of pollution, such as air pollution or water pollution. The air around us is constantly filled with sounds, yet most of us would probably not say we are surrounded by noise. The persistent and escalating sources of sound can often be considered an annoyance. This ‘annoyance’ can have major consequences, primarily to one’s overall health (Agarwal 2005).

Though noise nuisance is subtle and consistent killer, yet very poor attentions have been drawn to address this issue. It along with other types of pollution has become a hazard to the quality of life. It may cause hypertension, disrupt sleep and/or hinder cognitive development in children. The effects of excessive noise could be so severe that either there is a permanent loss of memory or a psychiatric disorder. Thus, there are many adverse effects of excessive noise or sudden exposure to noise (Kudesia and Tiwari 1994; Singh and Davar 2014).

Noise pollution has become a serious concern globally. Every nation is concerned towards the health effects of noise emitted from the expanding number of vehicles moving on the roads. It is thus obligatory to adopt long-term noise monitoring strategies to monitor the noise levels and planning of suitable noise abatement measures for noise control (Graphical representation of Noise levels in and around Mumbai during 2011). The European Environmental Noise Directive 2002/49/CE (2002) require that the noise maps should present the noise levels expressed in harmonised indicators: day–evening–night level, $L_{\mathrm{den},}$ and night equivalent level, L_{night} (Can et al. 2016). Also, there have been few studies focussed on sampling strategies for the measurements to be conducted as a substitute for the long-term noise monitoring. Thus, it is imperative to adopt continuous long-term noise monitoring for ascertaining the magnitude of ambient noise levels and comparison with the established noise limits (Graphical representation of Noise levels in and around Mumbai during 2011; Hueso et al. 2017; Mioduszewski et al. 2011; Pultznerová and Ižvolt 2014; Ragettli et al. 2015; Reza and Rahman 2016; Tumavičė et al. 2016). Apart a validated road traffic noise model useful in conducting Environmental Impact Assessment studies in respect of noise similar to that used in developed nations is required in Indian conditions. The Central Pollution Control Board (CPCB), New Delhi, has taken many initiatives and carried out numerous studies for monitoring the ambient sound levels at noise hot spots in metropolitan cities like Delhi city for the implementation of suitable measures for noise mitigation (Protocol for ambient level noise monitoring).

Fig. 1 Monitoring stations in and around Surat railway platform

2 Study Area

Surat is the fourth fastest-growing city in the world according to city mayors' foundation in 2016. It is located at 21° 10′ 12.86″ N, 72° 49′ 51.81″ E. Total area of the city is 326.515 Sq Km and population as per 2011 census is 4,467,797 persons. Surat is famous for textile industry and diamond. Surat railway station is situated in the eastern part of the city and is connected with major cities like Mumbai, Jaipur, Delhi and other cities in the country. It is one of the busiest railway stations in Gujarat and handles approximately one lakh passengers daily. To evaluate noise level due to the moving of trains and other modes of transportation near railway station, six monitoring stations were selected in and around railway platform in city. Study spots were selected based on exposure level of noise, frequency of trains, traffic volume and intensities, traffic peak hours and various contributors to nearby area. Locations of monitoring stations are shown in Fig. 1.

3 Methodolgy

The study was conducted at predefined monitoring stations using sound level meter. Considering the nature of train traffic and vehicular traffic, monitoring was scheduled on weekday and weekend days in three time shifts day, evening and night specified by CPCB (National Ambient Noise Monitoring Network 2011; Protocol for ambient level noise monitoring). The standards for ambient noise are mentioned in Table 1 (The noise pollution (regulation and control) rules 2000).

Table 1 Ambient noise standards

Area code	Category of area/zone	Limits in dB	
		Daytime	Night-time
(A)	Industrial area	75	70
(B)	Commercial area	65	55
(C)	Residential area	55	45
(D)	Silence zone	50	40

Fig. 2 Sound level meter and GPS receiver

3.1 Instruments Used

Sound level meter SL-4032SD has been used in this study to measure the level of sound. Sampling time can be set from one second to 3600 s and measured sound levels can be saved in SD memory card. GPS receiver has been used in this study to find out the latitude and longitude of study spots. Sound level meter and GPS receiver are shown in Fig. 2.

3.2 Monitoring Techniques

Various techniques are available for measurement of sound level. In this study, portable sound level meter with data logger was used. Here, the sound level was measured in terms of L_{eq}. Each location was monitored for 30 min at an interval of 05 s. L_{eq} of 05 s was obtained by considering sound levels of every second at each location.

Considering the traffic pattern of trains and other vehicles, one weekday (Wednesday) and one weekend (Saturday) were selected for monitoring the noise levels at selected study locations. Noise levels were measured on weekday and

Table 2 Peak values of noise for monitoring stations near railway station

Sr. no.	Location	Geographical coordinate	Peak value of noise				Remarks
			Weekday		Weekend		
			Min	Max	Min	Max	
1	1	21°12′16.29″N 72°50′24.91″E	60.7	120.4	66.3	122.4	Near city bus stand
2	2	21°12′21.95″N 72°50′25.35″E	57.1	98.5	56.9	113.5	Parking
3	3	21°12′31.67″N 72°50′25.18″E	52.3	128.1	51.9	134.7	North end of the platform
4	4	21°12′18.55″N 72°50′30.04″E	54.5	96.7	55.1	94.9	Backside parking
5	5	21°12′14.14″N 72°50′31.38″E	56.6	115	61.1	104.2	T Junction of road
6	6	21°12′12.26″N 72°50′28.72″E	52	112.2	51.6	114.5	South end of the platform

weekend in three times shifts a day (6 AM to 2:00 PM), evening (2:00 PM to 10:00 PM) and night (10:00 PM to 6:00 AM). Geographical locations of monitoring stations were traced by using GPS receiver and the same is represented in Table 2. Peak value of noise at the study location was noticed for weekday and weekend days. The collected detail is presented in Table 2.

4 Results and Discussion

Noise levels were measured at total six monitoring locations for 30 min. For better representation of results obtained, the results are analysed in two categories like location wise evaluation and statistical analysis of noise levels using a box plot.

4.1 Location Wise Analysis

Considering the contributors and their distance from railway line, a total of six monitoring stations were located. Two locations were selected at either ends of the railway platform (location 3 and 6). Two sites were finalised in the parking areas including two-wheelers parking and Eastside parking (location 2 and 4, respectively). Location 1 was studied for evaluating the influence of urban vehicular traffic and commuters and other surrounding commercial activities. Noise levels at location 5 were measured to study the exclusive effect of urban traffic at *T*-junctions which is 120 m away from the railway platform.

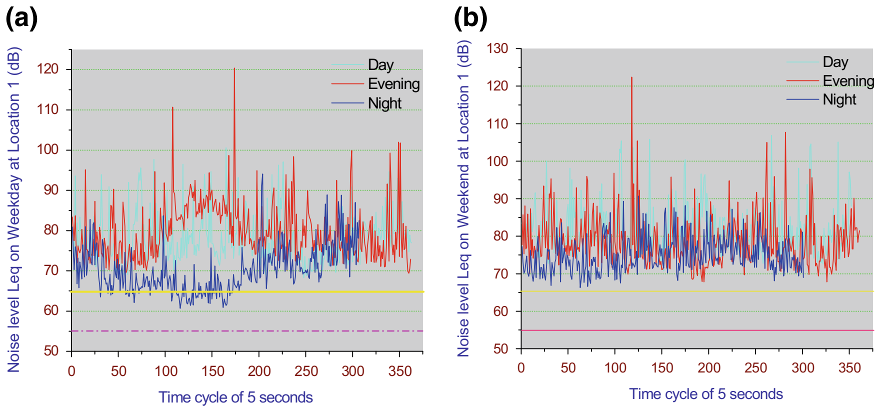

Fig. 3 **a** Noise level L_{eq} on weekday at location 1. **b** Noise level L_{eq} on weekend at location 1

First monitoring spot is located in the city bus stop where approximately 354 buses are continuously conveying passengers who are travelling to different direction of the city. Auto-rickshaws and taxis are also seeking for passengers nearby bus stop. This monitoring spot is near the main road where noise due to social, religious and political activities was observed at the time of monitoring. Other contributors to the noise level generated at location 1 are movement and conversation of local passengers, train commuters, hawkers and vehicles like bikes, cars, auto-rickshaws and buses. Noise levels for location one in weekday and weekend are shown in Fig. 3a and b, respectively.

It is revealed from Fig. 3a, b that comparatively higher noise level is observed during the evening time on weekday and weekend as evening hours were observed to be peak traffic hours for that location. It is interesting to note that low traffic was observed during day time. Hence, noise levels have comparatively low fluctuations during daytime in weekday as well as in weekend. It can be seen that fluctuations of noise levels during night-time in a weekday is mainly due to the frequency of trains and related variables, number of passengers and auto-rickshaws. It is stated that all the measured noise level during the daytime, evening time and night-time are exceeding the noise limits stipulated by CPCB. Spike (120 dB) observed in noise level on location 1 during evening time is solely due to horn blowing of bus.

Second monitoring station is located at two-wheelers parking nearby railway platform and considerable movement of two-wheelers and auto-rickshaws for seeking passengers was observed during daytime and night-time. In this case, the conversation of local passengers passed by, daily commuters, and vehicles like auto-rickshaws, bikes and trains are the main contributors to the noise level generated at this location. Noise levels for location two in weekday and weekend are shown in Fig. 4a, b.

Figure 4a, b reveals that comparative high level of sound was observed during both evening time and night-time in weekday and weekend. High frequency of trains, more number of passengers and vehicles like two-wheelers and auto-rickshaws were the

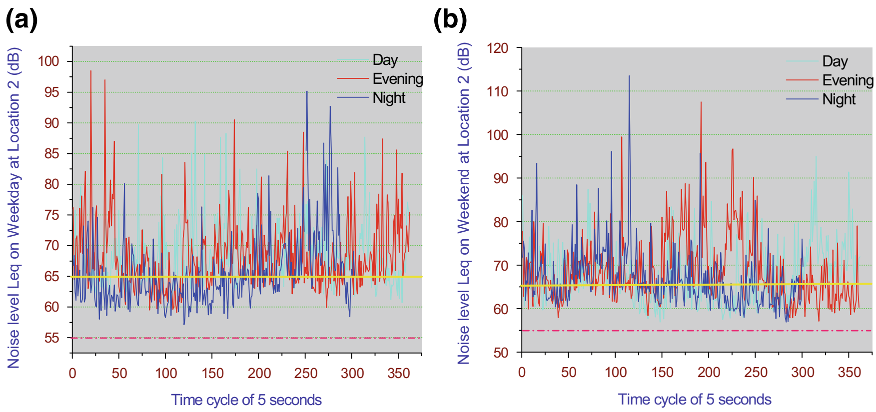

Fig. 4 **a** Noise level L_{eq} on weekday at location 2. **b** Noise level L_{eq} on weekend at location 2

main contributor to noise generated during evening time and night-time. It has been observed that all measured noise levels during evening and night time are exceeding the noise limits specified by CPCB. Majority of observations during daytime in weekday and weekend are higher than the noise standards, low frequency of trains and other dependent variables (passengers and vehicle) were the main contributors to the generation of noise. Spike observed in noise level at location 2 during evening time in weekday and night time in weekend is due to blowing of horn by auto-rickshaws and two-wheeler driver.

Third monitoring spot was located in north end of the railway platform, towards Vadodara and considerable movement of trains were observed during all time shifts. This location is at the North end of railway platform where the train is leaving and entering the railway platform. So, blowing horn is quite frequent and the same is reflected in the peak levels of noise. In this case, noise generated from trains, loading–unloading of goods and conversation of porters is the main contributors to the said location. Noise levels for location 3 in weekday and weekend are shown in Fig. 5a, b. Highest peak of 134 dB was observed when horns were blown by train drivers (running in the opposite direction). Frequent blowing of horn by train passed by is reflected in peaks observed in Fig. 5a, b.

It can be seen from Fig. 5a, b that comparatively high noise levels were observed during daytime and evening time for a weekday. Majority of noise levels during daytime and evening time in weekend were exceeding the limits. Almost all noise levels during night-time in weekday and majority of noise levels for weekday exceed the limits stipulated by CPCB. Schedule of the train and cumulative effect of movement of carts and conversation of passengers were main contributors for the generation of noise in this location. Spike observed in noise level at this location during daytime in weekday and weekend is due to blowing of horn by the train driver.

Location 4 is monitored at Eastside parking of railway station. Continuous noise is obtained from train passed by at the location 4. Movement of two-three and

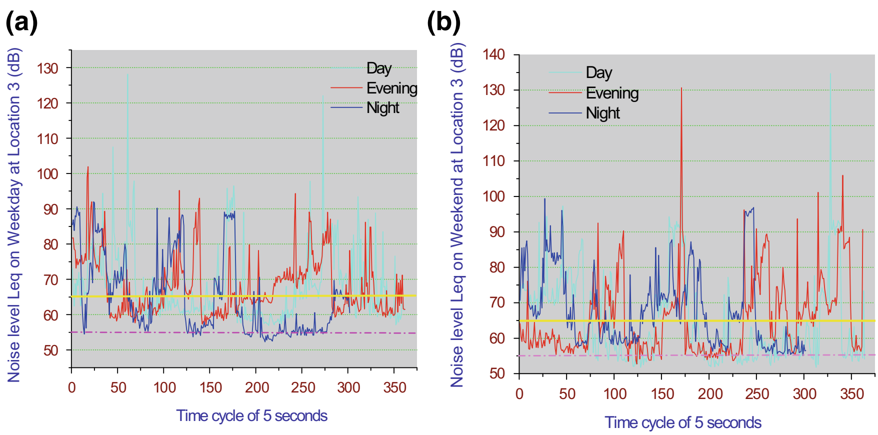

Fig. 5 **a** Noise level L_{eq} on weekday at location 3. **b** Noise level L_{eq} on weekend at location 3

four-wheelers for parking area and conversation of daily commuters and visitors. Sounds generated from speakers of the religious place are frequently observed during evening.

It can clearly be seen from Fig. 6a, b that more peak noise levels were obtained during daytime and evening time on weekday as well as weekend. It is interesting to note that all the measured noise levels during night-time on weekday as well as weekend are exceeding the noise limits stipulated by CPCB (National Ambient Noise Monitoring Network 2011). Noise generated from passing of trains, movement of two-wheelers as well as four-wheelers was main contributors to generating of noise. Spike observed in noise level at this location during daytime and evening time is due to blowing of horn by two-wheeler driver near the parking area.

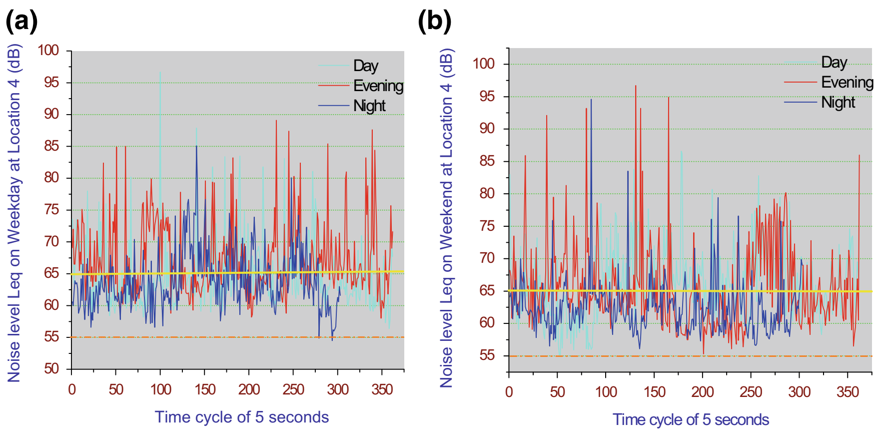

Fig. 6 **a** Noise level L_{eq} on weekday at location 4. **b** Noise level L_{eq} on weekend at location 4

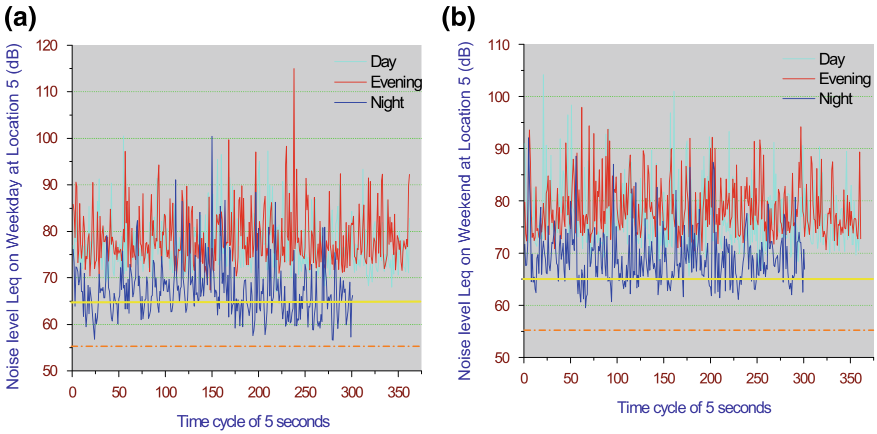

Fig. 7 **a** Noise level L_{eq} on weekday at location 5. **b** Noise level L_{eq} on weekend at location 5

Attempts were employed to study the contributions of urban traffic movement near railway station (location 5) so it is selected to evaluate noise levels at traffic junction nearly 120 m away from Surat railway station (Eastside). Here, vehicular noise is the main contributor. Loudspeakers for religious activities from nearby region are contributing to peak noise level during evening time. Other peak values are obtained because of horn blown by vehicle users.

Location 6 was selected at the South end of the railway platform where the minimum movement of passengers and goods was observed. Horn blowing of train and its movement are the sole contributors to noise levels obtained. Depending on the schedule of train, peak was obtained during daytime and night-time due to blowing of horn. There are more spikes in noise levels for daytime in weekend at this location (Figs. 7 and 8).

4.2 Statistical Analysis of Noise Levels Using Box Plot

In order to get better output of the research, frequency of noise levels should be evaluated at various locations with its peak values obtained. Box charts were plotted for representing the frequency of noise levels at various locations for three shifts on weekend and weekdays. The parameters evaluated for the measured noise levels are the median value and the box length or the distance between the first and third quartiles. This interquartile represents a quantity of the 50% dispersion of the noise levels measured. Outliers show the peak/spike observed during monitoring of noise levels at a particular location.

Box plot of noise levels during daytime on weekday is shown in Fig. 9. Majority of noise levels for all locations exceed the limit. Comparatively higher noise levels were observed at location 3 because of train entering and leaving the platform. Effect of

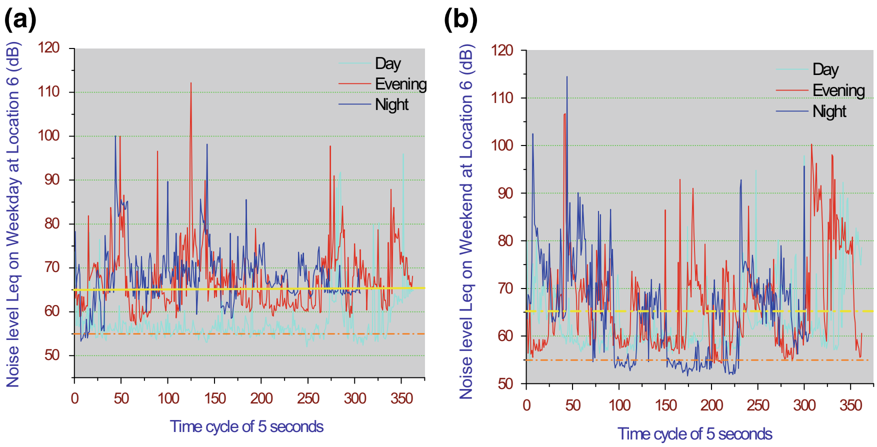

Fig. 8 **a** Noise level L_{eq} on weekday at location 6. **b** Noise level L_{eq} on weekend at location 6

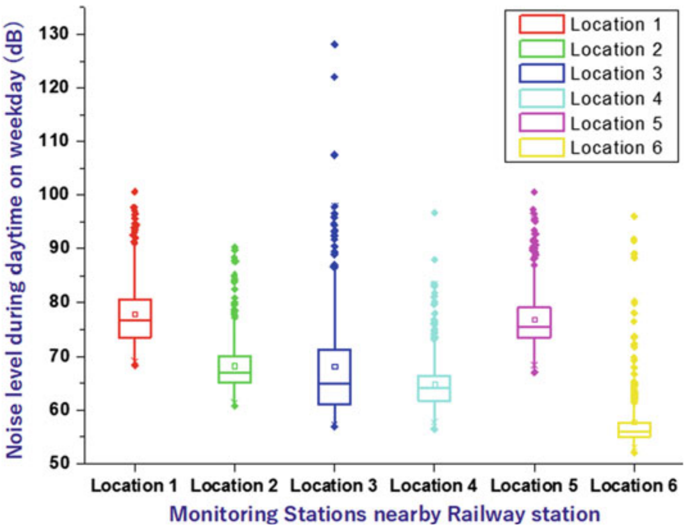

Fig. 9 Box plot of noise level during daytime on weekday

horn blowing is reflected in outliners of location 3. Due to low traffic during daytime in location 6, comparatively low noise levels were obtained. Majority of the noise levels obtained exceed the limit prescribed by CPCB.

Figure 10 illustrates the box plot of noise levels during evening time on weekday at all the monitoring stations. From the figure, it is reflected that 25% of the measured noise levels at location 1 exceed the level of 83 dB which is much more than the permissible limits for evening shift. Rally passed by and horn blown by vehicle users contributed to peak noise levels which were shown by outliers. In case of location

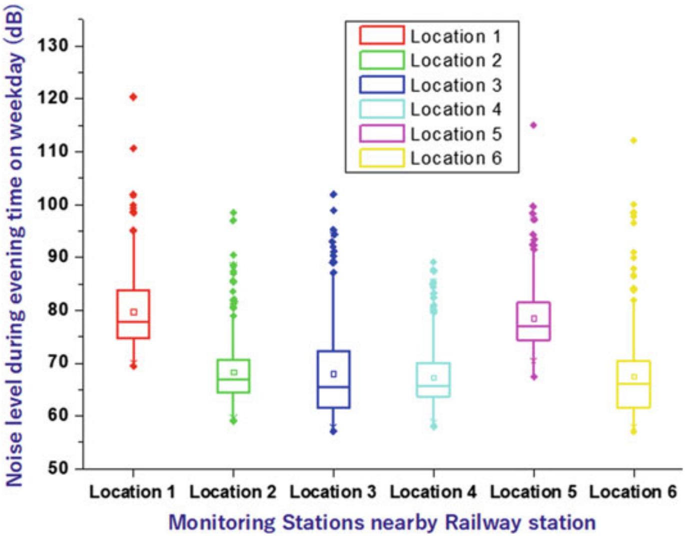

Fig. 10 Box plot of noise level during evening time on weekday

2, length of box plot is less and comparatively low noise levels were obtained as it had effect of parking area only. However, outlier in the box plot shows the existence of peak noise levels. 50% of the measured noise levels at location 3 exceed the permissible limits and value of 65 dB. Location 5 shows that the 25% of the noise levels are greater than 80 dB which is quite higher than the standards. Outlier for this box plot shows the comparatively higher peak values during evening time due to peak traffic hours. Location 6 was far from other urban activities and has the sole source of noise from train only. Frequency of train and its horn blowing is reflected in the outlier of the box plot for location 6.

Figure 11 reveals that comparatively higher noise levels were obtained in all locations during night-time on weekday. Effect of urban traffic is reflected in outliners at location 1 and 5. Higher noise levels at location 3 are due to cumulative effect from moving of trains, conversation of commuters and loading-boarding of trains while train frequency is governing parameter in higher noise generation at location 6.25% of measured noise levels at location 3 are observed to be greater than 82 dB.

Figure 12 illustrates box plot of noise levels during daytime on weekend. Cumulative effect of urban traffic, local commuters, social and commercial activities was main contributors to comparatively higher noise generation at location 1. Comparatively, low noise levels were obtained at location 2 and 4 due to the low movement of vehicles like two- and three-wheelers.

Box plot of noise levels obtained during evening time on weekend is shown in Fig. 13. Comparative higher noise levels were observed at location 1 and 5 as evening time was observed to be peak traffic hours of the day. 25% of the measured noise levels were greater than 78 dB at location 1. Effect of horn blowing is reflected in box chart plotted for location 3. Figure 14 illustrates box plot of noise levels

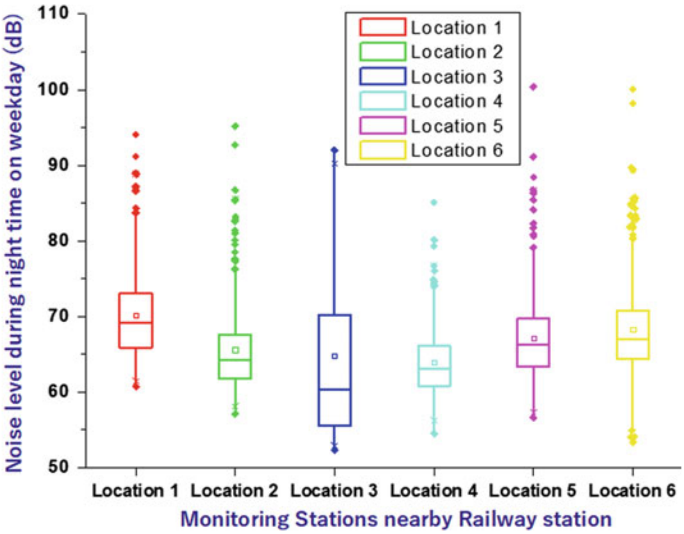

Fig. 11 Box plot of noise level during night-time on weekday

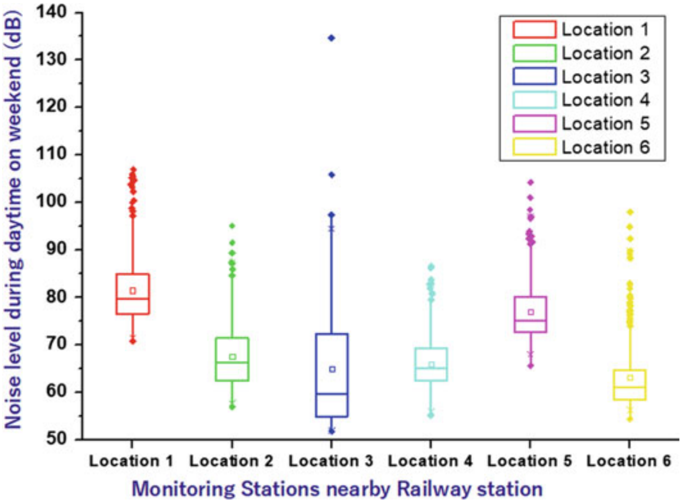

Fig. 12 Box plot of noise level during daytime on weekend

during night-time on weekend. Comparatively higher noise levels were obtained for location 3 and 6. Comparatively higher noise level at location 2 and 4 was obtained due to cumulative noise of two-wheeler, passengers and commuters. It is interesting to note that 25% of the measured noise levels at location 6 were found to be lesser than 55 dB in night-time on weekend as there is rare movement of commuter and

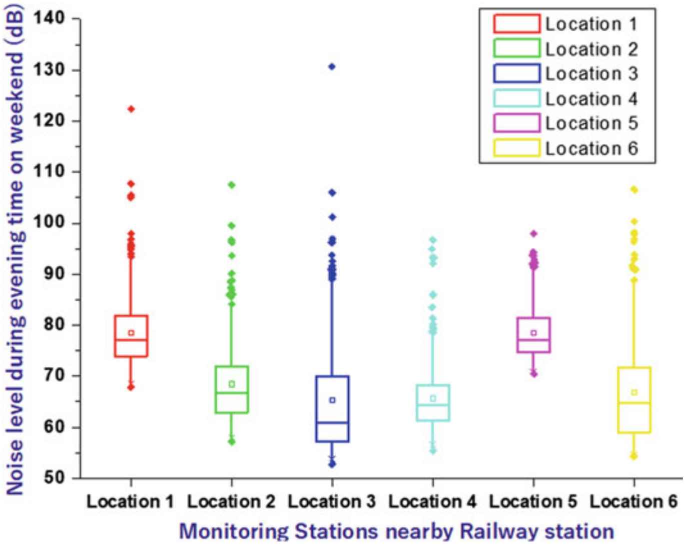

Fig. 13 Box plot of noise level during evening time on weekend

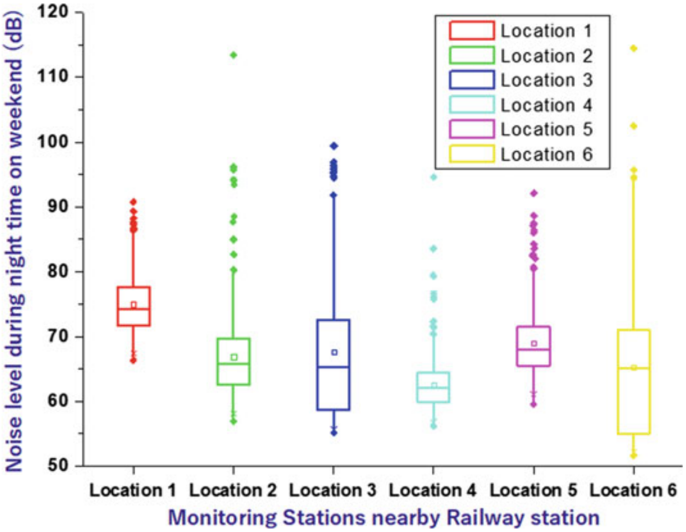

Fig. 14 Box plot of noise level during night-time on weekend

other vehicles during night time at that location. Movement of trains was the sole parameters responsible for higher noise levels obtained.

5 Concluding Remarks

The detailed study was carried out for monitoring noise levels at a total of six monitoring stations nearby Surat railway station. The research consists of monitoring of noise in three shifts (day, evening and night) on weekday and weekend. The responsible factors for the generation of noise were identified and analysed. From this research, the following conclusions were drawn:

1. Frequency of train, speed of train, loads on train (passengers as well as goods), geometrical configuration of platforms, number of total platforms and announcements on platform can also be considered as contributing elements.
2. Blowing of horn from trains and vehicles, sound of applied brakes of train, conversation of commuters, shouting of hawkers and sound from social/religious events are observed to be the major contributors for noise generation around the study area.
3. High noise level is also experienced due to small gaps between two rail joints when the moving wheels hit these gaps.
4. It can be seen that fluctuations of noise levels at location 1 during night-time in weekday is mainly due to the frequency of trains and related variables, number of passengers and auto-rickshaws. It is stated that all the measured noise level during daytime, evening time and night-time are exceeding the noise limits stipulated by CPCB.
5. It has been observed that high peak values of noise levels were obtained for all shifts in two locations 3 and 6 which are at either end of the railway platform. Highest peak of 134 dB was obtained during daytime at location 3 when two trains were passing by at a time. In location 2 and 4, moderate and low level of sound has been observed, and vehicle and human influence were the main causes of moderate level of sound.
6. Location 3 and 6 are considered as areas with exposure of high level of sound. Number of trains passing, horn blowing of trains entering and leaving the platform were the main causes of high levels of sound.
7. It is revealed from the box plot that 25% of the measured noise levels at location 1 on weekday exceed the level of 83 dB which is much more than the permissible limits for evening shift. Almost similar trend is followed on weekend too for this location.
8. To reduce the adverse effect of noise on human being some mitigation measures have to be applied. Awareness of people about the adverse effects of noise is the first and foremost work that has to be done. Most of the people working in and around railway platform do not know about the adverse effect of noise or they take it likely, so awareness programmes will help them.
9. Besides awareness, some safety aids like earplugs have to be considered for those people who are working continuously nearby railway platform.
10. Noise barriers have to be applied in order to prevent the dispersion of noise to nearby residential areas of the railway station. Vibration protection of areas nearby railway platform is another mitigation measure that can be applied.

11. The research was carried out to quantify the noise potential nearby railway station and the same can be addressed in order to mitigate the issue. Attempts should be made to reduce the noise level by adopting constructing and operating measures. This will lead to the reduction of noise levels for humans exposed to the area nearby railway station.

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Parking Study of Station Road, Valsad

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Abstract With increasing number of vehicles and mobility facilities, traffic and parking problems are ubiquitous in urban areas. The approaching road to public places such as railway station and bus stations is typically congested. On station road, public attractions such as shops, hotels, and restaurants generate huge parking demand. The same situation occurs on station road of Valsad city. Valsad is a situated in the southern part of Gujarat state, India. Valsad railway station is connected by India Railway Services with the major cities such as Surat, Ahmedabad in North, and Mumbai in South. In particular peak hour when the train arrives or departs this road observes heavy congestion. Traffic is interrupted due to nearby shop visitors also as vehicles are parked on the road itself due to inadequate parking facilities. Concerning these issues, a proper parking solution is needed. For that purpose, parking study is conducted on station road to find out parking demand and parking characteristics. Parking survey was conducted for three days at station road, Valsad. License-plate method was used in on-street parking survey. This road was divided into 19 different segments for accurate and rapid data collection. In analysis, parking accumulation, volume, load, duration, turnover, and parking index was calculated. Peak parking

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demand for two-wheeler, car, auto-rickshaw, and LCV is 281, 9, 22, and 3, respectively. On the basis of peak parking demand, angled on-street parking can be provided with using odd-even parking system.

Keywords Parking survey · Parking demand · Parking facilities · Parking accumulation

1 Introduction

Valsad city is located in Valsad district of the Indian state of Gujarat. Valsad is situated on Western Railway which connects major cities like Surat, Ahmedabad, Delhi, etc., in North and Mumbai in South. Also, an Industrial Hub-Vapi is located 40 km from Valsad. Valsad is connected with different regions only by bus and railway as the public transportation; railway took majority of movement of passenger. Valsad station road connects railway station to Kalyan baug road junction. Therefore, Valsad station road is the main spinal road of the city.

Increase in the number of private vehicles reflects the increased standards of living and growing demands for free choice of lifestyle and movement. Increased mobility is important but it confronts governing bodies, residents, and experts to a series of interrelated challenges of urban life. Urban mobility includes the movement of people, goods and services in the road infrastructure of city, placement process, and parking. (Abrams Associates Inc. 2011)

Due to inadequate availability of parking on Valsad station road, road observes congestion in peak hours during a day period. Therefore, it needs to be solved at the earliest.

2 Aim and Objectives

2.1 Aim

To reduce parking problems on station road, Valsad.

2.2 Objectives

Objectives are as follows:

- To find out parking problem on station road, Valsad
- To suggest the best suitable solution for existing parking and traffic problems.

3 Study Area

Valsad town has a flat topography, which slopes toward west. The average elevation of the town is 13 m above the sea level. The Arabian Sea lies to its west. Valsad station road lies to west side of railway track, bus station in North, Western Railway Colony in South and SH—67 in West (Figs. 1 and 2).

There are two different roads which connect Valsad railway station to Gujarat State Highway 67. One is under control by local municipal authority while other is a part of Western Railway Colony. Because of restricted traffic flow in Western Railway Colony, majority of traffic flow is passing through the road under municipal control called "Valsad station road."

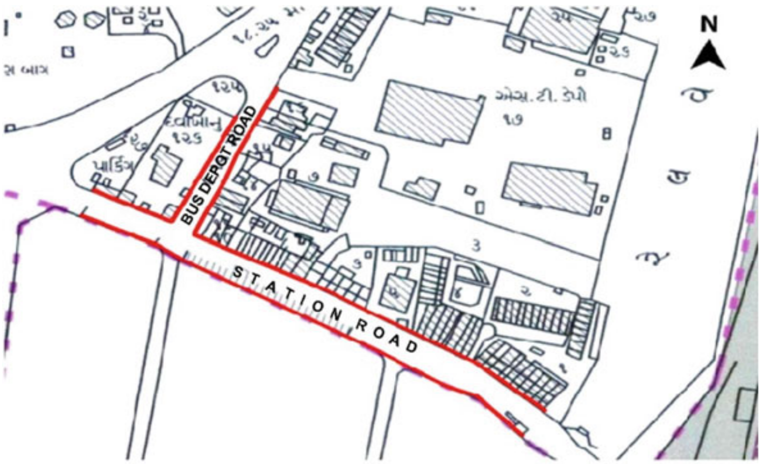


Fig. 1 Indication of survey area in development plan

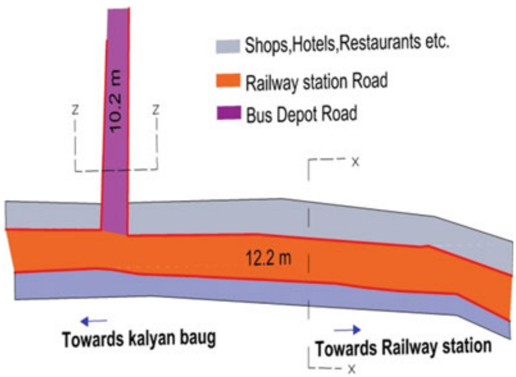


Fig. 2 Plan layout of survey area

Total length of Valsad station road is approx. 500 m. Valsad station road can be divided into two segments. Length of the first segment, from Kalyan baug junction, is 269 m which is maintained by Valsad Municipal Corporation and the second segment with remained length is owned and maintained by Western Railway Corporation. Majority of parking problems occurs on the first segment of Valsad station road denoted as "station road." Further discussion and related study work of this road was carried out.

4 Parking Survey

For the perspective of collecting present parking data, primary survey was conducted on Valsad station road and bus depot road. Length of station road is 269 m and length of bus depot road is 90 m.

While conducting survey, Valsad station road was divided into 19 different zones and bus depot road was divided into 5 zones for better accuracy and management. Survey was conducted for three continuous days of a week, i.e., February 14, 2018, February 15, 2018, and February 16, 2018 which are the peak days in a week. With duration of 12 h, survey was conducted from 8 AM to 8 PM each day. Parking survey data of four different vehicle types, two-wheeler, car, auto-rickshaw, and LCV was collected by license-plate on-street parking survey method. Interval of data collection was kept 15 min.

5 Data Analysis and Results

Analysis of collected parking survey data includes certain terms associated with parking such as parking accumulation, parking volume, parking load, parking duration, parking turnover, and parking index. Each day survey data was converted into ECPS.

5.1 Equivalent Car Parking Space

Equivalent car parking space (ECPS) is a term used for space needed for parking of 1 car and includes the circulation space needed for the same. As per IRC SP 12 (Indian Road Congress 2015a, b), parking dimension of different types of vehicles and their ECPS are shown in Table 1.

As per the calculation of collected survey data, peak ECPS is observed on February 14, 2017 which is **93** for station road and **22** for bus depot road.

Table 1 Parking space dimensions for different types of vehicles

Type of vehicle	Parking space dimension	ECPS
Two-wheeler	1.0 × 2.0 m	0.25
Cars/Taxi	2.5 × 5.0 m	1.0
Auto-rickshaw	2.5 × 2.5 m	0.50
LCV	2.5 × 5.0 m	1.0

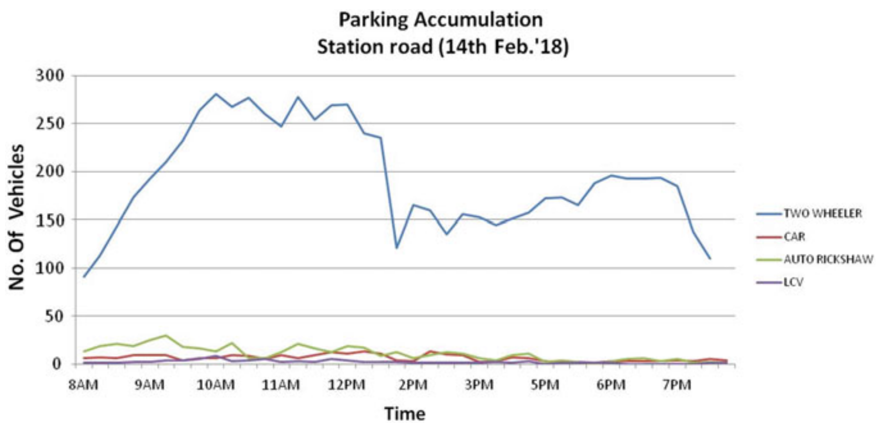


Fig. 3 Parking accumulation of station road

5.2 *Parking Accumulation*

It is defined as the number of vehicles parked at a given instant of time. Normally, this is expressed by accumulation curve. Accumulation curve is the graph obtained by plotting the number of bays occupied with respect to time. (Kadiyali 2011) Figs. 3 and 4 show the no. of vehicles parked on road per given time. It can be seen that two-wheelers are the most dominating types of vehicles in domain area for parking, followed by auto-rickshaw, car, and LCV.

From Figs. 3 and 4, it can be clearly seen that peak hours for parking are 10 AM to 12.30 PM for both roads.

5.3 *Parking Volume*

Parking volume is the total number of vehicles parked at a given duration of time. This does not account for the repetition of vehicles. (Kadiyali 2011) The actual volume of vehicles entered in the study area is recorded. The graphical presentation is shown in below Figs. 5 and 6 which shows a different type of vehicle parking on the same road

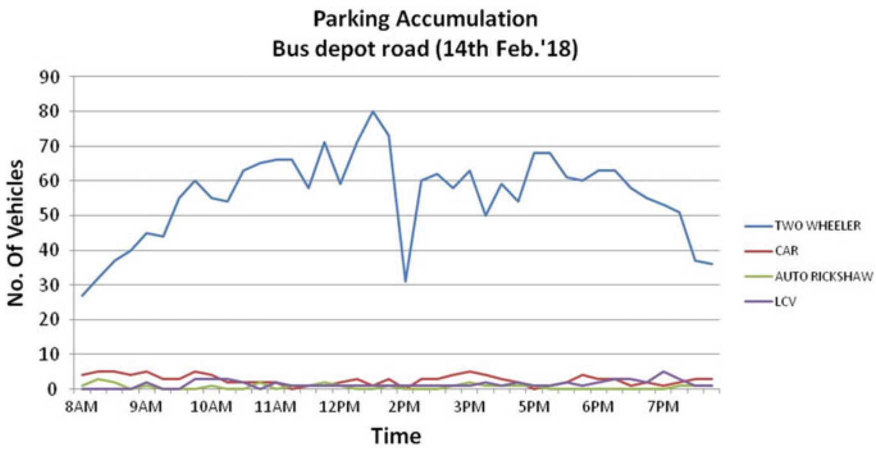
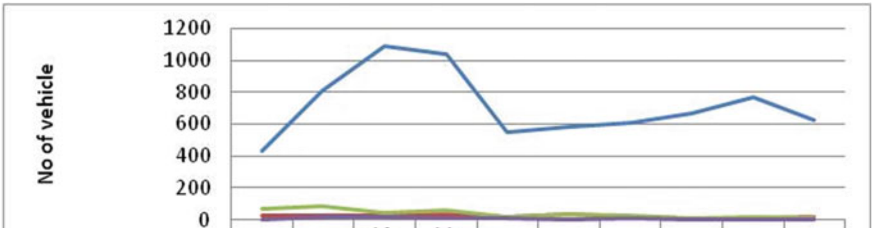


Fig. 4 Parking accumulation of bus depot road



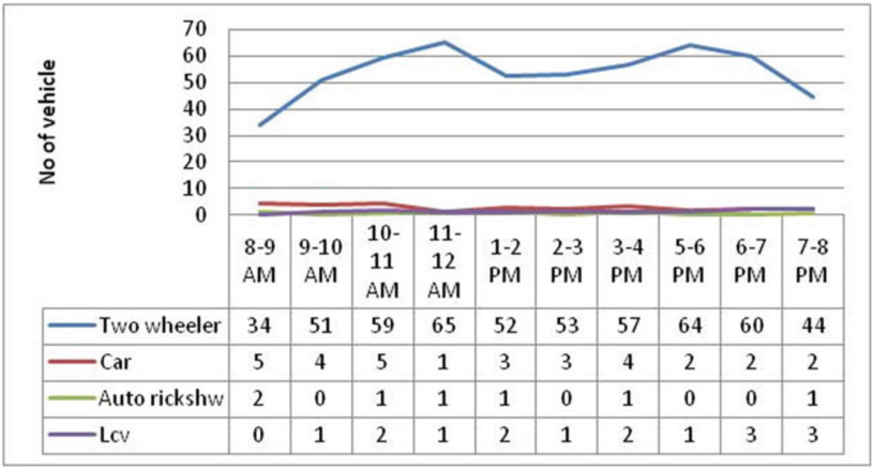
	8-9 AM	9-10 AM	10-11 AM	11-12 AM	1-2 PM	2-3 PM	3-4 PM	5-6 PM	6-7 PM	7-8 PM
Two wheeler	430	808	1090	1039	550	581	605	667	770	627
Car	28	28	28	36	17	35	18	7	12	16
Auto rickshaw	72	89	47	61	17	38	30	8	17	9
Lcv	5	15	20	12	7	4	7	3	1	2

Fig. 5 Parking volume of station road

for station road and bus depot road, respectively. Two-wheeler parking is dominating in both the roads.

5.4 Parking Load

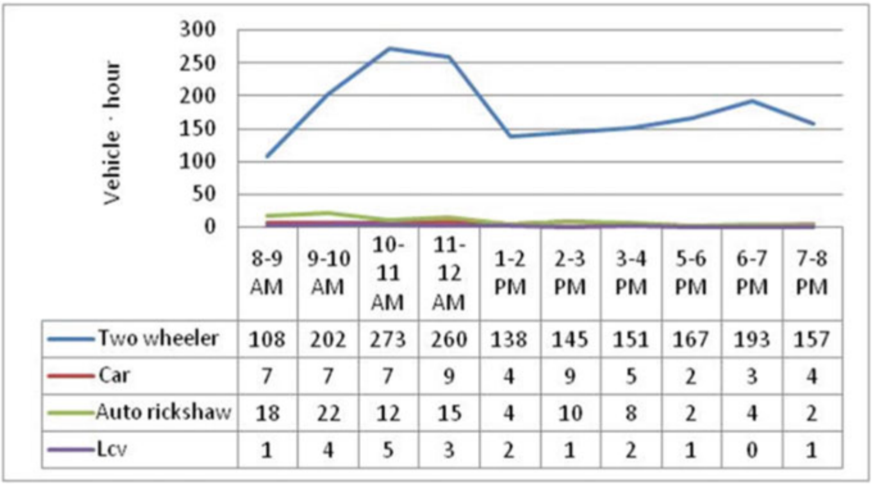
Parking load gives the area under the accumulation curve. It can also be obtained by simply multiplying the number of vehicles occupying the parking area at each



	8-9 AM	9-10 AM	10-11 AM	11-12 AM	1-2 PM	2-3 PM	3-4 PM	5-6 PM	6-7 PM	7-8 PM
Two wheeler	34	51	59	65	52	53	57	64	60	44
Car	5	4	5	1	3	3	4	2	2	2
Auto rickshw	2	0	1	1	1	0	1	0	0	1
Lcv	0	1	2	1	2	1	2	1	3	3

Fig. 6 Parking volume of bus depot road

time interval with the time interval. It is expressed as vehicle hours (Kadiyali 2011) (Figs. 7 and 8).



	8-9 AM	9-10 AM	10-11 AM	11-12 AM	1-2 PM	2-3 PM	3-4 PM	5-6 PM	6-7 PM	7-8 PM
Two wheeler	108	202	273	260	138	145	151	167	193	157
Car	7	7	7	9	4	9	5	2	3	4
Auto rickshaw	18	22	12	15	4	10	8	2	4	2
Lcv	1	4	5	3	2	1	2	1	0	1

Fig. 7 Parking load of station road

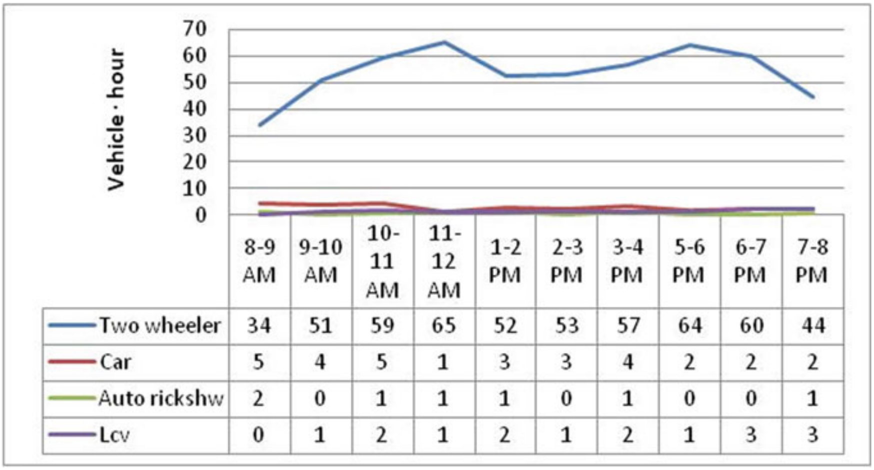


Fig. 8 Parking load of bus depot road

5.5 *Parking Duration*

A term parking duration is used to determine the time duration of parked vehicle. From primary survey data, parking duration was determined and related pie chart was made to understand the parking pattern easily (Figs. 9, 10, 11 and 12).

It is observed from the above graphs that more than 50% of all types of vehicles are parked for 15 min duration.

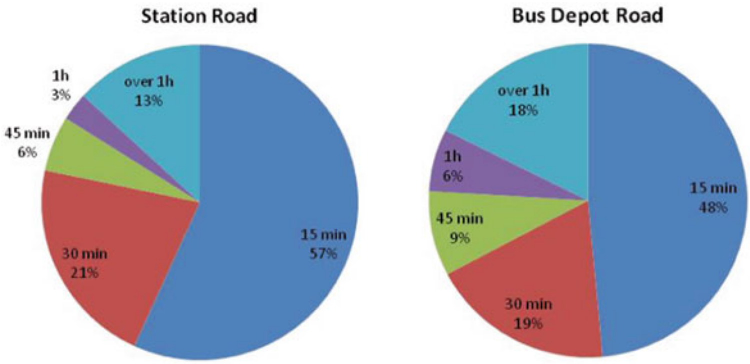


Fig. 9 Parking duration of two-wheelers on station road and bus depot road

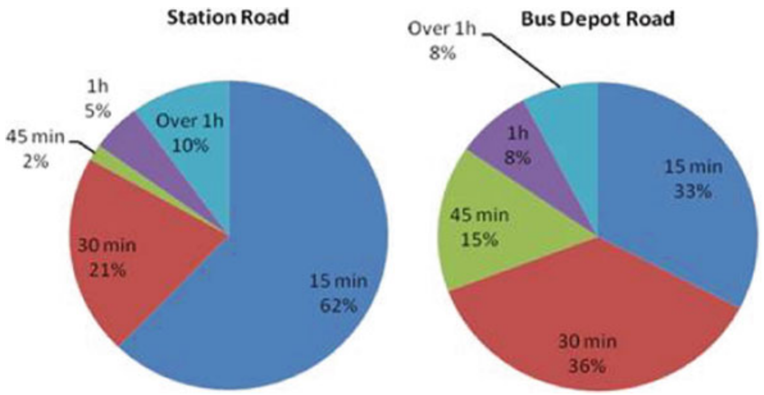


Fig. 10 Parking duration of cars on station road and bus depot road

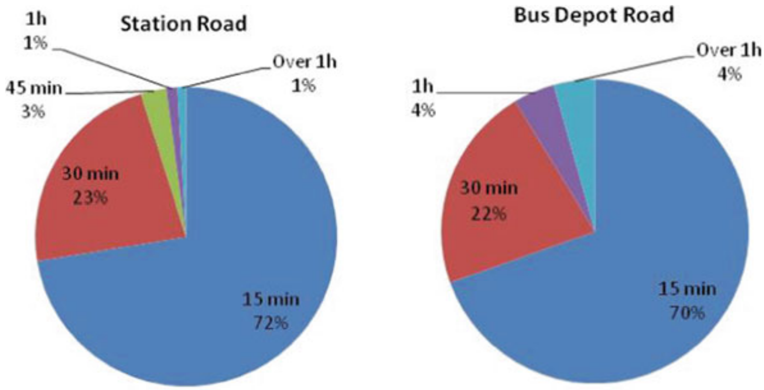


Fig. 11 Parking duration of auto-rickshaw on station road and bus depot road

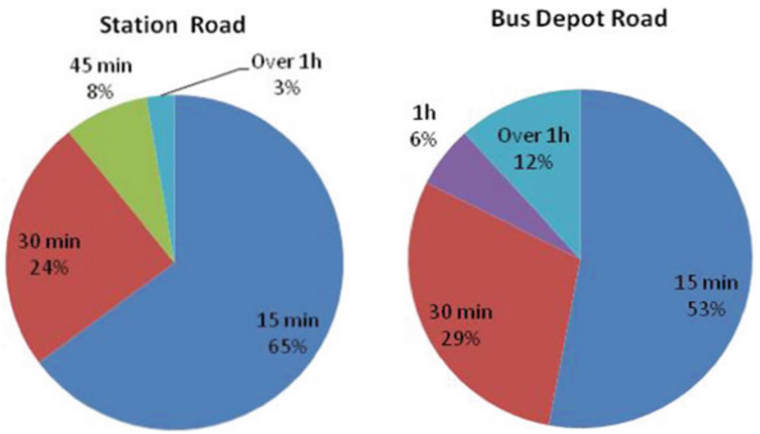


Fig. 12 Parking duration of LCV on station road and bus depot road

Table 2 Available space for on-street parking

Road	Available space for parking (m)	Intersection margin (m)	Total space available for on-street parking (m)
Station road(left)	258.8	10	248.8
Station road(right)	264.7	10	254.7
Bus depot road(left)	85.1	5	80.5
Bus depot road(right)	62.9	2	60.9

Table 3 Parking turnover

Road	No. of available parking bays	Turnover/hour
Station road(left)	176	4.02
Station road(right)	180	3.71
Bus depot road(left)	57	4.07
Bus depot road(right)	43	1.93

Note Above turnover values are calculated for an observed peak hour which is 10 AM to 11 AM

5.6 Parking Turnover

It is the ratio of number of vehicles parked in duration to the number of parking bays available. This can be expressed as a number of vehicles per bay per time duration. (Kadiyali 2011).

Length of the road where on-street parking can be provided is shown in Table 2.

Keeping 45-degree angle on-street parking on both roads, no. of available parking bays and their respective turnover are as follow (Table 3).

5.7 Parking Index

Parking index is also called occupancy or efficiency. It is defined as the ratio of a number of bays occupied in particular time duration to the total space available. It gives an aggregate measure of how effectively the parking space is utilized. (Kadiyali 2011).

Keeping 45-degree angle on-street parking on both roads, no. of available parking bays and their respective parking index are as follow (Table 4).

Table 4 Parking index

Road	Parking capacity	Parking index (%)
Station road(left)	176	100.42
Station road(right)	180	92.92
Bus depot road(left)	57	101.75
Bus depot road(right)	43	48.25

Note Above parking index values are calculated for an observed peak hour which is 10 AM to 11 AM

6 Senario Setting

From the analysis of collected parking survey data, it has been observed that peak demand for on-street parking on station road and bus depot road is between 10 AM and 12.30 PM. For scenario setting, probable solution is suggested on the bases of peak values of parking demand. Present peak parking demand (at 10.30 AM) for different types of vehicles on station road is as follow (Table 5).

Present peak parking demand (at 10.15 AM) for different types of vehicles on bus depot road is as follow (Table 6).

As discussed earlier, the major parking duration is 15 min that is more than 50%. Therefore, off-street parking as a major parking solution is not advisable on this road because this may affect on local businesses so that on-street parking becomes necessary on both roads. However, parking for auto-rickshaw is already provided by local authorities near station road and bus depot road as an "auto-rickshaw stand" (Indian Road Congress 2012). Hence, there is no need to provide on-street parking for auto-rickshaw on both roads. Parking demand of LCV on both roads is less compared to two-wheeler and car; hence, there is no need to provide on-street parking for LCV.

Table 5 Peak parking demand on station road

Vehicle type	Max. demand for parking	ECPS
Two-wheeler	281	70
Car	9	9
Auto-rickshaw	22	11
LCV	3	3

Table 6 Peak parking demand on bus depot road

Vehicle type	Max. demand for parking	ECPS
Two-wheeler	55	14
Car	4	4
Auto-rickshaw	1	1
LCV	3	3

Hence, the following scenarios are only dealt with on-street parking of two-wheelers and cars.

6.1 Scenario I

For station road and bus depot road, on-street parking is provided on both sides of the roads. Parking bays are marked at 45-degree angle where the dimension of each bay is 1.0 × 2.0 m. Footpath is provided on one side of both roads. (Indian Road Congress 2015a, b)

Road marking can be provided in such a way that both two-wheeler and car users can park their vehicles accordingly. For example, a car can be parked with using the parking space which is equivalent to the parking space of four two-wheelers. Car can be parked on road markings provided for two-wheelers as shown in Fig. 13. Cross-sections of station road and bus depot road are shown in Figs. 14 and 15 respectively.

Maximum total no. parking bays required on both roads to satisfy current peak parking demand is **388**. As per this scenario, maximum no. of parking bays which can be provided on both roads is **452**. Hence, this scenario satisfies current peak parking demand. Design layout of this scenario is as shown in Fig. 16.

6.1.1 Advantages

- There is less chance of traffic congestion and accidents while doing vehicle parking or removing vehicle from parking as compared to 60- and 90-degree on-street parking. (Nikolay 2010).
- With consideration of safety and land use, 45-degree angle on-street parking is an optimum parking pattern.
- As per RTO, Valsad data average growth rate of vehicles per year is 3% hence for same growth rate for parking demand, after five years maximum approx. **450** parking bays will be required to satisfy the peak parking demand. Hence, this scenario can satisfy peak parking demand for the next five years.

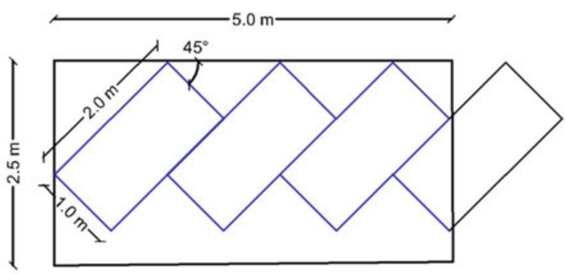


Fig. 13 Comparison of parking space for car and two-wheeler at 45-degree angle

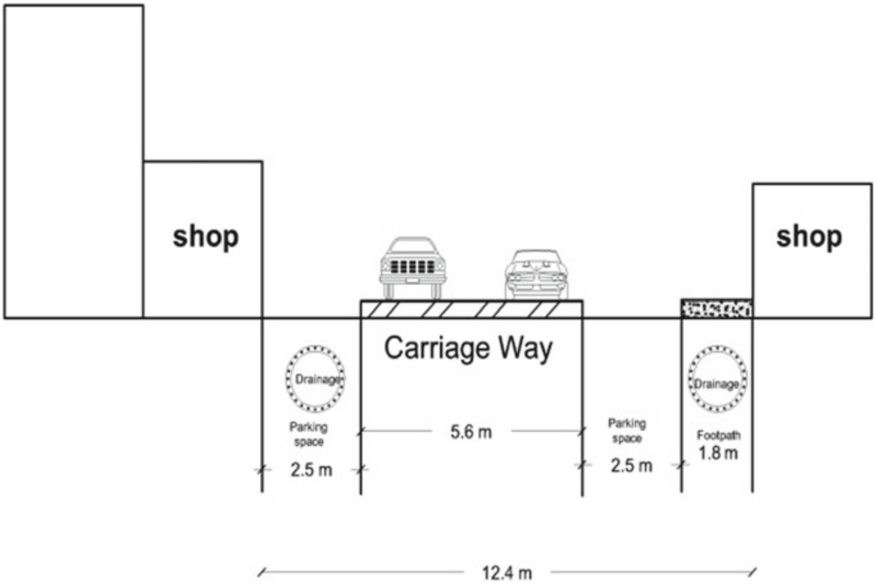


Fig. 14 Cross section (XX) of station road for scenario I

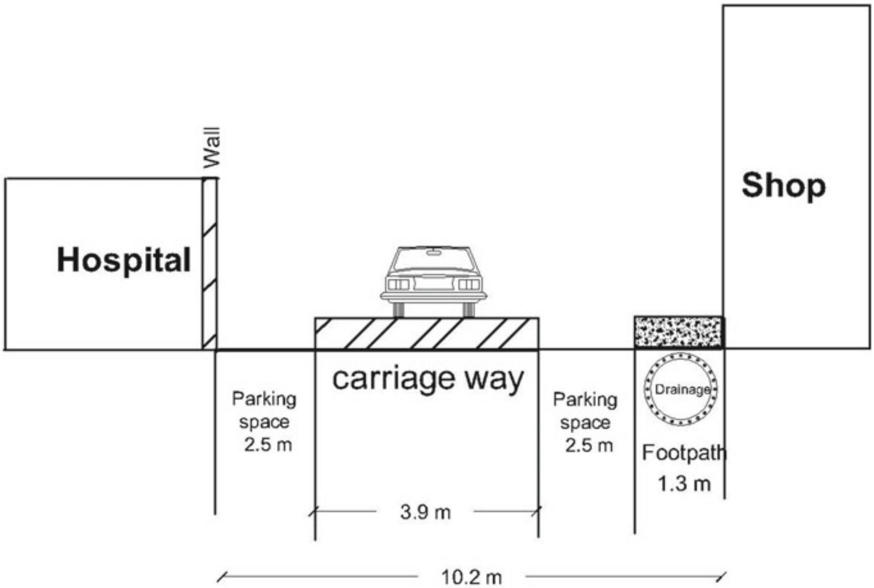


Fig. 15 Cross section (ZZ) of bus depot road for scenario I

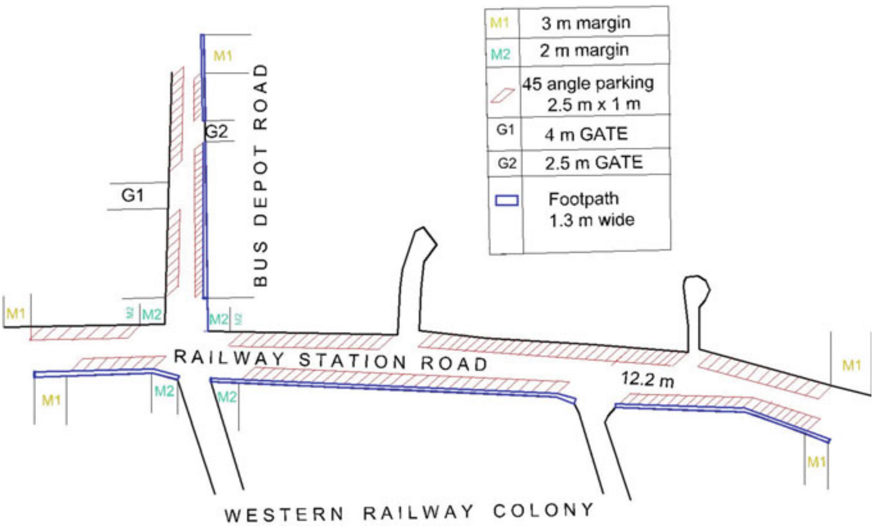


Fig. 16 Design layout of scenario I

6.1.2 Disadvantages

For parking of car, it is necessary that three or more parking bays should be available side by side which may not be possible every time especially in peak hours.

6.2 Scenario II

As the two-wheeler is dominating type of vehicle for parking on station road, on-street parking on station road can be provided only for two-wheelers at 45-degree angle.

Parking of car can be provided only on bus depot road, as this road is wide enough (10.2 m) to carry one-way traffic with two side parallel parking for car. Cars should park on bus depot road in lieu of station road.

Cross section of station road is as shown in Fig. 17, and bus station road for this scenario is same as the cross section of scenario I.

Maximum total no. parking bays required on both roads for two-wheelers to satisfy current peak parking demand is **342**, and on bus depot road for car, current peak parking demand in terms of no. of bays (of dimension 2.5 $\times$ 5.0 m) is **13**. As per this scenario, maximum no. of parking bays can be provided is **404** and **18** for two-wheeler and car, respectively. Hence, this scenario satisfies current peak parking demand.

Design layout of this scenario is as shown in Fig. 18.

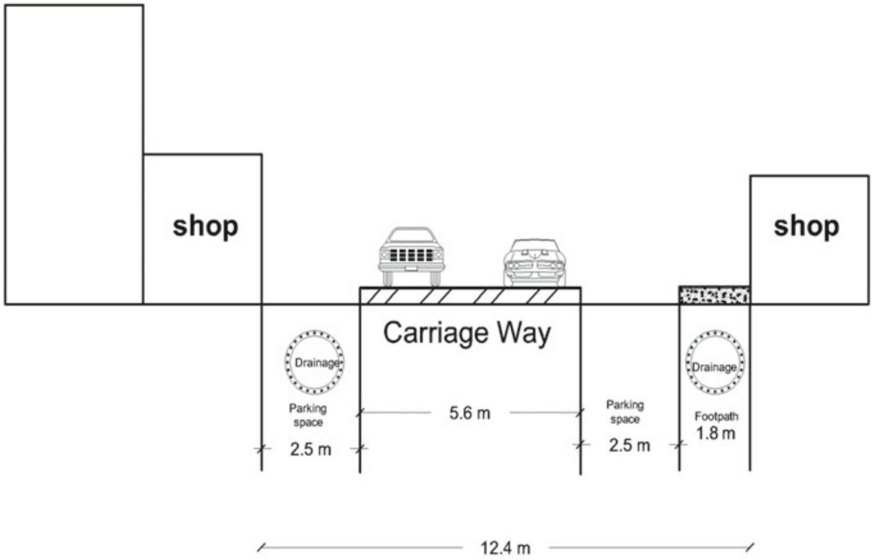


Fig. 17 Cross section (XX) of station road for scenario II

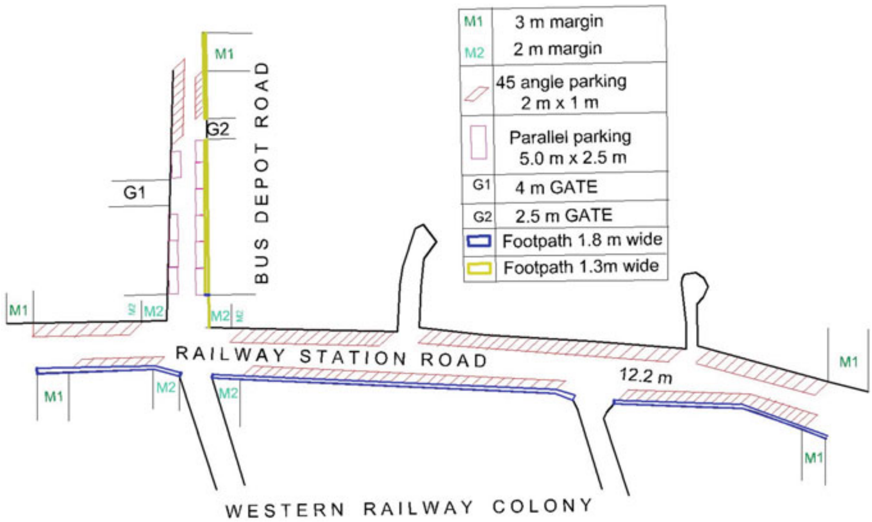


Fig. 18 Design layout of scenario II

6.2.1 Advantages

- As parking of cars is provided only on bus depot road, congestion due to parking of cars on station road can be eliminated.
- As parking space for cars is provided on bus depot road, there is no need to find parking space for cars.
- Considering 3% growth rate for parking demand, after five years maximum approx. **396** parking bays for two-wheeler will be required to satisfy the peak parking demand.

  Hence, this scenario can satisfy peak parking demand for the next five years.

6.2.2 Disadvantages

This scenario gives limited parking options for car users.

6.3 Scenario III

On station road, parking can be provided by using *odd-even parking system* with 90-degree angle on-street parking. In this system, on-street parking may do on one side of road at a time, i.e., on even dates such as 2, 4, 6, and so on, parking is provided on right side of the road and on odd dates such as 1, 3, 5, and so on, parking is provided on left side.

Car can be parked on road markings provided for two-wheelers as shown in. For illustration, car dimension is of 2.5 × 5.0 m is taken as shown in Fig. 19.

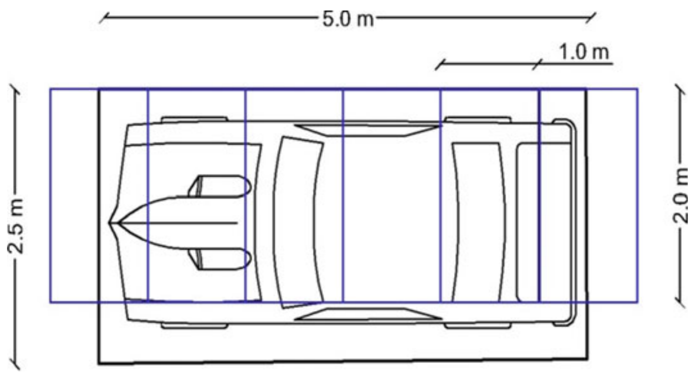


Fig. 19 Comparison of parking space for car and two-wheeler at 90-degree angle

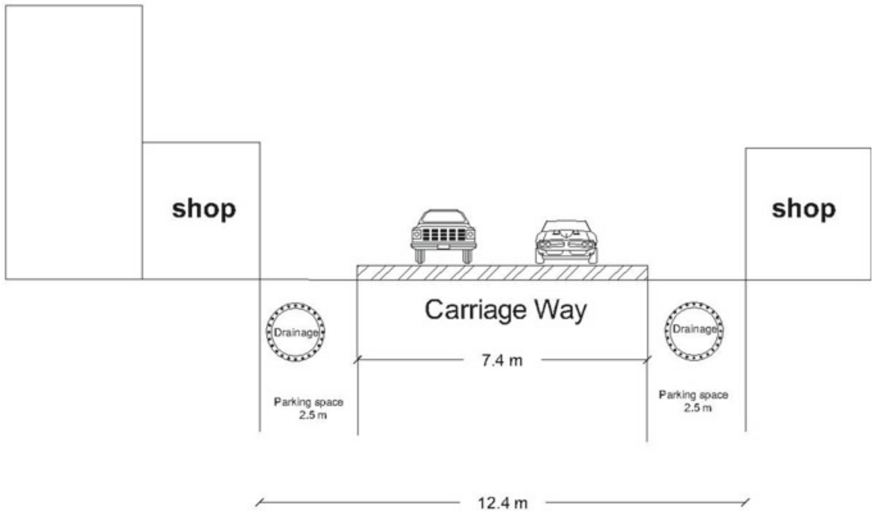


Fig. 20 Cross section (XX) of station road of scenario III

Cross section of station road is shown in Fig. 20 and cross section of bus depot road for this scenario is same as the cross section of scenario I. In this scenario, car parking is provided only on left side of station road and both side of bus depot road.

Maximum no. of parking bays (of dimension 1.0 × 2.0 m) that can be marked on one side of station road with 90-degree angle on-street parking is **250**. (Indian Road Congress 2015a, b)

Bus depot road is one-way road and width of road is 10.2 m. Therefore, on-street parking on both sides is allowable and does not interrupt traffic. Maximum no. of parking bays (of dimension 1.0 × 2.0 m) that can be marked on left and right side of bus depot road with 90-degree angle on-street parking is **80** and **60**, respectively.

For current year, maximum approximate parking bays required to provide sufficient on-street parking facilities on station road is 320 and for the bus depot road is 80. Hence, in total approx. **388** parking bays of dimension 1.0 × 2.0 m is required within peak hour to satisfy current on-street parking demand on both roads.

But as discussed, at 90-degree angle, maximum no. of parking bays that can be marked for both of roads is **390** (250 + 80 + 60).

Design layout of this scenario is as shown in Fig. 21.

6.3.1 Advantages

- Odd-even parking system will be beneficiary for traffic flow on station road. By using this system, on-street parking will not interrupt the traffic flow. Reasons for providing odd-even parking system on station road are as follow:

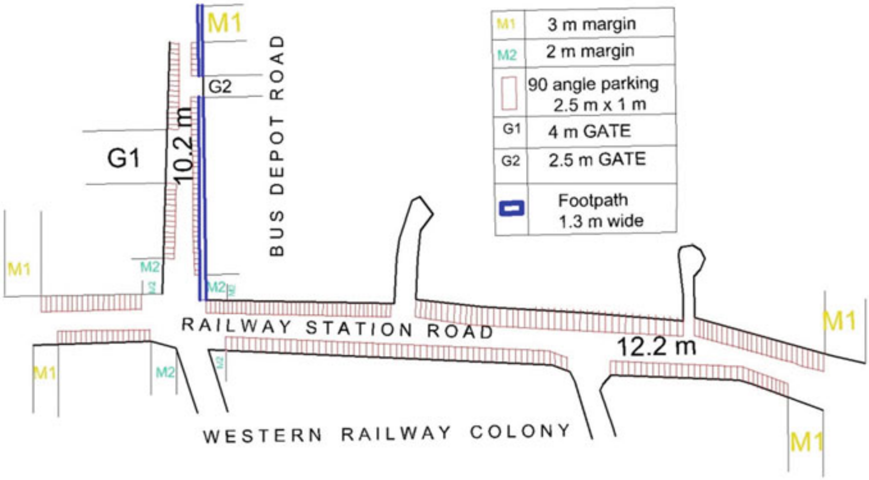


Fig. 21 Design layout of scenario III

- **Careless parking**: Due to odd-even parking system, one cannot leave their parked vehicle behind for more than a day so there is no unnecessary congestion.
- **Fair visibility**: On Valsad station road, there are shops on both sides of the road. The same shop's visibility would not be affected everyday due to on-street parking. Hence, odd-even parking will give equal visibility options to the shops.
- **Cleanliness**: Due to odd-even parking system, one side of the road can be cleaned on even days and the other on odd days.

- As parking is provided only on one side of the road, more width of carriageway will be available to use than current width. Hence, traffic on station road can flow smoothly.

6.3.2 Disadvantages

- For parking of car, it is necessary that minimum three or more parking bays should be available side by side which may not be possible every time especially in peak hours.
- Considering 3% growth rate for parking demand, after two years, i.e., in 2020, maximum approx. **411** parking bays will be required to satisfy the peak parking demand.

However, this scenario is suitable to satisfy parking demand till 2020.

7 Recommendation

Among above-mentioned three scenarios, scenario III is the best probable solution with a view to reduce both traffic and parking problems but the drawback is it will get exhausted from the year of 2020.

There are approx. 10% vehicles which were parked on station road for more than an hour. For these vehicles, off-street parking is suitable which is already available at two different locations near station road and these off-street parking facilities are owned by the private authority. As per development plan of Valsad city, off-street parking can be provided at Navrang Lassi center situated on station road. This proposed parking lot is encroached by Navrang Lassi center. It is highly recommended to vacant this plot and uses it as a parking lot as per the development plan.

To eliminate a major drawback of scenario III, parking meters can be introduced. By introducing parking meters, design years of any scenario can be increased by minimum five years (Johnson and Ponnuswamy 2012).

According to the current scenario, there are encroachments for proposed on-street parking such as hand carts, temporary stalls, and extra luggage of shops which are plied on footpath or sometimes on road which causes unnecessary congestion and traffic problems. These encroachments must be removed and proper enforcement must be applied for proper execution of proposed on-street parking. A parking policy should be implemented.

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Addressing Informal Public Transport Vehicle Problem in Kolkata

Ankita Baksi, Jayita Guha Niyogi, and Arup Guha Niyogi

Abstract Most of the Indian cities are suffering from rapid growth of traffic volume which causes disruptions in easy travel to the destinations. With the addition of informal public transport (IPT), these difficulties have multiplied. Rise in share of low passenger capacity IPT is increasing the passenger car units (PCU) on roads manifold. This, in turn, enhances delay, road density and pollution, while reducing speed. This paper aims to assess the problems caused by IPT and suggest strategies to control them in Kolkata. The busy Rashbehari and Hazra intersections of Kolkata are studied here, and some planning schemes are tested by restricting the IPT that results in overall reduction in the number of vehicles, PCU load, road density in that intersection.

Keywords Informal public transport · Passenger car unit · Flow rate · Speed · Density · Travel time

1 Introduction

Kolkata, like most Indian cities, suffers the problem of ingress from neighbouring suburban belt in search of easier livelihood. Thus, there is a rapid rise in urban traffic volume with increasing count of self-owned automated two- and four-wheelers and small-volume informal public transport (IPT) hitting roads to reach workplaces conveniently and quicker during the peak hours. The IPTs, with low carrying capacity,

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have smaller wait time, are affordably priced, are less crammed and have better mobility through the crowded streets. Though IPT was designed to act as a paratransit mode for bringing people from their door steps to the arterial roads, they have steadily been substituting the bulkier transit modes in the arterial roads, posing serious threat to the sustenance of private transit service. The transit fare, regulated by the state government, is meagre, and IPT, permitted to ply through arterial roads, is naturally winning a major share of transit passengers who travel shorter spans. The convenience of seating and ready availability is fast relegating the bus service, and the passenger car units (PCU) on roads have grown enormously. Gradually speed of travel has dropped since road width, on-street parking and queuing space are inadequate, while drop in air quality and noise pollution are other major issues. Private transit service has become a sick and unreliable industry.

There are many IPT vehicles on the road without prior licence and fake registration numbers. More than 50% auto rickshaws (AR) are commuting without legal permit in Kolkata city and metropolitan roads (The Telegraph 2008). Lack of control over vehicle maintenance and quality of drivers, coupled with rush driving on congested road, have increased occurrence of road accidents. AR drivers break traffic rules every three seconds in Kolkata. The violation is mainly related to over-speeding and parking in prohibited area (Times of India 2016). The state-run buses have shifted predominantly to air-conditioned (AC) buses with premium fare rates (Times of India 2013; The Hindu 2014), while the non-air-conditioned (N-AC) private buses are scanty, arrive after long intervals, remain overcrowded and have become uncomfortable. Their ergonomic quality has deteriorated over the decade, in absence of stringent vigil. Buses stop and start frequently to take and drop passengers, affecting the road blacktop badly (The Telegraph 2014). Often, the IPT avoids congested arterial roads and detours through the neighbourhood street posing further threat of accident within the neighbourhoods. Kolkata population prefer to travel by AR having affordable, easy-to-access, high-frequency and fixed-route operation unlike many other Indian cities (Auto rickshaws in Kolkata 2016). This comforts the passengers of IPT but is a threat to the locality and transit modes. Again, in absence of proper vigil or penal action, the IPT often plies over a truncated stretch that has heavier demand instead of full route to earn more. Now that the short-distance commuters have opted for IPT and personal vehicles, the long-distance commuters in Kolkata are clueless, either get fatigued inside sparsely available N-AC private buses (The Telegraph 2014) or pay manifold for state-run AC buses, and one must address this problem.

This paper aims to assess the schemes and policies undertaken in Kolkata to reduce the depressing effect of informal urban transport. The research also reviews the use of informal public transport in Rashbehari intersection, one of the busiest road intersections in South Kolkata and speed analysis of the intersection at peak hour, based on video footages received from the Kolkata Traffic Police Department.

2 Objectives

Since Kolkata has been a rapidly growing old city, the share of roads in the heart of the city is poor. With the introduction of IPT in the trunk roads where double-decker buses used to serve even in the 1970s and 1980s, but withdrawn for some unknown reasons, the buses are now stealthily being replaced by IPT. People are trying to cut short with the wait time and avail seating comfort that IPT provides. However, the process is affecting road density and telling upon speed and air quality now there being dearth of road space. Thus, this research aims to identify the problems of running IPT in the streets of Kolkata and suggest remedial measures that are practicable.

The content of this paper is presented in the following sections: A techno-economic appraisal of the vehicles used to carry people commercially has been documented. This is followed by two sections explaining the status report on riders' plight in Kolkata and measures adopted by the state government to address the situation. The report on the present study follows next.

For the present research work, video footages were acquired from the Traffic Police Department, Kolkata, at various important intersections leading to the Central Business District (CBD) of Kolkata. This paper, however, concentrates only on an arterial stretch between Rashbehari and Hazra intersections in South Kolkata as a one-off study that will be integrated later.

The footage has been analysed to obtain the modal split during the peak office hours in the morning, and weekends. The split clearly reveals inclination of riders towards auto rickshaws. It is understood that the auto rickshaws are presently challenging the sustenance of private owned non-AC buses that used to be a major supplement to state-run buses. Trials are made to restrict IPTs and boost usage of bus transit to de-congest streets in office hours. IPT could be engaged to assist people from their door steps to arterial roads.

3 Techno-economic Appraisal of IPTs and Buses in Kolkata

As a parallel study, the technical details, as well as financial involvement of different vehicles, namely auto rickshaw, Tata Sumo, Tata Winger, and Tata Magic, the four major informal public transports those operate in Kolkata city core area, are reviewed. Besides these, people from Kolkata peri-urban area also depend on Toto, e-rickshaw, tricycle rickshaw to connect with major transport stations. The fuel used in AR is either CNG or petrol with 4-stroke single cylinder forced air cooled spark ignition (ISI) engine, and it has electrical ignition system. Their features are provided in Table 1.

Buses that operate in Kolkata city are of many types: Volvo, AC and N-AC JNNURM (Jawaharlal Nehru National Urban Renewal Mission) Jan buses, private N-AC bus and minibuses. All types of buses described here generally use diesel as

Table 1 Techno-economic characteristics of IPT

Type of mode	Fuel type	Engine type	Starting	Max speed (km/hr)	Size	Passenger capacity	Price in Rs. (Approx)
Auto rickshaw[1,2]	CNG/petrol	4-stroke single cylinder forced air cooled spark ignition (ISI) engine	Electric	56	2645 mm (L) × 1310 mm (W) × 1700 mm (H)	1 + 4	135,130
Tata Sumo[3]	Diesel	4SP (3.0L) Turbo Charged	Electric	125	4258 mm (L) × 1700 mm (W) × 1925 mm (H)	1 + 7	700,000
Tata Magic[4]	Diesel	4-Stroke, Naturally Aspirated, Indirect Injection, Water Cooled Diesel Engine	Electric	64	3790 mm (L) × 1500 mm (W) × 1845 mm (H)	1 + 7	400,000
Tata Winger[5]	Diesel	2.0 litre diesel engine, 1948 cc, turbo-charged, inter-cooled (TCIC)	Electric	115	4920 mm (L) × 1905 mm (W) × 2050 mm (H)	1 + 12	750,000
Taxi[6, 7]	Diesel	1.5-litre 35.5bhp 16 V STRIDE Diesel Engine	Electric	129	4325 mm (L) × 1662 mm (W) × 1593 mm (H)	1 + 3	500,000
Toto[8, 9]	Battery operated	–	Electric	25	2850 mm (L) × 1050 mm (W) × 1850 mm (H)	1 + 5	120,000
E-rickshaw[10]	Battery operated	–	Electric	25	–	1 + 2	35,000

(continued)

Table 1 (continued)

Type of mode	Fuel type	Engine type	Starting	Max speed (km/hr)	Size	Passenger capacity	Price in Rs. (Approx)
Tricycle rickshaw[10]	Paddle system	–	–	–	–	1 + 2	14,000

[1]http://www.tuktukind.com/double-head-light-auto-rickshaw.html
[2]https://www.wlivenews.com/tvs-king-auto-features-and-price-in-india.html
[3]https://www.cardekho.com/tata/tata-sumo-specifications.htm
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[10]https://www.indiamart.com/proddetail/bicycle-rickshaw-15335329530.html

fuel and have electrical ignition system. Their techno-economic features are provided in Table 2.

4 Updates on Kolkata Transport System

Since IPT is controlled by nominated or elected association, in most of the cases, the association focuses on their own profit rather than passengers' needs. As recorded in 2008, around 55,000 legal and illegal auto rickshaws ply in Kolkata who pay a premium to the associations and often claim immunity from law (The Telegraph 2008; Times of India 2016). Altercations with passengers are a regular feature (Times of India 2017). Unions often increase fare citing rise in LPG price, or, sighting festivals without government concurrence (Times of India 2012; Times of India 2015) and commuters haplessly accept, ARs being the second biggest passenger carriers in the city. Often, the ARs illegally take two riders in the front seat instead of taking only one in absence of vigil. Complaints from passengers about quality of services, frequencies, playing loud music, misbehaviour, refusing long-distance passengers, truncating routes, etc., mostly go unheeded (Times of India 2015). ARs often dare to take a detour inside neighbourhood when there is a jam in the arterial roads. However, AR service could be a viable urban transport solution if the routes are regulated and drivers are properly counselled (Times of India 2013).

Rise in pollution level has resulted due to congested roads and plying in lower gears. Risks of lung cancer, respiratory problems are on the rise due to very poor air quality. Registered ARs that are supposed to be working on petrol contribute 69% hydrocarbon and 43% carbon monoxide to the Kolkata air. If the unregistered ARs are considered which run on adulterated oil, the emission rate of toxic gases will be much higher (The Telegraph 2008). Illegal trade of adulterated oil, a toxic

Table 2 Techno-economic characteristics of buses

Type of bus	Engine type	Max speed (km/hr)	Size	Passenger capacity			Price in Rs. (Approx)
				Seating capacity	Standing capacity (approx)	Total	
Volvo[1]	Turbocharger and Intercooler, powered with 290 hp, max torque 1200 Nm	80	12,290 mm (L) × 2500 mm (W) × 3200 mm (H)	32 + 2	25	59	71.84 lakhs
JNNURM_Jan bus type 1 (N-AC) Ministry of Urban Development Government of India, Gov. of India, (2013)[2]	Turbocharger and Intercooler, powered with 230 hp	75	12,000 mm (L) × 2600 mm (W) × 3200 mm (H)	43 + 1	20	64	58 lakhs
JNNURM_Jan bus type 2 (AC) Ministry of Urban Development Government of India, Gov. of India, (2013)[2]	Turbocharger and Intercooler, powered with 250 hp	75	9000 mm (L) × 2500 mm (W) × 3200 mm (H)	30 + 2	30	62	60 lakhs
JNNURM_Jan bus type 3 (N-AC) Ministry of Urban Development Government of India, Gov. of India, (2013)[2]	Turbocharger and Intercooler, powered with 230 hp	75	9000 mm (L) × 2500 mm (W) × 3200 mm (H)	30 + 2	30	62	50 lakhs

(continued)

Table 2 (continued)

Type of bus	Engine type	Max speed (km/hr)	Size	Passenger capacity			Price in Rs. (Approx)
				Seating capacity	Standing capacity (approx)	Total	
JNNURM_Jan bus type 4 (N-AC) Ministry of Urban Development Government of India, Gov. of India, (2013)[2]	Turbocharger and Intercooler, powered with 230 hp	75	12,000 mm (L) × 2600 mm (W) × 3200 mm (H)	46	Not allowed	46	58 lakhs
Minibus	–	–	5070 mm (L) × 1465 mm (W) × 1900 mm (H)	36 + 1	15	52	–
Private bus (N-AC)	–	–	9000 mm (L) × 2500 mm (W) × 3200 mm (H)	32 + 2	25	59	–

[1] https://buses.cardekho.com/buses/volvo/8400-city-bus/specifications
[2] https://buses.cardekho.com/buses/ashok-leyland.html

mixture of petrol, kerosene and naphtha, was found at different points of Kolkata before the introduction of green ARs that use CNG around 2010–11. However, the older systems are still plying in the city outskirts. Auto drivers preferred this mix being cheaper than petrol (The Telegraph 2009).

Accidents occur due to the rivalry among the vehicles and poor maintenance (Times of India 2018). Private buses tend to block the road during boarding and alighting of passengers. Bus and AR drivers prefer to adopt cheaper alternative in the form of resoled tyres that offer poor braking ability (Times of India 2018).

In 2013, Calcutta High Court directed the state government to install metres in auto rickshaws and to fix a fare structure, which was summarily ignored. In 2014, the state government formed a committee to make a systematic structure. The committee instructed the auto drivers to maintain a common dress code for all auto drivers and to carry all the documents which too went unheeded (Times of India 2018).

5 Strategies Adopted by the State Government to Control IPT

The Government of West Bengal took initiative to display a helpline number, 1073, to register passengers' complaints. The government banned installation of distracting music systems and LED light decorations on the vehicles. Periodical workshops were proposed to be organised by the police to observe that the AR drivers know all the places and roads in the city. The traffic police have been instructed not to stop the offending drivers suddenly but inform the police at the next intersection where the errant driver could be nabbed. ARs have been debarred from carrying more than four passengers (Times of India 2012). Calcutta High Court has banned plying of all 2-stroke ARs as well as those 4-stroke ARs who have not adopted to using clean fuel within and outside Kolkata Municipal Corporation Area (The Telegraph 2008). A task force has been constituted to frame an AR policy for convicting culpable drivers (Times of India 2016). As an alternative to fast AR service, the government is planning to introduce ten air-conditioned minibuses with 29 seating capacity and some space for standing. The fare will vary from Rs. 15 to 40. Minibuses could be negotiating better through the narrow stretches in Kolkata than bigger buses with larger turning radius and higher capacity that take time to fill in. By introducing the small AC buses, government is hoping to reduce such kind of problem (The Telegraph 2017). Kolkata police has planned to debar AR from parking and picking up passengers along main roads randomly. Its passengers will have to go aboard and alight in designated stops (Kolkata Transportation Thread 2017). Real-time vehicle information of high-speed zone is received by Kolkata traffic control room and Road Transport Authority. If any violation is done by any motorists, the camera scans the number plate and an e-challan is automatically sent to their registered mobile number via SMS (Times of India 2017). As per the West Bengal Motor Vehicle Rules, penalties are imposed for driving without licence, owner allowing other driver without proper driving licence, holding of licence by disqualified person, driving vehicle without registration and certificate of fitness, intersection of the speed limit, violating mandatory traffic signs, overloading vehicles that causes obstruction to road users, parking vehicle at public space, not showing driving licence to police officers in uniform, not stopping while asked by the police officers in uniform, driving vehicle dangerously, drunken driving, racing in public place, using unsafe vehicle, violating road safety and pollution control norms, carrying hazardous goods, driving on footpath, use of ear- or microphone and audio devices while driving, using faulty number plate, driving with imperfect tyres, and not having pollution under control certificate (Penalties under Central 1988). Rs. 100 penalty is imposed for the first time observed using resoled tyres and Rs. 250 in following occurrences. On an average, around 7500 cases of resoled tyres are filed in Kolkata per month (Times of India 2018). However, these steps still seem inadequate in view of regular accidents, on-road altercations and dropout in number of bus services, demanding a deeper probe.

6 The Present Research

6.1 Formulation

Rashbehari and Hazra crossing, two of the intersections within Southern Kolkata, is surveyed through CCTV footages to study the effect of IPTs on road intersections leading to Kolkata CBD. This contribution by the Kolkata Traffic Police Department (KTPD) is gratefully acknowledged.

Travel time of car, N-AC bus and bike at morning peak period is identified from Rashbehari crossing to Hazra crossing and vice versa. Traffic volume of one hour of Rashbehari crossing is studied for both peak and off-peak periods. Based on Eqs. (1) and (2) (Papacostas et al. 1993), speed at off-peak time (u_f) and peak time (u_j) has been computed by using travel time at off-peak time (TT_f), travel time at peak time (TT_j) and road length (L). Flow rate at off-peak time (q_f) and peak time (q_j) is identified from CCTV footages.

$$u_f = \frac{1}{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{1}{L/TT_f}\right)} \tag{1}$$

$$u_j = \frac{1}{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{1}{L/TT_j}\right)} \tag{2}$$

Density at off-peak time (k_f) and peak time (k_j) is measured using Eqs. (3) and (4) (Papacostas et al. 1993).

$$k_f = q_f \big/ u_f \tag{3}$$

$$k_j = q_j \big/ u_j \tag{4}$$

After identification of existing condition, three experiments have been formulated to analyse the effects concerning traffic volume, PCU load and density.

6.2 Speed Analysis Between Rashbehari and Hazra Intersections

Speed of a traffic intersection has been measured using the distance travelled by the vehicles between two consecutive intersections at two different time instants. Distance of the two intervals is measured from the **Google Earth** application. Number plates of the vehicles are read (Vision-Based Vehicle Speed Measurement Method Witold Czajewski and Marcin Iwanowski Warsaw University of Technology Institute of Control and Industrial Electronics), and reference starting and ending points are detected from the clips to measure travel speed from the length of position change and corresponding travel time (Devender et al. 2015).

Reportedly, in various road stretches in Kolkata, the speed limit for cars is 60 km/h, and for heavy vehicle, it is 40 km/h (The Times of India 2013; The Times of India 2017). An attempt has been made here to assess the same from the CCTV footages provided by KTPD for July 2017 at Hazra intersection and Rashbehari intersection (Fig. 1), recorded at morning weekday peak period and weekend morning off-peak period. Distance between these two intersections is 740 m. One car was spotted in Rashbehari intersection travelling towards Hazra intersection, i.e. towards the CBD, at morning weekday peak period at 10:21 a.m., which crossed Hazra intersection at 10:24 a.m. (Figure 2). It took three minutes to travel 740 m. Thus, the speed of the car at morning weekday peak period in this stretch is observed to be 15 km/hr. Similarly, the speed of a motor bike and an N-AC bus moving towards Hazra intersection, i.e. towards the CBD, was observed which were 15 km/hr and 4 km/hr, respectively. Again, in opposite flow, the speed of a car, a motor bike and an N-AC bus travelling from Hazra intersection to Rashbehari intersection was observed to be 6 km/hr, 25 km/hr and 4 km/hr, respectively (Table 3). The buses that move in the non-peak direction travel slowly to gather more passengers and create constrictions that deter other fast-moving vehicles. Comparison of admissible and recorded speeds suggests that some regulatory measures are essential to promote average vehicular speeds.

Twelve kinds of traffic flow could be identified at the Rashbehari intersection (Fig. 3), there being four arms and three possible movements (left, right and straight) from each arm.

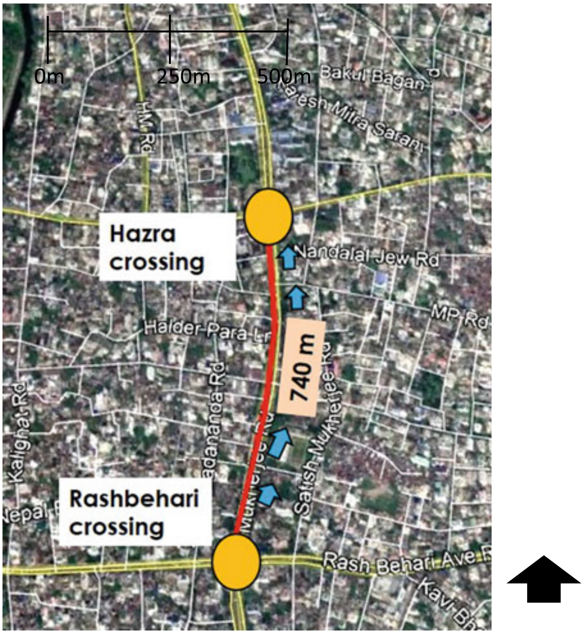

Fig. 1 Source: Google Map

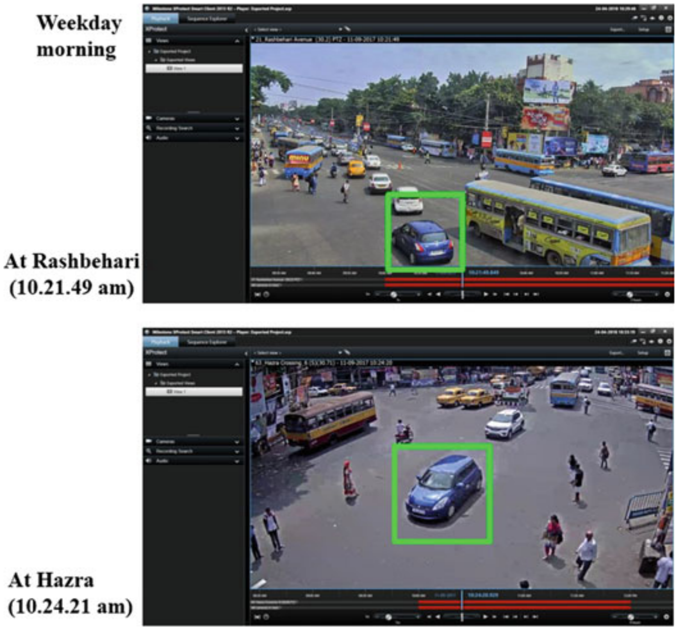

Fig. 2 Screenshot from KTPD CCTV footage

Table 3 Mode-wise speed at different directions (KTPD video footage analysis)

Vehicular mode	From Rashbehari intersection to Hazra intersection			From Hazra intersection to Rashbehari intersection		
	Travel time at peak period in hr	Road length in km	Speed at peak period in km/hr	Travel time at peak period in hr	Road length in km	Speed at peak period in km/hr
Car	0.05	0.74	15	0.12	0.74	6
N-AC bus	0.2	0.74	4	0.17	0.74	4
Bike	0.05	0.74	15	0.03	0.74	25

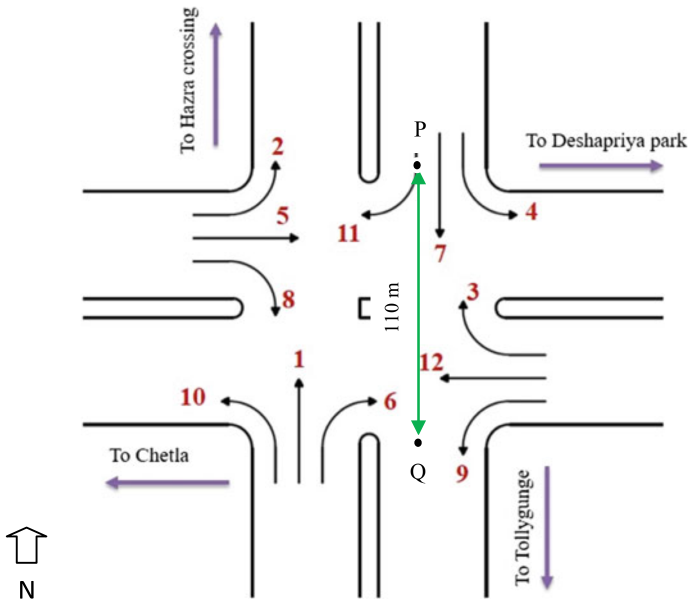

Fig. 3 Schematic drawing of Rashbehari crossing showing the movement of modes

6.3 Peak Time Traffic Condition at Rashbehari Intersection

Rashbehari intersection is located in the Southern Kolkata which connects South Kolkata to the CBD due north (Figs. 4 and 5). CBD of Kolkata comprises of B. B. D Bag area, Esplanade area and Park street area (Fig. 4). Rashbehari intersection connects Chetla towards west, Deshapriya Park towards east and Tollygunge towards south (Fig. 5), while Hazra intersection and the CBD are to the North.

One-hour morning peak period traffic count during weekdays at the Rashbehari intersection revealed movements of 849 taxies, 2153 cars, 62 44-seater AC buses, 391 34-seater N-AC buses, 121 27-seater minibuses, 104 bicycles, 825 bikes, 670 ARs, 74 heavy vehicles (HV) and 48 police vans (PV). Being morning peak, vehicle density

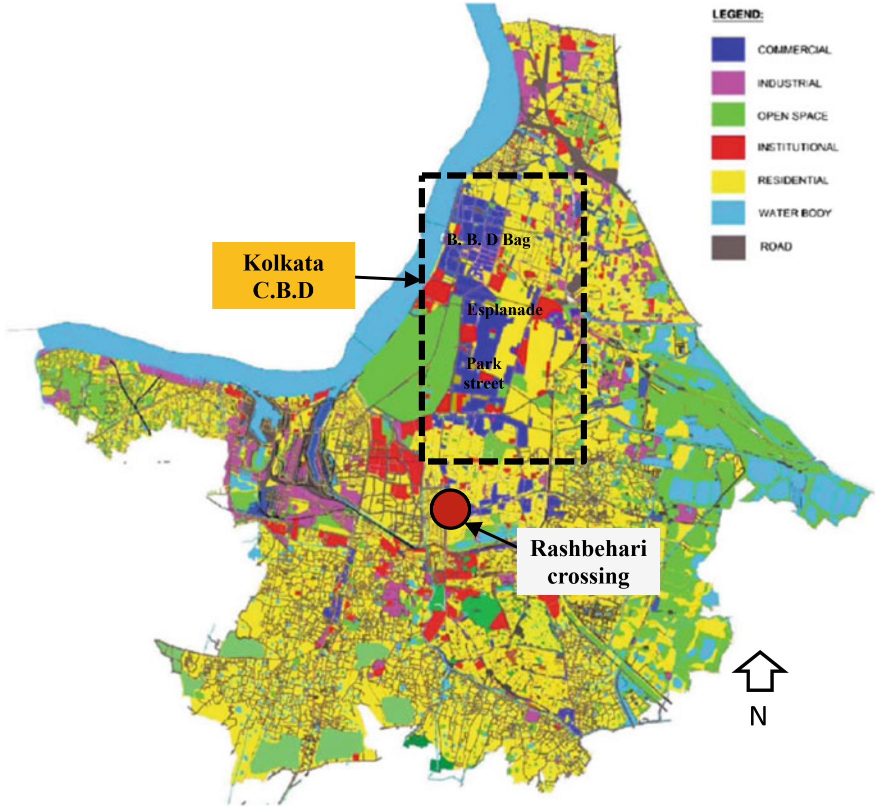

Fig. 4 Source: Plan for Development of Kolkata Municipal Corporation Area—2025, March 2007) (Unpublished report) Not to scale

is maximal towards Hazra intersection than any other directions since the CBD is also located towards north (Table 4). Paratransit modes observed in Rashbehari intersection are taxi and AR. The number of taxis is more to direction—7 i.e. from Hazra to Tollygunge, and heavy AR movement is found to direction—12, i.e. from Deshapriya Park to Chetla (Fig. 3 and Table 3). These vehicles amounted to 7062 passenger car units (PCU). As per IRC Standards (IRC 1990), PCU of car, taxi, AC bus, N-AC bus, minibus, bicycle, motor bike, AR, heavy vehicle and police van are considered to be 1, 1, 2.2, 3.7, 2.2, 0.4, 0.75, 2, 2.2 and 1, respectively. To assess total number of passengers transferred during this one-hour peak, average ridership of taxis and cars in office time (i.e. peak hour) is considered to be 2 as revealed in the CCTV footages, while cycle and bike are considered to be one and two, respectively. Respective ridership of AC bus, N-AC bus and minibus at the weekday peak hour is found to be approximately 60, 60 and 50, including the standing passengers. This transforms to around 44,104 passenger travel through Rashbehari intersection during peak hour of any weekday.

Fig. 5 Source: Google Map

Vis-à-vis, one-hour morning peak period traffic count during weekend at the Rashbehari intersection revealed movements of 693 taxies, 713 cars, 99 numbers of 30-seater AC buses, 239 numbers of 34-seater N-AC buses, 42 numbers of 27-seater minibuses, 46 bicycles, 228 bikes, 295 ARs, 36 heavy vehicles (HV) and 1 police van (PV).

Vehicle density is fewer towards Hazra on weekends compared to weekdays, since people's need to travel to CBD reduces on weekends (Table 5). These vehicles amounted to 3460 passenger car units (PCU). To evaluate total number of passengers transferred during this one-hour off peak, passenger capacity of taxis and cars is once again considered to be 2 as predicted from the CCTV footages, while cycle and bike are considered to be one and two, respectively. No standing passengers are observed in AC, N-AC and minibus in weekend, but most of the seats are seen to be occupied. Therefore, the seating capacity of those vehicles is considered to measure passenger capacity. This estimated to around 16,802 passengers off-peak hourly transfer through Rashbehari intersection during weekend (Sunday).

6.4 Controlling Measures

Three controlling experiments have been planned to study how paratransit modes are instrumental in affecting the efficiency of Rashbehari intersection during morning weekdays in peak hours (Table 6). To perform this experiment, some variables are used which are indicators of road congestion, namely traffic volume, PCU load,

Table 4 Number of vehicles towards different directions at Rashbehari intersection at peak time

Traffic flow directions		Number of										Total	Total vehicles in different directions
		Taxi	Car	AC bus	N-AC bus	Minibus	Cycle	Bike	AR	HV	PV		
To Hazra	1	135	595	12	102	31	21	273	38	20	14	1241	**1824**
	2	19	48	0	6	1	9	32	36	5	1	157	
	3	55	146	9	72	14	5	76	37	8	4	426	
To Deshapriya	4	99	255	8	62	16	12	114	26	8	8	608	1650
	5	122	324	8	37	17	47	203	170	20	6	954	
	6	4	27	3	24	3	1	17	3	4	2	88	
To Tollygunge	7	325	489	18	66	30	0	44	35	7	8	1022	1231
	8	44	90	0	0	0	3	0	0	1	0	138	
	9	18	15	4	15	4	0	4	8	1	2	71	
To Chetla	10	3	34	0	6	0	1	12	6	0	1	63	592
	11	0	0	0	0	0	0	0	0	0	0	0	
	12	25	130	0	1	5	5	50	311	0	2	529	

Bold value indicates the highest digit of the particular column

Table 5 Number of vehicles towards different directions at Rashbehari intersection at off-peak time

Traffic flow directions		Number of										Total	Total vehicles in different directions
		Taxi	Car	AC bus	N-AC bus	Minibus	Cycle	Bike	AR	HV	PV		
To Hazra	1	68	117	6	44	10	0	44	4	9	1	303	523
	2	27	7	0	0	0	5	1	3	0	0	43	
	3	70	49	23	20	3	3	9	0	0	0	177	
To Deshapriya	4	70	39	23	78	8	8	8	0	0	0	234	712
	5	23	129	0	0	5	14	64	129	5	0	369	
	6	29	34	13	29	0	0	0	4	0	0	109	
To Tollygunge	7	322	208	27	44	13	0	37	10	13	0	674	**794**
	8	22	34	0	0	0	0	14	6	0	0	76	
	9	14	7	4	7	4	0	4	4	0	0	44	
To Chetla	10	8	15	0	3	0	0	0	0	6	0	32	365
	11	0	0	0	0	0	0	0	0	0	0	0	
	12	41	75	3	10	0	17	48	136	3	0	333	

Bold value indicates the highest digit of the particular column

Table 6 Different planning schemes for peak time

Experiment	Taxi	Car	AC bus	N-AC bus	Minibus	Cycle	Bike	AR	HV	PV	Total vehicle	Total PCU	Total passenger capacity
0	849	2153	62	391	121	104	825	670	74	48	5297	7062	44,104
1	0	2153	68	391	121	104	825	670	74	48	4454	6260	44,104
2	849	2153	62	438	135	104	825	0	74	48	4272	6116	44,104
3	0	2153	68	427	132	104	825	0	74	48	3830	5108	44,104

travel time, speed, flow rate and density. Peak hourly moving passenger is a constant input. So is passenger capacity of each kind of vehicles. Only taxis are dropped in Experiment 1. In Experiment 2, AR is restricted. In Experiment 3, all paratransit passengers are forced to use some transit mode disallowing paratransit to ply. Let Experiment 0 be an identifier for the present situation. During the observed time, it is found that 4% passengers travel by taxi, 7% travel by AR. As per the fare structure of Kolkata public transport vehicles, the minimum fare of AR, minibus and N-AC bus is more or less comparable which is Rs. 10 or less. Minimum fare of AC bus is around Rs. 20, and minimum fare of taxi is above Rs. 25 (The Times of India 2017; The Telegraph 2014; www.bengalspider.com/resources/6203-A-complete-list-new-bus-taxi-fares Kolkata.aspx). Based on this economic criterion, AR passengers are transferred to minibus and N-AC bus, whereas the taxi riders are shifted to AC bus in the alternative experiments. 4% N-AC bus is increased in Experiment 1. No changes are done for N-AC and minibus in this experiment, and only taxi is omitted but AR is permitted to ply. In Experiment 2, 11% N-AC bus and 1% minibus are increased to accommodate the additional AR passengers (Table 7). The allotment of the passengers to N-AC bus and minibus is done based on the existing percentage of passengers travel by these two modes. Among the N-AC and minibus passengers, 79% travel by N-AC bus and 21% travel by minibus. In Experiment 3, 4% AC bus, 4% N-AC bus and 1% minibus are increased to accommodate additional riders. Table 7 shows the reduction in PCU demand that takes place through these controlling measures.

24%, 29%, 42% decrease in number of vehicles and 13%, 28%, 40% decrease in PCU load are found in experiments 1, 2 and 3 (Table 8). These measures can certainly reduce PCU demand, thereby enhancing speed, air quality and noise quality.

Likewise, three controlling experiments are also conducted for weekend between 9 and 10 AM as shown in Table 9 with estimated 16,802 passengers. To analyse average travel time at peak and off-peak time, 30 randomly selected vehicles are observed from CCTV footages of weekday morning peak time and weekend morning off-peak time. Average travel time during peak time and off-peak time to cross 50 m (*L*) from point P to point *Q* (Fig. 3) of Rashbehari intersection is revealed as 11 s (TT_j) and 7 s (TT_f) (Table 10). From the variations, one can clearly understand travel time of transit vehicles since they are much bigger than that for self-driven modes. The paratransit modes could go either way; they could wait to fill in vacant seats, or, rush for the next trip, if full.

Table 10 further reveals that the weekend mean travel time drops but the variance is comparable to weekdays due to presence of transit and paratransit modes. A reduction in paratransit on the arterial roads could alleviate this situation in two ways: the transit modes gather its share of passengers faster, reducing delay, and secondly, the drop in PCU on road enhances speed, air quality and noise quality, as shown in the experiment in Table 9.

From Eqs. (1) and (2) (Papacostas et al. 1993), speed at off-peak time (u_f) and peak time (u_j) has been calculated which were identified to be 25 km/hr and 17 km/hr, respectively. Flow rate at off-peak time (q_f) and peak time (q_j) shown in CCTV footages is 3460 pcu/hr and 7062 pcu/hr, respectively.

Table 7 Outcomes from different experiments at peak time

Experiment	AC bus				N-AC bus				Minibus			
	No.	% of passengers	Change (%)	% of AC bus	No.	% of passengers	Change (%)	% of N-AC bus	No.	% of passengers	Change (%)	% of Minibus
0	62	8	0	1	391	53	0	7	121	14	0	2
1	83	12	4	2	391	53	0	7	121	14	0	2
2	62	8	0	1	479	60	4	11	151	15	1	3
3	83	12	4	2	434	58	4	11	134	15	1	3

Table 8 Changes of PCU in different experiments at peak time

Experiment	Total vehicles	Change in total vehicles	% Change in total vehicles	Total PCU	Change in PCU	% Change in PCU
0	5297	0	0	7062	0	0
1	4469	−828	−24	6226	−836	−13
2	4746	−551	−29	5512	−1516	−28
3	3855	−1442	**−42**	5042	−2021	**−40**

Bold value indicates the highest digit of the particular column

Density at off-peak time (k_f) and peak time (k_j) is measured to be 138 pcu/km and 415 pcu/km using Eqs. (3) and (4). It is observed that density is decreasing in all experiments. During peak time, density is decreasing to 12%, 22% and 29% in experiments 1, 2 and 3. Density is decreased to 17%, 13% and 31% in experiments 1, 2 and 3 during off-peak time (Table 11).

7 Conclusions

It is found in all the synthetic experiments that number of vehicles and PCU load are decreasing after omitting the paratransits from the intersection. Maximum decrease of number of vehicles, PCU load, and road density is clearly evident in Experiment 3. In Experiment 1 decrease of PCU load is 13% whereas in Experiment 2 decrease of PCU load is 28% during peak hours. Further, during weekday peak time, decrease of density is higher in Experiment 2 than Experiment 1. Therefore, it could be inferred that better reduction in PCU load and road density can be achieved by enacting control over AR than taxi. Definitely the families that sustain on paratransit require an alternative livelihood so that all stakeholders find Kolkata equally vibrant. This job is taken up in the full research with temporal route manipulation.

Table 9 Different planning experiments for off-peak time

Experiment	Taxi	Car	AC bus	N-AC bus	Minibus	Cycle	Bike	AR	HV	PV	Total vehicle	Total PCU	Total passenger capacity
0	693	713	99	239	42	46	228	295	36	1	2392	3460	16,802
1	0	713	145	239	42	46	228	295	36	1	1731	2869	16,802
2	693	713	99	269	49	46	228	0	36	1	2133	2994	16,802
3	0	713	145	269	49	46	228	0	36	1	1471	2402	16,802

Table 10 Travel time at peak and off-peak time

Sequence no. of observed vehicles	Travel time (TT_j) (sec) in weekday morning peak hour for 50 m stretch	Travel time (TT_f) (sec) in Sunday morning off-peak hour for 50 m stretch	Sequence no. of observed vehicles	Travel time (TT_j) (sec) at weekday morning peak hour for 50 m stretch	Travel time (TT_f) (sec) at Sunday morning off-peak hour for 50 m stretch	Sequence no. of observed vehicles	Travel time (TT_j) (sec) at weekday morning peak hour for 50 m stretch	Travel time (TT_f) (sec) at Sunday morning off-peak hour for 50 m stretch
1	9	6	11	11	5	21	12	5
2	10	3	12	10	12	22	13	5
3	10	5	13	8	8	23	14	7
4	10	7	14	10	6	24	12	10
5	8	4	15	12	4	25	15	10
6	11	9	16	7	7	26	11	9
7	9	9	17	8	12	27	15	8
8	15	11	18	10	7	28	11	7
9	16	6	19	11	6	29	9	6
10	11	8	20	9	8	30	9	9
Average TT (sec) in peak time			10.8667		Average TT (sec) in off-peak time			7.3
Average TT (hr) in peak time			0.003		Average TT (hr) in off-peak time			0.002
Standard deviation in peak time			2.33		Standard deviation in off-peak time			2.321

Table 11 Flow rate, density and changes of density at different experiments on peak and off-peak time

Experiment	Flow rate at peak time (pcu/hr)	Flow rate at off-peak time (pcu/hr)	Density at peak time in pcu/km	% of change in density at peak time	Density at off-peak time in pcu/km	% of change in density at off-peak time
0	7062	3460	415	0	138	0
1	6226	2869	366	−12	115	−17
2	5512	2994	324	−22	120	−13
3	5042	2402	297	**−29**	96	**−31**

Bold value indicates the highest digit of the particular column

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Feasibility Study and Demand Estimation for Ferry Service from Hazira to Alang

Jigar Patel, Trupal Patel, Maitri Shah, Mehul Patel, Omkar Bidkar, and Gaurang Joshi

Abstract India has an extensive network of inland waterways in the form of rivers, canals, backwaters and creeks. The total navigable length is 14,500 km, out of which about 5200 km is of the river and 4000 km is of canals that can be used by mechanized crafts. Water transportation by waterways is highly under-utilized in India compared to other large countries. So, it causes unsustainable transport and more pressure on road and rail. In this study, demand estimation of regional transport from Surat to Saurashtra region is carried out with Do-Nothing condition and after applying ferry service. Home interview survey is carried out in particular zones of Surat, and data collected are income, vehicle ownership, working members for each household and travel time and travel cost of current mode of travel from Surat to Saurashtra region. Stated preference survey is carried out with designed travel time and travel cost for ferry service to check behaviour of people to shift to ferry or not. Multinomial logit model (MNL) is applied by considering travel time, travel cost, income and vehicle ownership as variables, and it is found from model that people will shift to ferry services with designed travel time and cost.

Keywords Ferry · MNL · Travel time · Travel cost

1 Introduction

Source of water transportation in India is widely available such as rivers, lakes and canals. Total 14,500 km of waterway is existing in India. Waterways are mostly used by village-side people in historic period. Development of roads and railways in India in past few decades is one of the major reasons to reduce the water transportation in India. In past period, the government has also not taken sufficient steps to develop water transportation. So, in India, water transportation is under-utilized. There are only few states in India, namely Assam, Goa, Kerala, Mumbai and West Bengal, and some creeks in coastal areas where water transportation is running in small

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percentage. Water transportation in state of West Bengal is highest than any other states. In state of West Bengal, navigable length is about 97% of total length available. In state of Gujarat, total length is 15.62% of total available length for water transportation. India's transport sector is a rapidly growing sector and contributes 6.4% to the GDP of the country. Intercity transport is mainly met by road (88%), rail (11%) and a limited share of air transport. In Gujarat state, road transportation is major from Surat to Saurashtra region.

2 Need of Study

People travel from Surat to Saurashtra region by road and rail. Egress is not possible to the most of places in Saurashtra region by railway. Due to the long distance to travel, travel time is high which leads to the high cost, high fuel consumption, larger amount of pollution, larger accident rate, traffic jams, delay at intersection and tax toll booth, etc. Government buses are not providing proper services, and private buses are having high travel cost which is unaffordable from people of middle- and low-income group. Boundary of Gujarat state is covered by seashore of large length which is much higher than other states. Water transportation is sustainable option for transportation than any other option with less travel time, travel cost and less congestion. So, it is necessary to use ferry transport as option for transportation from Surat to Saurashtra region.

3 Methodology

Figure 1 shows bar chart for methodology

Problem definition is the first step of the study. After defining problem, objective and scope of the study is determined. Objective and scope will give the idea regarding literatures which are required to be studied. Study area is defined after literature review. After defining study area, data collection is done which involves both pilot survey and main survey. When data collection is done, logit model is applied for mode choice. Finally, result and conclusion is drawn.

4 Objectives of the Study

1. To understand the current travel pattern between two regions, Surat and Saurashtra regions, which includes current modes, current travel time and cost required.
2. To study the factors affecting in mode choice for different people between two regions.

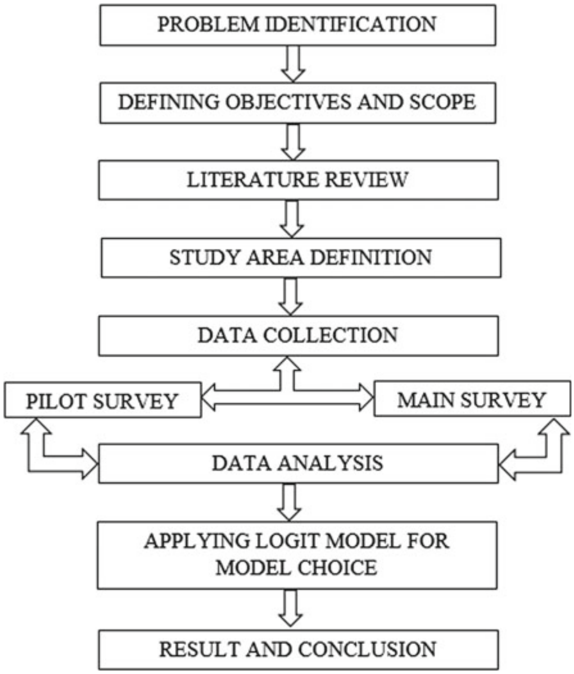

Fig. 1 Bar chart showing methodology

3. To study feasibility of study by carrying out stated preference survey.
4. To apply mode choice modelling and to calculate final demand.

5 Literature Review

Praveen and Jegan (2015) explored transport and trade as two broad service sectors of inland water resources. An attempt is made to find out the key issues and challenges from this sector with the evolving understanding of Indian inland water transportation system. Study explains the background of inland water transport sector in India along with the discussion of issues and challenges faced by the same. In this paper, it is also stated that co-operation and co-ordination between inter-state governments is a strategic element to expand the network of inland water transport system in India beyond state boundaries.

Nagabhatla and Jain (2013) presented a comparison of river transportation system with surface road–rail network to explain prospective contribution of IWT for green economic growth. In this paper, it is found that the prospect of inland navigation looks promising, wherein issues on infrastructural gaps and institutional support are addressed suitably.

Sulaiman and Kader (2011) studied the hybridization of transportation system by considering comparative advantage and use of benefit provided by the two seas of information and environmental technology that is currently dynamically transforming world in order to maximized use to improve our transportation system, incorporating new inland water transportation system that will efficiently in an innovative manner link our cities with existing transportation by providing quality facilities and services for people at affordable rate through system hybridization and integration.

Pauli (2010) found that inland navigation is the most environmentally friendly and sustainable mode of transport, with fewer harmful emissions and less consumption of resources than road or rail transport, and also concluded that a new approach to the development of transport policy is needed and that this approach has to ensure that important factors determining sustainability are integrated within a single policy.

Broils (1967) discussed hybridization of transportation system by considering comparative advantage and use of benefit provided by the two seas of information and environmental technology that is currently dynamically transforming our world in order to maximized use to improve our transportation system, incorporating new inland water transportation system that will efficiently in an innovative manner link our cities with existing transportation by providing quality facilities and services for people at affordable rate through system hybridization and integration. This will include infrastructure that will link logistics equipment together for a better management, control and the reality of putting concept of togetherness into practice to achieve greater things that will solve transportation problem.

Nagabhatla and Jain (2013) explored transport and trade as two broad service sectors of inland water resources. On the other hand, attempt is made to link the services from this sector with the evolving understanding of green economy. The first section explains the historical background of inland water transport (IWT) sector along with the discussion of options that can channel its optimum efficiency. The authors state that co-operation and co-ordination between trans-boundary governments is a strategic element to expand the network of inland water transport beyond national boundaries. The second section presents a comparison of river transportation system with surface road–rail network to explain prospective contribution of IWT for green economic growth.

6 Study Area Definition

Two regions are considered for study: one is Surat and other is Saurashtra. Surat is fast-growing industrial city in Gujarat having commercial base. It is the second largest city in Gujarat state and is situated on bank of River Tapti. It is known for its textile manufacturing, trade, diamond cutting and polishing industries, intricate sari works, chemical industries and the gas-based industries established by leading industry houses such as ONGC, Reliance, ESSAR and Shell. Having strong industrial base urbanization process is of higher order; as such, it is prime city in South Gujarat region. Population of Surat is 45 lakhs. Vehicle growth rate of Surat is high in present decades. Saurashtra is the region which consists of Junagadh, Jamnagar, Morbi, Rajkot, Porbandar, Amreli, Gir Somnath, Bhavnagar and Surendranagar, etc. Saurashtra region is source-deficient region. Opportunities available for job and business are less in Saurashtra region. So most of people will travel to Surat region for job and business also.

7 Data Collection

It is found that most of the passengers travel from Surat to Saurashtra region by Private Bus, Government Bus and Railway. Percentage of people travelling by Private Bus and Government Bus is more. Government buses are GSRTC buses. Private buses involve luxurious buses by different private agencies. Private and government buses are available on daily basis. Railway is also available from Surat to Saurashtra up to Rajkot. So it is necessary to study attributes of existing modes which are travel time and cost. Data collection is carried out by home interview survey of different zones of Surat, namely Varachha, Katargam and Adajan, where people of Saurashtra region are residing. Home interview survey involves number of family and working members, income, vehicle ownership, destination at Saurashtra region, current mode, current travel time and cost by existing different modes. Ferry travel cost is designed which involves cost required for accessibility of ferry system from origin of travel in Surat (access), in-ferry travel cost and cost required to reach to the destination in Saurashtra region from ferry system (Egress). Ferry travel time is calculated in the same way as ferry travel cost which involves time required to reach access time, in-vehicle time and time required to reach the destination. Stated preference survey is conducted to check whether passenger will shift from existing travel modes to Ferry with designed ferry travel time and travel cost as criteria. Data samples collected are 0.05% of total number of passengers travel to and from Surat and Saurashtra region.

8 Model Development

Data are collected from different zones of Surat, and logit model is applied. Basic concept of logit model is given below.

8.1 Logit Model

It is believed that the stochastic model is better than the deterministic model, as the deterministic model of choice may be confined in its replication to the real-life situations. On the other hand, behaviour of individuals may not always consider the judicious rules of choice exactly. Also, the random behaviour of a traveller cannot be predicted from the deterministic model. It is also true that the potential travel may not have correct information about the transportation system and the availability of the number of alternatives modes to be selected. It can be said that a good model is the one where choice functions consider a random function which takes different values with certain probabilities.

The perceived utility curve $U(.)$ can be called as random function and expressed as random utility model, as given under:

$$U(i) = V(i) + e(i)$$

where $U(i)$ is the choice function for the alternative i, $V(i)$ is the deterministic function of the attributes of (i) and e(i) is a stochastic component, a random variable that follows some distribution.

It may be said that the individual will select an alternative (i) if the perceived U(i) of alternative (i) is the largest of all such values. Therefore, the probability that (i) is chosen can be given as

$$P(i) = P[U(i) > U(j)]\text{for all } j$$

It may be further written as

$$P(i) = P[V(i) + e(i) > V(j) + e(j), \text{for all } j \neq i]$$
$$P(i) = P[e(i) < V(i) - V(j) + e(i), \text{for all } j \neq i]$$
$$P(i) = \int [[F[V(i) - V(j) + e(i)] \text{ for all } j \neq i]] f_i(\phi) \cdot d\phi$$

where $F(.)$ is the joint distribution function (Sriraman 2010) of [e(i), e(j), …] terms of the alternatives and $f_i(f)$ is the marginal density function of e(i).

Random components of a choice utility function (as expressed above) are all independent. The expression of logit model can also be derived from Gumbel distribution.

$$Fe^x = \mathrm{e}^{-\theta e^{-x}} \theta > 0; -\alpha < x < \alpha$$

Using the Gumbel distribution, the following expression is derived:

$$\frac{1}{\sum_{j=1} \mathrm{e}^{V(f)-V(i)}}$$

$$\frac{\mathrm{e}^{v(i)}}{\sum_{j=1} j^{\mathrm{e}^{v(i)}}}$$

This is known as multinomial logit model.

8.2 *Travel Demand Estimation*

Feasibility of ferry service is determined by travel demand estimation. Analysis of existing modes is done. It is found that three major modes with which passengers travel from Surat to Saurashtra region are Private Bus, Government Bus and Railways. In order to find travel demand, logit model is applied for Do-Nothing condition by using NLOGIT software.

8.2.1 Travel Demand Estimation (Do-Nothing Condition)

Demand estimation for before applying ferry service is nothing but Do-Nothing condition. Multinomial logit model (MNL) is applied for Do-Nothing condition. For Do-Nothing condition, there are three modes available in choice set of passenger, namely Government Bus, Private Bus and Railways. Travel characteristics and household characteristics are considered for model development. Travel characteristics include travel time and travel cost of journey. Household characteristics include vehicle ownership and income. All variables are considered alternate specific variables, and multinomial logit model is applied. The following equations are framed for Do-Nothing condition:

$$
\begin{aligned}
&U(\text{Government Bus}) = \text{a1} * \text{TT} + \text{a2} * \text{TC} + \text{a3} * \text{INCOME} + \text{a4} * \text{VO} \\
&U(\text{Private Bus}) = \text{b0} + \text{b1} * \text{TT} + \text{b2} * \text{TC} + \text{b3} * \text{INCOME} + \text{b4} * \text{VO} \\
&U(\text{Railway}) = \text{c0} + \text{c1} * \text{TT} + \text{c2} * \text{TC} + \text{c3} * \text{INCOME}
\end{aligned}
$$

8.2.2 Model Validation

Table 1 shows model validation for Do-Nothing condition.

8.2.3 Utility Equations

The following are utility equations for Do-Nothing condition before ferry service:

$$
\begin{aligned}
&U(\text{Government Bus}) = (-0.4064 * \text{TT}) - (0.00113 * \text{TC}) \\
&\quad - (0.133289\text{D} - 04 * \text{INCOME}) - (0.9212 * \text{VO}) \\
&U(\text{Private Bus}) = (-7.93) - (0.2235 * \text{TT}) - (0.00046 * \text{TC}) \\
&\quad + (0.326763\text{D} - 04 * \text{INCOME}) + (0.97 * \text{VO}) \\
&U(\text{Railway}) = (-10.32) - (0.629 * \text{TT}) - (0.00066 * \text{TC}) \\
&\quad - (0.195656\text{D} - 04 * \text{INCOME})
\end{aligned}
$$

Table 1 Model validation for Do-nothing condition

Variables	Coefficient	$P[\|Z\| > z]$
A1	−0.4064	0.049
A2	−0.00113	0.009
A3	−0.133289D-04	Fixed parameter
A4	−0.9212	0.0000
B0	−7.93	0.0000
B1	−0.2235	0.0000
B2	−0.00046	Fixed parameter
B3	0.326763D-04	Fixed parameter
B4	0.97	Fixed parameter
C0	−10.32	Fixed parameter
C1	−0.629	Fixed parameter
C2	−0.00066	Fixed parameter
C3	−0.195656D-04	Fixed parameter

8.2.4 Model Description

Utility of Government Bus decreases by 0.4064 with unit increase of travel time, 0.00113 with unit increase of travel cost, 0.133289D-04 with unit increase of income and increases by 0.9212 with unit increase of vehicle ownership. Utility of Private Bus decreases 0.2235 with unit decrease of travel time, 0.00046 with unit increase of travel cost, increases by 0.326763D-04 with unit increase of income and increases by 0.97 with unit increase of vehicle ownership. Utility of Railway decreases by 0.629 with unit increase of travel time, decreases by 0.00066 with unit increase of travel cost and decreases by 0.195656D-04 with unit increase of income.

8.2.5 Modal Shares

Modal shares are calculated from utility equations. Table 2 shows modal shares for different modes. From Table, it is found that 1.5% passengers travel by Government Bus, 98% of passengers travel by Private Bus and 0.5% of people travel by Railways. There are totally 35,000 passengers travelling from Surat to Saurashtra region. So, 550 passengers will travel from Surat to Saurashtra region by Government Bus, 34,300 passengers by Private Bus and 175 passengers by Railways.

Table 2 Modal share for Do-nothing condition

Mode	Probability	Passengers
Government Bus	1.5	525
Private Bus	98	34,300
Railways	0.5	175

8.3 *Travel Demand Estimation (After Ferry Service)*

Demand estimation is done after applying ferry service; when ferry service is applied, then there are four modes with which passengers travel which are Government Bus, Private Bus, Railways and Ferry. Similarly, travel characteristics and household characteristics are considered and model is applied. Variables considered are travel time, travel cost, income and vehicle ownership. For model development, the following utility equations are formed which are as follows.

$$U(\text{Government Bus}) = \text{a1} * \text{TT} + \text{a2} * \text{TC} + \text{a3} * \text{INCOME} + \text{a4} * \text{VO}$$
$$U(\text{Private Bus}) = \text{b0} + \text{b1} * \text{TT} + \text{b2} * \text{TC} + \text{b3} * \text{INCOME} + \text{b4} * \text{VO}$$
$$U(\text{Railways}) = \text{c0} + \text{c1} * \text{TT} + \text{c2} * \text{TC} + \text{c3} * \text{INCOME}$$
$$U(\text{Ferry}) = \text{d1} * \text{TT} + \text{d2} * \text{TC}$$

8.3.1 Model Validation

Table 3 shows model validation for after ferry provision.

Table 3 Model validation for after ferry provision

Variables	Coefficient	$P[\|Z\| > z]$
A1	−0.1106	0.004312
A2	−0.0091	0.002211
A3	−0.440274D-04	Fixed parameter
A4	−0.5198	Fixed parameter
B0	−15.02	Fixed parameter
B1	−0.171	0.0000
B2	0.0085	0.0042
B3	0.171340D-04	Fixed parameter
B4	0.968	Fixed parameter
C0	−21.11	Fixed parameter
C1	−1.546	Fixed parameter
C2	−0.00182	Fixed parameter
C3	−0.476183D-04	Fixed parameter
D1	−1.2643	Fixed parameter
D2	−0.00116	Fixed parameter

8.3.2 Utility Equations

The following are the utility equations after applying ferry service:

$$
\begin{aligned}
U(\text{Government Bus}) &= (-0.1106 * \text{TT}) - (0.0091 * \text{TC}) \\
&\quad - (.440274\text{D} - 04 * \text{INCOME}) - (0.5198 * \text{VO}) \\
U(\text{Private Bus}) &= (-15.02) - (0.171 * \text{TT}) + (0.0085 * \text{TC}) \\
&\quad + (.171340\text{D} - 04 * \text{INCOME}) + (0.9688 * \text{VO}) \\
U(\text{Railways}) &= (-21.11) - (1.54 * \text{TT}) - (0.00182 * \text{TC}) \\
&\quad - (0.476183\text{D} - 04 * \text{INCOME}) \\
U(\text{Ferry}) &= (-1.26 * \text{TT}) - (0.0011 * \text{TC})
\end{aligned}
$$

8.3.3 Model Discussion

Utility of Government Bus decreases by 0.1106 with unit increase of travel time, decreases by 0.0091 with unit increase of travel cost, decreases by 0.440274D-04 with unit increase of income and decreases by 0.5198 with unit increase of vehicle ownership. Utility of Private Bus decreases by 0.171 with unit increase of travel time, increases by 0.0085 times with unit increase of travel cost, increases by 0.171340D-04 with unit increase of income and increases by 0.9688 with unit increase of vehicle ownership. Utility of Railways decreases by 1.54 times with unit increase of travel time, decreases by 0.00182 with unit increase of travel cost and decreases by 0.476183D-04 with unit increase of income. Utility of Ferry decreases by 1.26 times with unit increase of travel time and decreases by 0.0011 times with unit increase of travel cost.

8.3.4 Modal Shares

Modal share is calculated from utility equations. Table 4 shows modal shares for different modes. After ferry service provision, it is found that 1.2% people will travel by Government Bus, 17% people travel by Private Bus, 0.8% people travel by Railways and 81% people by ferry system from Surat to Saurashtra. There are

Table 4 Modal shares after ferry provision

Mode	Probabilities	Passengers
Government Bus	1.2	420
Private Bus	17	5950
Railways	0.8	280
Ferry	81	28,350

35,000 passengers travelling from Surat to Saurashtra region. So, 420 passengers travel by Government Bus, 5950 travel by Private Bus, 280 passengers by Railways and 28,350 passengers by Ferry.

9 Conclusion

From this paper, it is found that water transportation in India is very less which leads to the unsustainable transportation and behaviour of choosing travel mode not only affected by travel characteristics (travel time, travel cost) but also by household characteristics such as income and vehicle ownership. Passengers are travelling from Surat to Saurashtra region mostly by road. If proper ferry service is provided with good access from Surat and Egress to Saurashtra region, then people will shift to the ferry service. From model, it is found that probability of choosing Private Bus and Government Bus decreases and probability of choosing Ferry for transportation will increase.

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Planning and Usage Analysis of Bike Sharing System in a University Campus

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Abstract Bike sharing system is increasingly seen as a sustainable way of making short trips in a limited space like university campuses. This paper is the case study of a bike sharing system PEDL by Zoom Car that is introduced in Indian Institute of Science, Bangalore (IISc) campus. The main objective behind introducing this system is to reduce the vehicular movement inside campus and to create a safer, sustainable and pollution-free environment in the campus. In this paper, authors talk about the planning and design involved in launching the bike sharing system in the university campus. Further, a comprehensive analysis to identify the parking stations across the campus is shown in the paper. The importance of GIS in identifying these stations and accessibility analysis will be explained in this paper. A preliminary usage analysis and statistical analysis are carried out to estimate the OD matrix, peak hour trips and non-peak hour trips. The data is also used in estimating the factors like maximum/minimum number of trips per day, maximum round trips and also the trip durations which further help in understanding the bike usage pattern of the users. This being a new system in the campus has a limited data for analysis which is a limitation for this paper. The future work involves in collecting more data on demographics and to model the mode share in the bike sharing scenario in university campuses.

Keywords Bike sharing · University · Sustainable · Pollution · Bangalore · GIS

1 Introduction

India is one of the fastest urbanizing countries in the world, and the urban centres have a major part in improving nation's economy. Due to the growing economies of cities, employment is generated which leads to a lot of migration to the cities (Verma et al. 2018). The rapid growth in the economies resulted in urbanization and increased vehicle ownership leading to congestion and pollution. This scenario is affecting the

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peaceful and healthy environment in university campuses in India. Increased vehicles inside the universities are creating pollution, congesting the roads, increasing the local temperature making it difficult and dangerous to live and commute. There can be various measures that can be taken to curb these problems like restricting the usage of motorized vehicles inside the campus or by providing alternative sustainable transportation options like provision of mini buses and bicycle sharing system. It is becoming increasingly evident that to regulate the changing climatic conditions and reduction in available energy resources and increased costs shifting to a non-motorized transportation is a viable option. University campuses are spread across smaller areas which create shorter trips, and cycling can be seen as a good option for these trips.

In this paper, a case study of bicycle sharing system in the campus of Indian Institute of Science Bangalore (IISc) is showcased. Bicycle sharing system in this context refers to rental schemes, whereby IISc community can pick up, ride and drop off bicycles at numerous points across the campus—usually at designated parking stations. Bicycle sharing system called “Namma Cycle” was first introduced in IISc campus in the year 2012 by the government of Karnataka to promote sustainable transportation and reduce CO_2 emissions. Unfortunately, Namma Cycle ended its services in IISc after running for a year, and there has been a lot of private vehicle growth in campus for the past few years. As mentioned above, increased vehicle ownership in the university campus has led to congested roads during the peak hours. These events led to the re-introduction of the bicycle sharing system called PEDL by Zoom Car. The cycle sharing system PEDL by Zoom Car is launched with 100 cycles aiming to create a congestion-free, safer environment and modal shift from using private transport. The PEDL cycles have a smart lock system which can be unlocked from a mobile app. The locks also have an inbuilt GPS which tracks the movements of cycle and generate the fare as per the distance travelled.

2 Planning and Design

2.1 Identifying Parking Stations

The station placement plays an important role in maximizing the trips and benefits from being easily visible and located near to appropriate roads for biking. Stations are conveniently located, next to, but not in, major thoroughfares. The stations have been set up at all the entrances and exit gates of IISc campus and also at various locations across the campus facilitating users to move from one corner of the campus to another. GIS plays a major role in the app-based bicycle sharing system by letting the user know the number of cycles that are present at a certain station. Figure 1 shows few parking locations in IISc and the number of cycles available for usage at those locations. For this purpose, all the locations are geocoded and an accessibility analysis is done to obtain the best parking spot. Figure 2 shows all the parking locations across the IISc campus.

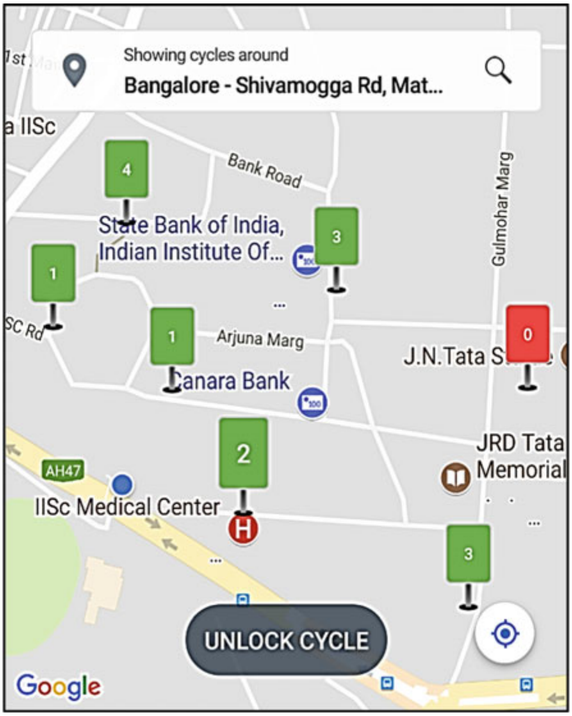

Fig. 1 Image showing the parking locations on map in PEDL app and availability of cycles at that station

2.2 *Accessibility Analysis*

Accessibility is the key driver in making the user pick up the cycle from the station and making a trip. This paper presents a simple GIS-based accessibility analysis in selection of the parking spots at various locations across the IISc campus. Figures 3 and 4 show the accessibility analysis carried out in fixing a parking spot near civil engineering (CE), computer science and automation (CSA), electrical engineering (EE) and Structures Laboratory. In this paper accessibility is calculated using least distance as the prime evaluation criteria. If there is more than one major trip production centre located close to each other, the parking spot is fixed at a location that is almost equidistant to all the production centres. A study conducted by Rahul and Verma (2014) suggests that the acceptable trip distance for walking when used as an access mode varies from 562 to 733 m for male and female depending on the age, education level and vehicle ownership. The same numbers are adopted and kept as an upper limit for choosing the best spot for parking during accessibility analysis. A parking spot has been fixed at the parking space in front of CE and CSA departments because the road beside the parking spot is one of the main accessed roads by the IISc community. GIS-based accessibility analysis is calculated from the nearest

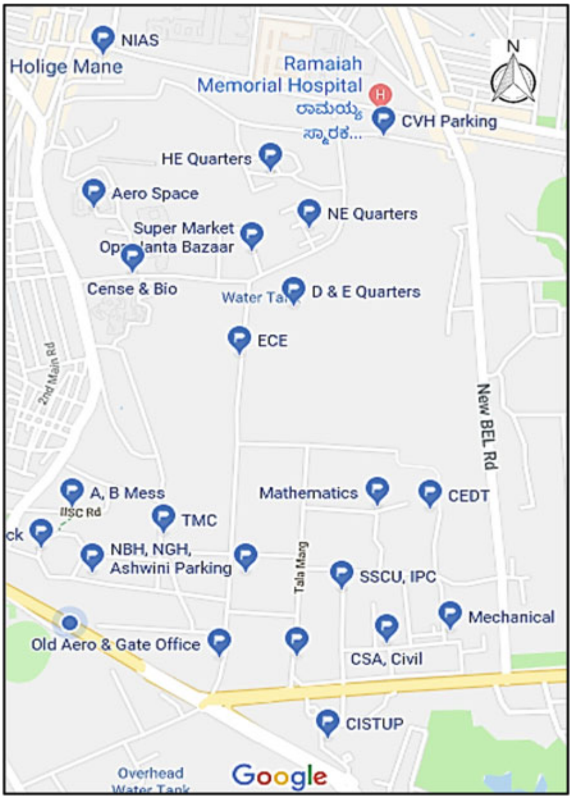

Fig. 2 Image showing PEDL locations of IISc

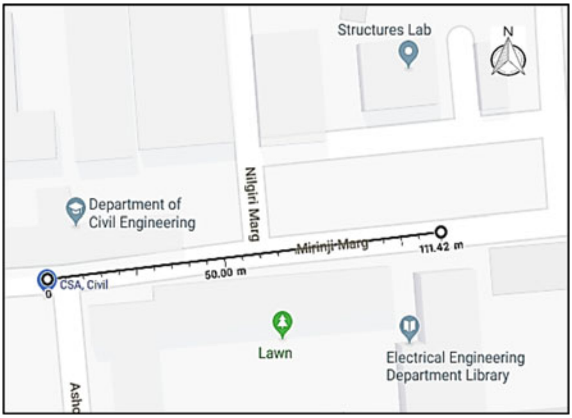

Fig. 3 Accessibility analysis of Civil/CSA parking spot and electrical engineering department

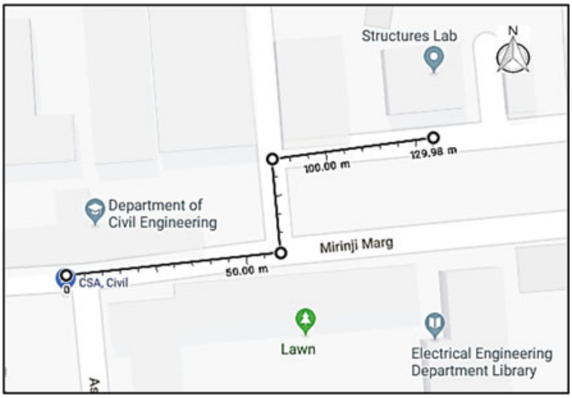

Fig. 4 Accessibility analysis of Civil/CSA parking spot and Structures laboratory

departments to this parking spot. From Fig. 3, it is observed that distance between electrical department and the parking spot at CE/CSA parking is 111.42 m and the distance from structures laboratory to CE/CSA parking spot is 130 m.

Similarly, accessibility analysis has been calculated for all the PEDL stations across the campus. The closest walking distance to the parking spot from a trip production centre is 37 m, and the farthest is 230 m from the trip production centre. These distances are within the acceptable walking trip distances as suggested by Rahul and Verma (2014).

3 Data Analysis

As mentioned earlier, PEDL bicycle sharing system was introduced in IISc campus on 8 January 2018. PEDL cycles are fitted with smart lock system which can be unlocked using a Quick Response code (QR code) or by manually entering the cycle code in the PEDL app. The cycles also have an inbuilt GPS system that tracks the movement of the cycles including start and end points based on the coordinates. The data analysis presented in this paper consists of the trips from February 8 to March 17. The stored data of each trip has the names of the locations where the trip started and trip ended, start time and end time of the trip, latitude and longitude of the start and end location. This information is used to develop the productions and attractions of all the PEDL stations. It is observed that there have been 13143 trips between February 8 and March 17 with an average of 355 trips per day with most number of rounds trips being recorded at biological sciences department. Figure 5 shows the productions and attractions of each PEDL station across the campus and Fig. 6 shows the bar chart of the same. The maximum number of trips are produced and attracted at Biological sciences parking spot because of the concentration of number of departments near the parking location as shown Fig. 7. The other reasons for more

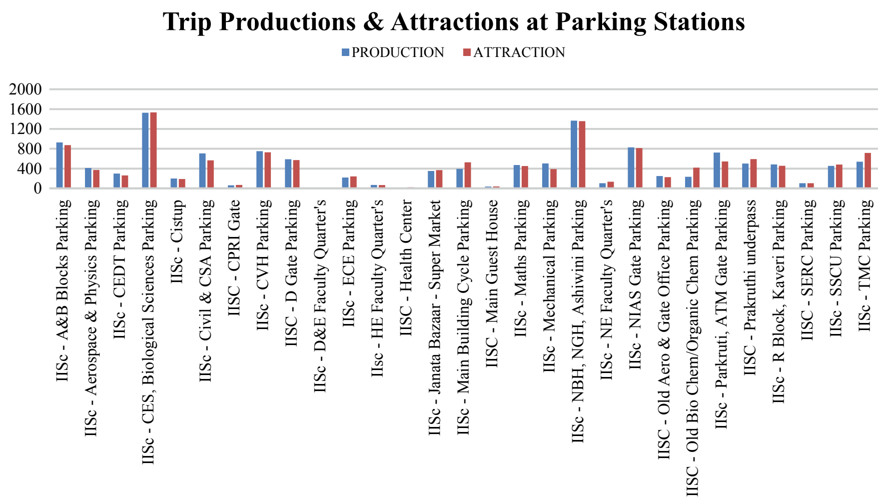

Fig. 5 Trip productions and attractions of all parking locations

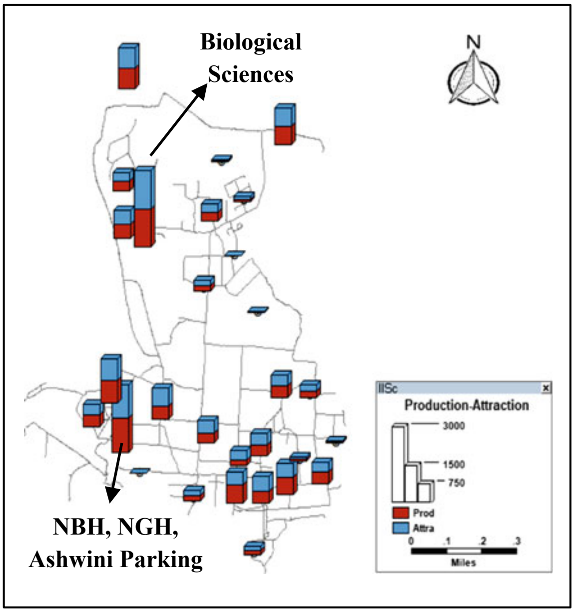

Fig. 6 Production-Attraction bar chart on IISc map

Fig. 7 Image showing biological sciences parking spot

number of trips from this location are that it is situated close to the gate of the campus where people can pick and drop the cycles easily.

The second major trips are produced and attracted from hostels parking. The hostels in the campus are usually the places from where trips are produced in the morning and attracted in the evening or night. The reason for this particular PEDL parking spot (NBH, NGH, Ashwini parking) having higher trips compared to other hostels is because of its location and occupancy of the hostels. This parking spot is located close to four other hostels as shown in Fig. 8, and two of these hostels have high occupancy levels compared to others. This trip data is recorded continuously from February 8 to March 17 round the clock, and it was observed that the maximum number of trips happens from 1 to 2 pm which is primarily because of lunch hours. Figure 9 shows that trips start to gradually increase from 6 am and the first peak is observed around 9 am. The reason is that the students and other members of IISc would be travelling to their respective work places using PEDL in the morning hours. The drop down after the peak is because of the working hours and the trip gradually decrease with respect to time of the day. A desired line diagram shown in Fig. 10 is plotted for the top 5 trip producing locations which show that the maximum trips are happening between NBH hostel parking spot and biological sciences parking spot followed by A and B hostel blocks to biological sciences again. Every parking location is arranged with a fixed number of cycles beyond which will be rejected for parking. There is a PEDL team which rearranges the cycles at these locations so that the new cycles can be parked. This is important because all the cycles should not be concentrated at one location making it difficult for other users to find a cycle at their location. The aim of introducing PEDL is to reduce the usage of motor vehicles inside campus and encourage cycling for short trips. About 51% of the trips that happened in IISc are for a short duration of 10 min and less where people travel from home/hostel to work place and vice versa. PEDL is also used as an access mode to

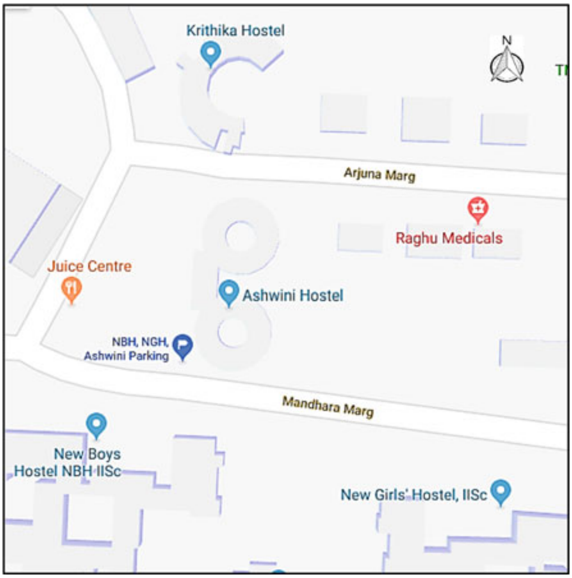

Fig. 8 Image showing the parking spot at NBH, NGH hostels

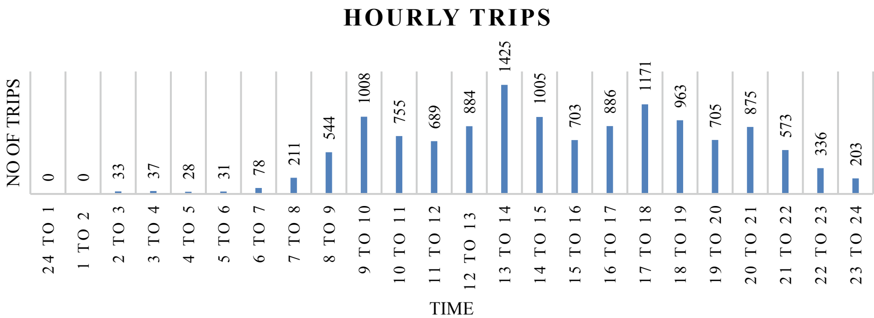

Fig. 9 Plot showing hourly trips on PEDL in IISc campus

reach the nearest bus stops outside the campus while the user has to park the cycle at the nearest parking station inside the campus. It is observed that some trips are longer than 420 min (7 h) as shown in Fig. 11 which could be because of low usage cost.

Bike sharing system is helpful for the visitors to the college campus who do not have an idea about the distances between locations they want to visit. Some universities are larger in terms of area compared to others which makes it difficult for some users who has met their needs at multiple places. Unlike the usual days, there will be special events and gatherings in colleges which allow a huge number of people from in and out of the campus. Bicycle sharing system can cater to their

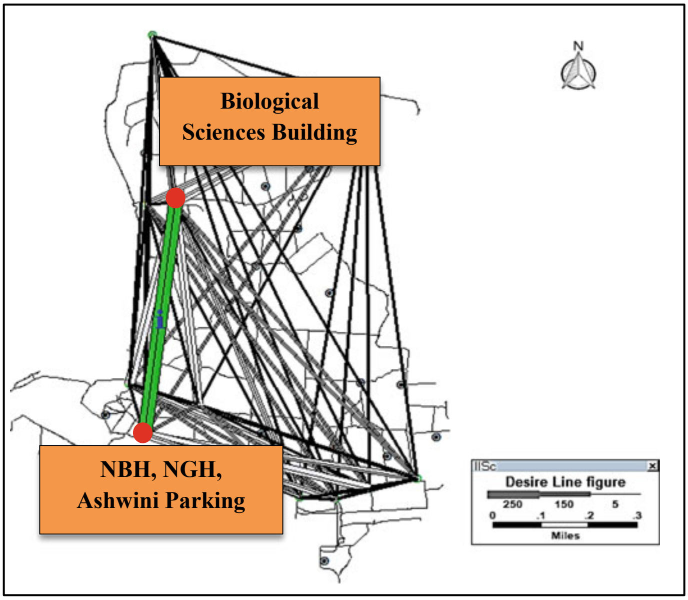

Fig. 10 Desire line diagram showing trips between parking stations

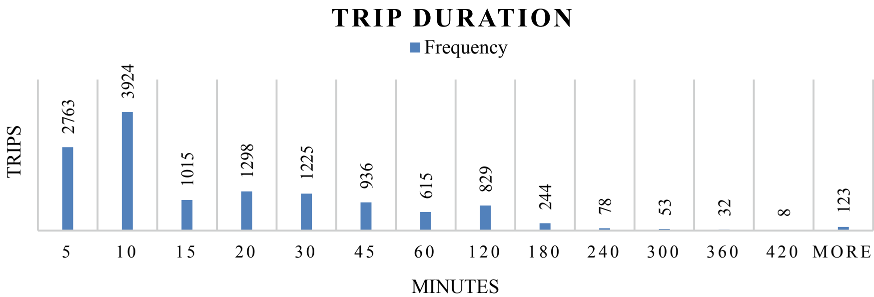

Fig. 11 Plot showing the duration of trips on PEDL

needs in such situations where the person can spend very little time pick bicycle and reach the destination. The daily trips in IISc from Feb 8 to March 18 are shown in Fig. 12. It is observed that on March 10 highest number of trips have been made and very low trips on 21 February. The reason for the surge in trips is because of a special event called OPEN DAY celebrated in IISc where the college opens its gates for the public. During this event, people used PEDL bicycles to visit as many locations as they can across the campus. The decline in trips on February 21 is because of the cycles going under maintenance which is important for a smooth ride. An average of 3.46 trips/day/cycle is observed from the data, and an average of 3.56 and 3.42 trips/day/cycle is observed for weekdays and weekends, respectively. Since there is

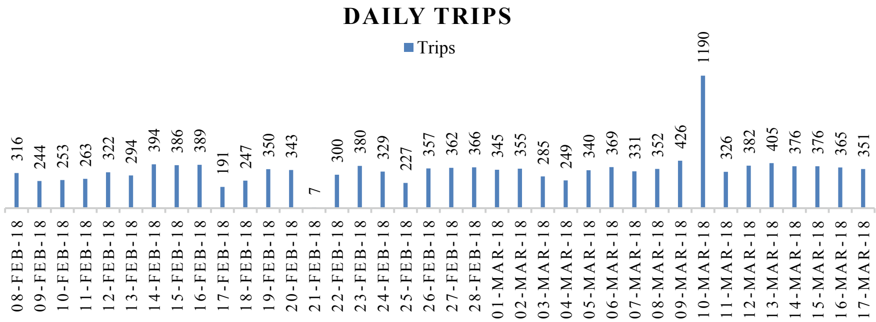

Fig. 12 Daily trips on PEDL from February 8 to March 17

one public event in the data set, the average trips on weekends are showing higher than usual 2.78 trips/day/cycle if the March 10 trips are replaced with trips as per the trend.

PEDL is a new initiative in IISc campus, and this study is based on a month and half data. The basic guidelines on selecting parking locations and trip information have been presented in this paper, and no analysis of mode share is shown, which is a limitation to this paper. The future work involves in collecting the demographic information of the users and estimating the mode share of vehicles in campus and develops appropriate measure to achieve more sustainability in the campus.

4 Summary

Bicycle sharing is the sustainable way to reduce emissions and congestion in college campuses. Indian Institute of Science Bangalore has introduced the smart bicycle sharing system called PEDL on 8 January 2018. The data analysis which is the basis for this paper is from 8 February to 17 March 2018. GIS is an important tool in identifying the locations for parking cycles for this app-based bicycle sharing wherein users can spot the nearest parking spot and check the availability of cycles. It can be inferred from this paper that parking spots have high visibility and usage if they are setup close to a gate and beside the road for easy usage. Strategically placing the parking spot surrounded by multiple trip producing hot spots involves in higher trips so the demand needs to be catered. It is difficult, if not impractical to place parking spots at every department or office in the campus. Accessibility analysis is an important driver in fixing a parking location with few departments or offices in its close proximity. This paper provides a short insight into GIS-based accessibility analysis which is used to fix the parking spot when there are multiple trip production centres. The parking spots at biological sciences department and NBH, NGH hostels are an example of strategic location of parking stations. It is observed that they have high number of trips produced from their stations and also more number of trips between these two stations. Cycling is primarily used for short distance trips as seen in earlier sections. It is observed that 51% of the total trips are within 10 min' duration while there are trips above 7 h as well. It can be inferred that low usage prices for bicycle sharing system encourage the users to use the cycle for longer hours also. In a college campus, it is not so uncommon to have events that involves in large gatherings filling roads and making vehicles difficult to move. It is observed that there is a huge surge, about 3 times the usual trips on March 10 which IISc celebrates as open day. In order to cater to the demand, additional cycles need to be provided in the campus so the visitors can reduce the usage of private vehicles inside the campus when the roads are congested. The peak hour for the trips is observed to be 1–2 pm which is usually a lunch break. It is observed that the peak hours are during the break hours and the bicycle sharing management should rearrange the cycles at the parking stations and fill the slots that are entitled at the particular station. It is suggested that with proper infrastructure for bicycling bicycle sharing can be introduced not just in university campuses but also across the city.

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Fuel Loss Estimation Due to Idling Phase of Signalized Intersection

Hardik Gosar, Ravi Sekhar Chalumuri, and Manoranjan Parida

Abstract Sustained economic development and expanding road network have led to rapid increase in the number of motorized vehicles in India. The total number of registered motor vehicles was 230 million in 2016 with a growth rate of about 10.7% annually. Presently, the transport sector accounts for nearly 18% of the total energy consumed in India. Diesel and petrol are strategic commodities, and they play a vital role in the socio-economic development, and nearly, 98% of the transportation energy needs are met through petroleum products. Rising crude oil prices and uncertainty about their supply can have negative implications for the Indian economy. In order to reduce this uncertainty, it is important to plan and use the resources judiciously. Thus, energy conservation has become a vital national concern. One of the ways to conserve fuel is minimize its wastage. The simplest way might just be to switch off engines while the vehicles are in idling condition waiting for their turn to cross the intersection at signals. Therefore, it is important to quantify the fuel loss due to idling delay. On the basis of the idle time and idle fuel consumption rate, the loss of fuel is estimated and converted in monetary terms. The outcomes of this study may help in proposing suitable remedial measures to reduce idling fuel consumption.

Keywords Signalized intersection · Idling · Fuel · Heterogeneous traffic

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1 Introduction

Intersections are an important part of any road network and play vital role in the efficiency of the network. It is a location where two or more roads carrying traffic in different directions cross at grade and compete with each other for the use of common space in between them. Generally, the traffic flow at intersections is always chaotic leading to problems of safety and lower efficiencies. Intersections are generally the bottleneck in the road network and therefore are the common cause of jams and delays. Various strategies are employed at intersections to control the traffic flow and also improve its safety and efficiency. Installation of traffic control devices such as traffic is one such strategy to improve the safety at intersections. Traffic signals control the traffic from different approaches and prohibit the movement of conflicting traffic simultaneously. Signal cycle length for each individual approach includes red time during which the traffic flow is at halt, amber and green time during which traffic from a particular approach is allowed to cross the intersection. The amber time is given to warn the drivers that red signal is awaiting but vehicles of that stream can cross the intersection until it becomes red. Poorly designed traffic signals can increase delay for the vehicles traversing the intersection resulting in an increase in the overall travel time and lower efficiency.

The primary reason of installing traffic control devices is an increase in vehicular traffic. Ever increasing population along with rapid urbanization has led to a rapid increase in number of vehicles in the developing countries. In India, the registered vehicular population has grown from 0.3 million in 1951 to 230 million in 2016 at a growth rate of 10.7% annually (Road transport year book 2015–2016). While during the same time, development of traffic management systems and the road infrastructure have not been able to cope up at the same speed, thereby leading to congestion. Also, the trend analysis carried out in India revels that road user prefers personalized mode of transport over public mode.

Delay is the time lost by the road user and reflects the inconvenience caused while crossing a signalized intersection and can be converted in monetary terms. It depends on various parameters like traffic composition, type of vehicle, geometry of the intersection, driving behaviour and availability of space. Delay caused to a vehicle at a signalized intersection is an important parameter to assess the level of service of an intersection and is commonly used as a measure for intersection performance. Delay estimation is a complex procedure due to randomness in the arrival and departure of vehicles, heterogeneous and over saturated flow conditions in developing countries.

1.1 Delay Under Heterogeneous Traffic Conditions

Most of the developing countries experience traffic which is heterogeneous in nature. Heterogeneous traffic comprises of a mix of vehicles having very different static and dynamic characteristics. Static properties of a vehicle include length, width and weight, whereas dynamic properties are acceleration, deceleration and speed. These huge variations in the static and dynamic properties cause number of problems. Moreover due to such highly varying characteristics the vehicles do not follow strict lane disciple and tend to occupy the entire width of road based on the availability of space. Thus, the concept of standard lane width and lane disciple becomes invalid under such unrestricted flows.

Heterogeneous traffic may consists of non-motorized as well as motorized vehicles varying from slow moving cycle rickshaws to high speed cars. The presence of slow and fast moving vehicles on the same road results in unwanted interactions between these vehicles. Slow speed vehicles present in the traffic stream cause significant level of friction in the movement of other vehicles which reduce the average speed of traffic stream, thereby resulting in delays and a significant increase in the operational cost of vehicles. Under such conditions, different vehicle types may experience different levels of service on the same road. Such a complex scenario of traffic in developing countries poses a serious problem.

1.2 Delay Diagram

Analysis of all analytical delay models start with a graph of cumulative number of vehicles departing and arriving versus time at a signalized intersection. Solid line in the graph shows cumulative departure, whereas dash line represents cumulative arrivals. Time values are denoted on x-axis which is divided into intervals of effective green and effective red. The horizontal line in the graph shows the time spent by a particular vehicle in queue, whereas the vertical line shows the number of vehicles waiting in queue. The area under arrival and departure curve gives the value of total delay. On the basis of delay diagrams, delay can again be classified into uniform delay, random delay and overflow delay.

Uniform delay is based on an assumption that vehicle arrival at the intersection is at a uniform rate and the flow is stable with no individual cycle failure and is depicted in Fig. 1. No cycle failure can be described as no vehicle waiting for more than one green phase to cross the intersection. The departure rate becomes equal to the arrival rate towards the end of every green phase. In such a case, summation of the triangular area under arrival departure curve gives the total uniform delay. The uncertainty in the arrival rates of the vehicles at signalized intersection as shown in Fig. 2 is the main cause of random delay. In this system, cycle failure occurs during some of the signal phases leading to the formation of residual queues, but by the end of complete time period, the departure rate becomes equal to the arrival rate and thereby leaving no residual queue. Therefore, the overall period of analysis is stable. Random delay has an additional component along with uniform delay which can be found out by area under arrival rate and capacity function.

Additional delay occurring when the arrival rate is more than the capacity function is called as the overflow delay. When the traffic flow exceeds capacity ($v/c > 1$) for

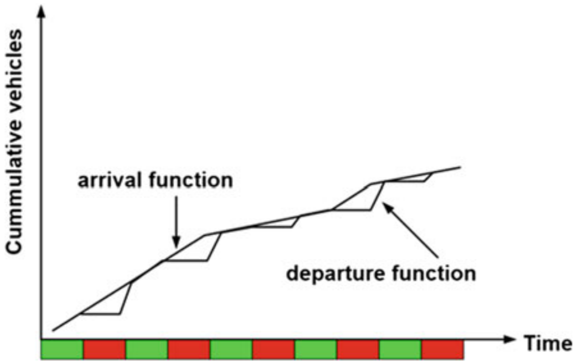

Fig. 1 Uniform delay

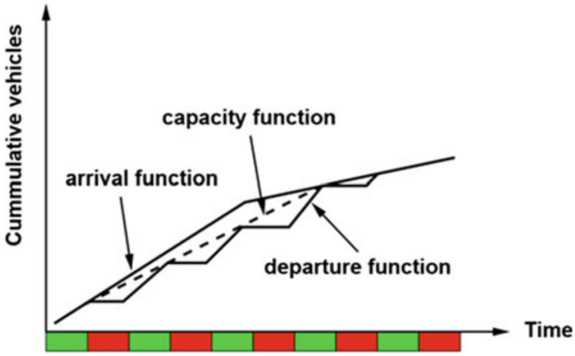

Fig. 2 Random delay

a long time period, the delay experienced by vehicle keeps on increasing during the over saturated period as shown in Fig. 3 where cycle failure occurs in each phase and the residual queue keeps on increasing.

1.3 Need of the Study

At present, India is the fastest growing economy in the world with a projected growth rate of 7.3% for the year 2018–2019 forecasted by the World Bank. Also, according to the present trends and future projections, India will have highest population surpassing China by the year 2025. A report by the Global Commission on the Economy and Climate has estimated that India's urban population is expected to reach up to 60 crore by 2031 which would be equivalent to 40% of the population at that time. Such a rapid increase in the urban population causes huge problems of congestion in the cities which are unable to keep up with the demand. According

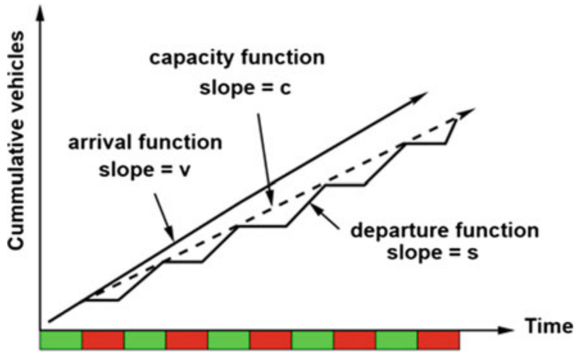

Fig. 3 Overflow delay

to a study conducted by Global Consultancy Firm, traffic congestion in four major Indian cities, namely Mumbai, Delhi, Bengaluru and Kolkata, causes a loss of Rs. 1.47 lakh crore annually to the economy.

While India accounts for one sixth of the world's population, its natural resources are limited as India accounts for only 5.7% of the world's coal reserves, 0.8% of geological reserve and only 0.4% of hydrocarbon reserves. India's energy use in the transportation sector is growing at the fastest rate in the world projected at 5.5%, whereas the average growth rate of transportation energy use is expected to be 1.4% per year. The transport sector accounts for almost 41% of country's total petroleum products consumption and 18% of the total energy consumption in India (Transport and Energy in India). Road transport consumes over three-fourths of the total energy employed by transport sector and thereby has significant implications on import of petroleum products. Therefore, Indian economy is heavily dependent on imported crude oil to meet the energy demands. This dependence on imported crude oil is increasingly alarming as our economy is heavily reliant on it. Uncertainty in the security and availability of crude oil supplies and rising prices and emissions are the major challenges. Development of more efficient transportation system policies and adoption of alternative vehicle technology can reduce the fuel demand and help in a more sustainable long-term development.

2 Literature Review

Many researchers have been studying the concept of delay at signalized intersection Webster was the first amongst them to study delay under the traffic conditions in United Kingdom (UK). Later Highway Capacity Manual (HCM) also proposed method to estimate delay based on the traffic conditions of United State of America (USA). But these models were developed for countries with homogeneous traffic conditions following lane disciple. However the traffic experienced in developing countries is highly heterogeneous in nature with poor lane disciple; therefore, the performance of these models may not correctly represent the field conditions in India.

Teply (1989) carried out studies to measure delay in field using two methods and their problems. The findings indicated exact match between analytical formula and field measured delay cannot not be expected. He also concluded that a fixed value for the ratio between overall delay and idling delay was inappropriate and depended on the duration of red time in the signal cycle length and therefore recommended a multiplicative adjustment factor that varied between the values 0.36 and 0.83.

Olszewski (1993) demonstrated that delay ratio is dependent on red time. In the Highway Capacity Manual 1985, the fraction of overall delay to idling delay was assumed constant with a value of 1.3; however, it was found that the delay ratio was near 1.3 only when the red time exceeded 60 s for random arrivals; therefore, he suggested subtractive factor varies with the vehicle approaching speed. The average value for acceleration–deceleration delay was found to lie in a range of 7.4–9.4 s for a deceleration length of 0.19 km considered in the study.

Quiroga and Bullock (1999) measured control delay at a signalized intersection in Florida Boulevard and Airline Highway in Baton Rouge, LA using GPS-based travel time data for a test car. They found that for the intersections considered for analysis the acceleration–deceleration delay value was approximately equal to 20 s for an average acceleration and deceleration rate of 3 km/h/s. On the basis of GPS data, they also reported on an average the vehicles start decelerating 0.29 km before the stop line and take approximately 0.13 km to attain a uniform speed. A linear relationship is observed between stopped delay and control delay.

Dion et al. (2004) compared simulated delays with delay obtained using existing deterministic and stochastic models for undersaturated to oversaturated conditions (v/c = 0.1–1.4). They came up with the conclusion that in case of low traffic demand all the models showed similar results but when the demand was approaching to saturation ($v/c > 0.7$) differences in the estimated and field delay also started increasing.

Parida and Gangopadhyay (2008) estimated the fuel loss during idling phase of signalized intersection in Delhi. They measured the idling fuel consumption rate by attaching flow metre for petrol diesel engines, whereas for vehicles using gaseous fuels their tanks were completely filled and the engines were run at idling condition. The intersections were divided into three categories, low volume (<75,000 veh/day), medium volume (75,000–100,000 veh/day) and high volume intersection (>100,000 veh/day) (Table 1).

Sekhar et al (2013) estimated the fuel loss due to idling of vehicles at signalized vehicles in Ahmedabad. They carried out traffic volume count for 16 h at four intersections in the study corridor. A test car fixed with GPS-based V-Box apparatus was run on the test stretch to study the delay and speed characteristics. A traffic simulation model was made for study corridor using VISSIM micro-simulation software, and the model was validated using data obtained from V-Box. The model was used to estimate the total and idling delay values at the signalized intersection and quantified in monetary terms. Mitigation measure involved the development of flyovers in the

Table 1 Annual fuel loss due to Idling in Delhi

Type of intersection	Number of intersections	Idling fuel consumption in million litres		
		CNG	Diesel	Petrol
Low volume	69	8.4	1.4	7.6
Medium volume	118	17.7	5	18.1
High volume	413	109.7	41	122.1
Total	600	135.9	47.4	147.8

Table 2 Idling fuel consumption per day in Ahmedabad

Name of intersection	CNG (L)	Petrol (L)	Diesel (L)
Himalaya intersection	107.586	127.4014	39.07578
Helmet intersection	361.2284	486.6723	81.94494
SAL Hospital intersection	178.1452	307.4386	358.1164

simulation model which led to an average reduction of around 74% in idling delay (Table 2).

3 Data Collection

3.1 Study Area Characteristics

The road which connects Noida to Delhi through DND flyway was considered as the test stretch on which two intersections that are 1.8 km apart were chosen for data collection. The selected intersections had different approach widths, and significant amount of traffic was experienced at the intersection. The intersection selection criteria was based on working traffic signal, presence of median and ease of collection of data. Both the intersections selected were four-legged signalized intersections. The stadium intersection has channelized left turn for all the four approaches of the intersection as shown in Fig. 4, whereas sector 19 intersection has left turn channelization just on two adjacent approaches shown in Fig. 5. The intersections had different signal cycle lengths and were coordinated. The intersection approaches have been named as shown in the figures.

It is also observed that 80% of the traffic falls along the Noida–Delhi corridor, thus exhibiting a strong relationship between Noida and Delhi. The traffic volume studies carried out by the city development authority also reveal that higher volumes of traffic enter the city in the morning hours, whereas in the evening hours the traffic leaving the city increases. The traffic surveys have also revealed that personalized motor vehicles have a significant share (40–50%) in the traffic volume. The increased

Fig. 4 Stadium intersection

Fig. 5 Sector 19 intersection

vehicle ownership has caused excessive demand for parking spaces, particularly in the central areas. Absence of organized car parking has resulted in parking on the streets and roads. This has not only reduced the capacity of the streets and roads but has also caused unnecessary delays (Noida Master Plan 2031).

3.2 Data Collection Velocity Box (V-BOX)

V-BOX is a data acquisition system helpful in measuring the performance of a vehicle. It can measure the speeds with an accuracy of ±0.1 kmph. Data of speed and stopped delay characteristics on the study corridor was collected by conducting several test runs with a probe vehicle fixed with a GPS-based V-Box apparatus provided by CSIR-CRRI. For setting up the device on the vehicle, cameras were attached at the front and year end of the vehicle to record the traffic and assess the real cause of delay. The GPS receiver was attached at the top of the vehicle for recording the position of the vehicle. It should be noted that the GPS receiver loses signal while moving under a tunnel or bridge and therefore cannot record data at these location. The vehicle used for data collection was Hyundai Xcent (Diesel Engine). All the devices were attached to V-BOX data logger which is USB powered and therefore care must be taken the vehicle does not turn off while recording the data. A complete set up of V-BOX is shown in Fig. 6. After the setup was completed, the probe vehicle was run by a skilled driver at the speed of the traffic without sudden acceleration and deceleration and without any erratic manoeuvre of vehicle (Fig. 7).

Fig. 6 Setting up of V-BOX

Fig. 7 Snapshot of video data from V-BOX

3.3 Data Collection Videography

Video graphic survey was carried out over the arms of intersection over two days. Placing the camera at a building nearby was not possible due to plantation along the divider and sidewalk due to which queue lengths were not visible. Therefore, three cameras were used to simultaneously capture the arrival and departure of vehicles for each arm of intersection as shown in Fig. 8.

Since the intersection was signalized, vehicles would sometimes stop in the field of view of the camera, and thus, left turning departure could sometimes not be recorded, and since signal cycle also played an important role in the analysis, it also had to be visible in the video. A single camera could not record all the three turning movements and signal timings simultaneously.

Fig. 8 Positioning of camera

Camera 1 was placed at the median near the stop line of the selected arm to record the through and right turning departure of vehicles. Camera 2 was placed on the sidewalk to specifically record the left turning departure of vehicles. Camera 3 was placed at some distance before the intersection to record the arrival rate of vehicles. Two approaches were selected for data collection on both the intersections. Data collection was carried out for two days during morning peak (9:00 AM–11:00 AM) and evening peak (5:00 PM–7:00 PM) hours.

4 Data Extraction

4.1 Data Extraction V-BOX

The file generated from V-BOX data logger is of.vbo extension which contains the speed and acceleration data. This file is accessible through a software Racelogic Performance Box which is used to analyse the data.

Data from each test run was analysed; graphs were made for each run. The data from V-BOX can be exported to excel through which the idle time for different runs was calculated. Idling time of the vehicle can easily be determined by the speed profile of the vehicle. The duration for which the vehicle is completely stopped (speed is zero) is the idle time.

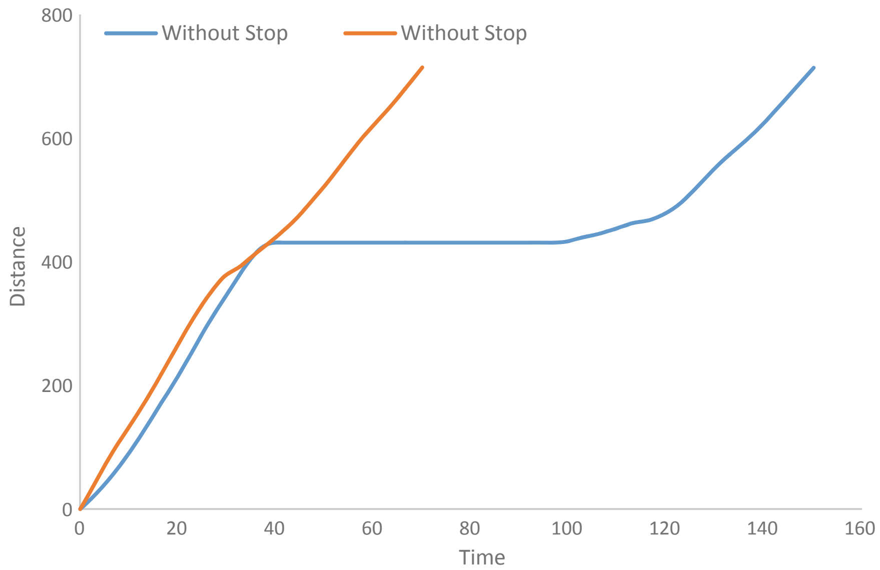

Fig. 9 Time space diagram for vehicle negotiating intersection

Table 3 Ratio of idling delay to stopped delay

Intersection	Approach	Ratio of idling delay to total delay
Stadium	A	0.86
	B	0.82
Sector 19	A	0.83
	B	0.84

Table 4 Idle time for probe vehicle obtained from V-Box during morning peak hour

			Idle time			
			Stadium intersection approach		Sector 19 intersection approach	
			A	B	A	B
Day 1	Morning	Run 1	57.8	22.1		
		Run 2	14.3	5.5		
		Run 3	68	12.5		
		Run 4	127.5	70.1		
		Run 5	13.7	44		
		Run 6	12.9	38.2		
Day 2	Morning	Run 1	118.6	21.1	166.4	60.1
		Run 2	0	29.1	84.3	139.1
		Run 3	28.6	98	64.1	119.1
Average morning idle time			49.04	37.84	104.93	106.1

Figure 9 shows a comparative time space diagram of a vehicle negotiating an intersection with and without stopping. Such time space diagram was drawn for all the speed runs. The ratio for idling delay to total delay was estimated from the graph which depends on the amount of red time at the signal (Table 3).

Tables 4 and 5 show the idle times obtained in various runs conducted on the intersection approach under consideration. Since the signal times at the intersection varied during morning and evening, the idle timings have been calculated as average of morning and evening idle time separately.

4.2 *Data Extraction Videography*

Video data of the intersection was used to collect the traffic volume count and turning movement of the vehicles. Moreover, the video data was also used to plot cumulative arrival and departure of vehicle versus time to determine the delay and queue lengths (Table 6).

Table 5 Idle time for probe vehicle obtained from V-Box during evening peak hour

			Idle time			
			Stadium intersection approach		Sector 19 intersection approach	
			A	B	A	B
Day 1	Evening	Run 1	116.9	6.1	49.4	278.7
		Run 2	88.9	49.4	77.7	446.4
Day 2	Evening	Run 1			59.8	
		Run 2			105	
		Run 3			201	
Average evening idle time			102.9	27.75	98.58	362.55

Table 6 Hourly traffic volume

Vehicle	Stadium intersection		Sector 19 intersection	
	A	B	A	B
2 Wheeler	1480	1312	834	1096
Auto	118	95	53	104
Cars	939	1088	1094	1691
Bus	67	73	63	80
LCV	21	91	24	22
Truck	9	26	16	14
Tractor	5	5	3	3
Cycle	205	126	34	36
Cycle rickshaw	41	49	42	33
E-rickshaw	314	207	218	212
Total	3199	3072	2381	3291

Delay diagrams are drawn for each approach as shown in Fig. 10 to determine the delay as discussed in Sect. 1.2. The graphs were drawn by counting the number of vehicles entering and exiting the intersection at every five seconds interval to achieve better accuracy in delay calculation. The departure rate during red phase in not zero as intersection had free left turn, and therefore, graph shows some vehicles departing even during red phase. The area between the two curves gives the total delay at the intersection (Table 7).

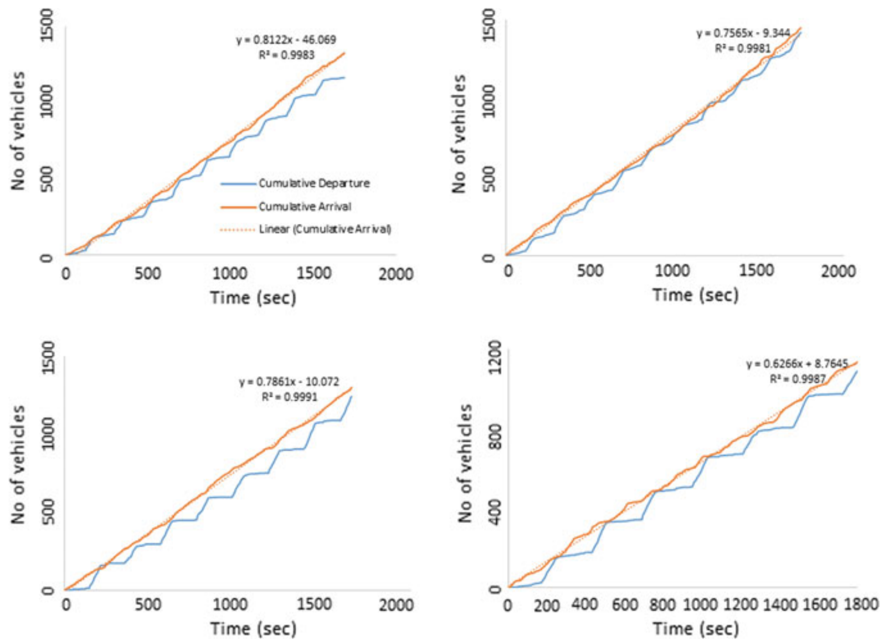

Fig. 10 Delay diagram for all the approaches

Table 7 Delay values for the intersection approach

Data collection	Intersection	Approach	Cycle length (s)	Red time (s)	Vehicles	Total delay (veh s)	Average delay (s)
Day 1 morning	Stadium	A	170	125	1164	93,817.5	80.60
Day 1 evening	Stadium	B	175	115	1351	60,317.5	44.65
Day 2 morning	Sector 19	B	210	145	1303	143,307.5	109.98
Day 2 evening	Sector 19	A	205	145	1149	100,117.5	87.13

5 Idling Fuel Consumption

Idling fuel consumption can be estimated as the product of idling delay and idling fuel consumption rate and total number of vehicles of each category. The area under arrival departure curves gives the value of total delay (veh s). The ratio of idle delay to total delay given in Table 3 is used to determine the value of average idling delay from average total delay per vehicle (Table 8).

Table 8 Average idling delay

Intersection	Approach	Idling delay/total delay	Average total delay (s)	Average idling delay (s)
Stadium	A	0.86	80.6	69.32
	B	0.82	44.65	36.61
Sector 19	A	0.83	87.13	72.32
	B	0.84	109.98	92.38

Table 9 Idling fuel consumption rate (mL/h)

Vehicle type	Fuel consumption	Remark
Two wheeler	255	Petrol
Auto rickshaw	677	CNG
Car	868	Petrol
Car	800	Diesel
Car	989	CNG
LCV	690	Diesel
HCV	920	Diesel
Bus	930	Diesel

Table 10 Fuel consumption at approach during one hour period

Intersection	Approach	Petrol (L)	Diesel (L)	CNG (kg)
Stadium	A	16.677	7.271	1.538
	B	9.166	4.868	0.654
Sector 19	A	15.711	8.549	0.721
	B	29.780	16.176	1.807

The idling fuel consumption rate has been taken from a previous study conducted by Parida and Gangopadhyay (2008) (Table 9).

Since the video graphic survey conducted could not determine the fuel used by vehicle, it is assumed that the ratio of car consuming petrol and diesel is 60:40 (Table 10).

The cost of petrol and diesel in Noida as of 9 May 2018 is Rs. 75.87 and Rs. 66.08, respectively. The fuel consumption data is converted into monetary terms (Table 11).

6 Remedial Measures

Extra amount of fuel consumed due to drivers keeping their engines in idle condition can be reduced if the drivers keep their engines off as recommended by PCRA. PCRA

Table 11 Fuel loss in monetary terms

Intersection	Approach	Daily fuel loss (Rs.)	Annual fuel loss (million Rs.)
Stadium	A	29,089.8	10.62
	B	16,766.34	6.12
Sector 19	A	28,653.11	10.46
	B	51,613.45	18.84

Table 12 Amount of fuel saved

Intersection	Approach	Annual fuel loss without switching off engine (million Rs.)	Annual fuel loss with engine switched off (million Rs.)	Cost of fuel saved annually (million Rs.)
Stadium	A	10.62	2.58	8.04
	B	6.12	2.34	3.78
Sector 19	A	10.46	2.02	8.44
	B	18.84	3.02	15.82

recommends drivers to switch off engines if the red time is more than 14 s. If all drivers follow this, there can be a huge amount of fuel saved (Table 12).

7 Conclusion

The congestion on roads is increasing due to rapid urbanization, but the traffic management systems and the road infrastructure have not been able to cope up at the same speed, thereby leading to congestion and high consumption of crude oil. Two signalized intersection were considered for data collection in Noida. Videography and test car method were used as data collection methods for the survey.

On the basis of data obtained, it was found that fuel worth Rs. 46.04 million is being wasted on the selected four approaches of the intersections under consideration. After the implementation of remedial measure, the fuel loss can be reduced up to 78.4% just by switching off engines at intersection during idling phase which would lead to fuel saving worth Rs. 26.12 million annually.

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Commuters' Exposure to Fine Particulate Matter in Delhi City

Rajeev Kumar Mishra, Ankita R. Mishra, and Abhinav Pandey

Abstract This study investigated exposure of $PM_{2.5}$ to the commuters travelling in various modes of transportation. Three types of transport mode were selected to conduct this study, namely bus, car and auto-rickshaw. Further, a total of five categories made out of three transport modes were finalized for the study, i.e. car with and without air-conditioning; bus with and without air-conditioning; and auto-rickshaw. The monitoring was done during morning, evening and off-peak hours of the day. The measured concentration inside non-AC bus as well as in non-AC car was found much higher than the permissible limits. On the other hand, among AC vehicles, slight variation was observed in the concentration of particulate matter during different monitoring hours. AC car was found to be the most efficient among the rest with $PM_{2.5}$ detected as minimum as 43.8 $\mu g/m^3$ in evening peak hours followed by AC bus with minimum average $PM_{2.5}$ detected as 52.2 $\mu g/m^3$ during off-peak hours. Remarkably, among all the selected commuting modes, the highest concentration was found in auto-rickshaw (>100 $\mu g/m^3$). The results report that the fine particulate level to a very large extent is affected by the mode of transport as well as the air-conditioning and ventilation system. It was observed that close modes with air-conditioning has reduced the exposure level than open modes.

Keywords Commuter · Exposure · NAAQS · $PM_{2.5}$ · Vehicles

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1 Introduction

Air pollution has remained consistently high and rising in Delhi and has also shown a mixed trend over the years. During the initial years of the last decade, the Indian Supreme Court directives have helped arrest soaring air pollution trend in Delhi in terms of improvement of emissions standards, implementation of largest ever public transport strategy on CNG; capping on age of commercial vehicles; improvement in vehicle inspection survey and diversion of substantial truck traffic otherwise plying on the city roads. Following other remedial measures, Delhi government also relocated polluting industries, cut power plant pollution and banned open burning. According to EPA, particulate matter in ambient air can also be called as particle pollution which is mixture of solid particle and liquid droplets found in the ambient air by natural or anthropogenic sources. It can see by naked eye as well as electron microscope as per their size classification (coarse to nanoparticle size). Over the past few decades, it has been verified that particulate matter affects the health and shows different mechanisms to affect different organs of body which are associated with its exposure duration, particle size, particle chemical composition. Transportation is major source of particulate matter emission in urban cites. Gupta and Kumar (2006) have mentioned that car, buses, trucks as mobile and heating furnaces, power plants and factories as stationary sources are responsible for particulate matter concentration in urban cites. The high ambient air pollution is associated with various anthropogenic sources like industry, garbage burning and traffic (Lestiani and Santoso 2011; Santoso et al. 2008, 2011). PM can be emitted as primary particle as well as secondary particles which may be generated from natural or man-made sources. In ambient air, PM may present in various size distribution, which is extended to nanoparticle to course particles. Particles may be classified into the following categories—nanoparticles <30 nm, ultrafine particles <100 nm, fine particle (0.1–2.5 μm), coarse particles larger than 2.5 μm (Hester et al. 2016). In urban area public as commuter spend comparatively good time during transportation than rural areas. According to Adams et al. (2001b), there is lot of work to explore the $PM_{2.5}$ health effect in urban microenvironment of transportation. It has been proofed that different transportation mode has different type of PM exposure to commuter. The objective of this study is to investigate the commuters' exposure to fine particulate matter, while travelling in different modes of transportation.

1.1 Health Impacts of Particulate Matter

It has become evident from several researches worldwide that severity of ambient air pollution is directly linked with adverse and extreme medical conditions of human population having identified as fifth largest contributor towards human mortality in India (Lim et al. 2012). The study found that mortality cases in India went up by about 12% with loss of life reported to be about 3% between the investigation years 2005

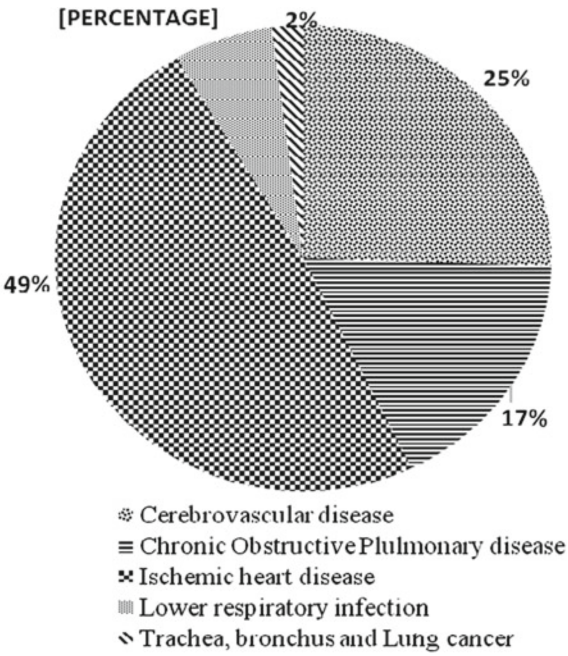

Fig. 1 Percent disease-wise deaths from ambient PM pollution in India. *Source* Lim et al. (2012)

and 2010. The same period of 5 years also reported 72,000 excess deaths in India indicating towards an alarming scenario of air pollution. Figure 1 presents percentage distribution of deaths with respect to a disease induced by ambient particulate matter pollution in India. The worldwide rise in adverse human health conditions such as brain abnormalities, asthma, decreased lung function, cardiopulmonary disease, cognitive deficits, allergic disorders, cardiovascular disease and death in utmost cases is attributable to exposure of fine particulate matter (Brook et al. 2010; Brunekreef and Holgate 2002; Dockery and Stone 2007; Padhi and Padhy 2008).

As shown in Fig. 1, the highest value, i.e. 49% of total deaths, is attributable to ischaemic heart disease, while the lowest, i.e. 2%, deaths to trachea, bronchus and lung cancer. There have been various studies worldwide including those in India evidencing that outdoor air pollution has become a severe environmental risk factor directly contributing to causes or aggravate diseases having acute and chronic nature. It is also reported through studies in India that child morbidity is significantly aggravated by an increase in ambient air pollution (Gurjar et al. 2008). The six metropolitan urban conglomerations investigated in this study were Chennai, Delhi, Hyderabad, Indore, Kolkata and Nagpur. The study observed that an ascend in ambient air pollution leads to significant increase in the likeliness of a child suffering from cough and fever. It is to be noted therefore that adopting measure of control on urban air pollution could significantly reduce child morbidity and should be given greater emphasis while infrastructure development and framing policies of sustainable urban living. Recent epidemiological studies have identified the adverse health effects of particulate matter, especially, $PM_{2.5}$ (Gascon et al. 2016; Han and Naeher 2006). It has also been reported that short- and long-term exposure to $PM_{2.5}$ is attributable to different types of adverse health effects, such as respiratory and cardiovascular diseases (Pope 2002; Dominici et al. 2006). Furthermore, exposure to such particulate matter, in specific relation to toxicological studies, has been found to induce oxidative DNA damage (Risom et al. 2005). Similar studies have also indicated that ultrafine particles (UFPs) may be more damaging to human health compared to large particles in view of the fact that former has relatively larger surface area available for adsorption or condensation than large particles with the same mass and can get deposited in alveoli, where these interact with epithelial cells (Delfino et al. 2005). Thus, exposure to ambient PM is considered to be one of the most serious environmental risks contributing to cardio-pulmonary and lung cancer mortality at global scale (Adams et al. 2001a).

2 Study Methodology

The study domain is designated for various transport modes of commuting vehicles along a particular transport corridor. The method was adopted after considering a combination of studies conducted to evaluate the individual's exposure while travelling in different cities of the world. The different transport modes chosen for the study were AC bus, non-AC bus, AC car, non-AC car and auto-rickshaw. According to local authorities, these means of transportation represent major distribution in Delhi.

2.1 Selection of Route

The study was conducted along a particular transport corridor between Moolchand and Dr. Ambedkar Nagar in Delhi region, covering a distance of 5.8 km (Fig. 2). The monitoring was conducted inside such transport modes/vehicles which are mostly used in day–to-day life by every commuter. The measurement of $PM_{2.5}$ was made during morning peak, evening and off-peak hours of the working days only.

2.2 Sampling and Measurement Equipment

Environmental Particulate Air Monitor (EPAM-5000) was used to monitor the $PM_{2.5}$ inside different modes/vehicles of transport along selected road stretch (Fig. 3). The figure shows the overview of EPAM-5000 instrument. It is such kind of handy equipment that can directly measure the real-time concentration of particulate matter.

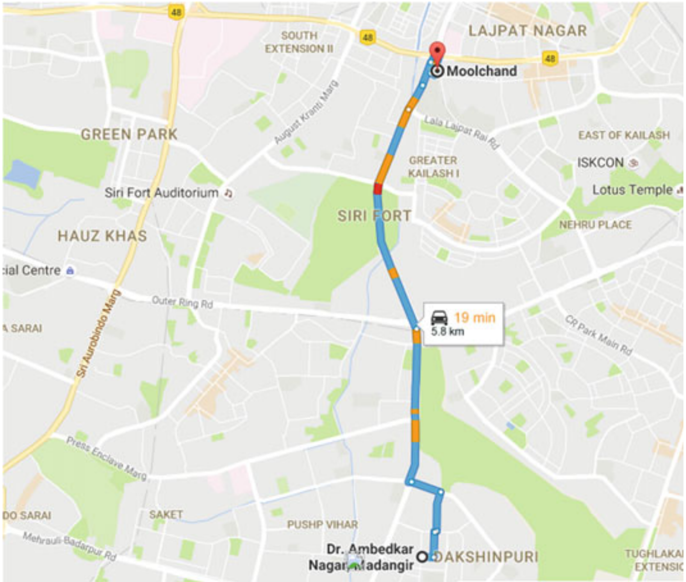

Fig. 2 Route map of study conducted in Delhi

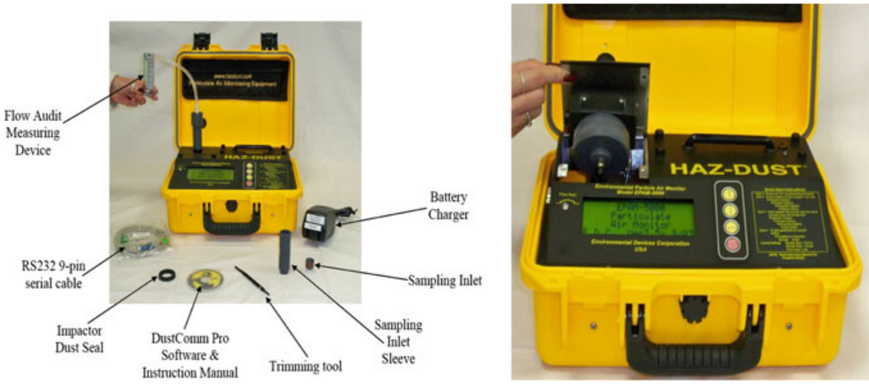

Fig. 3 Haz-Dust™ Model EPAM-5000 systems

Its handy nature is designed for measuring trace levels for ambient air pollutants. The peculiar sampling design allows for real-time data and filter gravimetric analysis utilizing the FRM 47 mm Cassette located directly behind the optical sensor. $PM_{2.5}$ was sampled using this instrument at breathing level of commuters travelling on the selected modes of transport along identified route. The sampling of particulate matter was done four times in each mode of transport during morning, evening and off-peak hours of the different days.

3 Result and Discussions

The average concentration of fine particulate matter in AC and non-AC buses during different hours of the days is presented in Fig. 4. During morning as well as evening peak hours of the days, the maximum concentration was found (93 μg/m^3) in non-AC buses during 10:00 AM to 11:00 AM and 7:00 PM to 8:00 PM, whereas in AC buses, the maximum concentration of $PM_{2.5}$ was found as 56 μg/m^3 during 7:00 PM to 8:00 PM, quite lower than non-AC buses. The figure shows the lowest concentration of 50.4 μg/m^3 in AC buses during evening peak hour, whereas in non-AC buses, the lowest concentration was found 62% higher than the highest concentration of AC-buses. The concentration measured in both types of buses during off-peak hours of the day was found slightly lower than the peak hours measured concentration. The concentration analysis in microenvironment of AC and non-AC buses proves AC buses as a better alternative for the commuters who commute from one place to another place.

The concentration variation of $PM_{2.5}$ in AC and non-AC car is depicted in Fig. 5. Without considering the model of the car, only concentration has been measured inside AC and non-AC car along the particular road stretch. Three modes of time interval, i.e. morning, evening and off-peak hours, of the days were chosen to conduct the study. The sampling was conducted at the back seat of the car at the height of breathing level. After analysing the data of morning and evening peak hours along with off-peak hours, quite high difference of particulate matter concentration was observed inside AC and non-AC car. From the figure, it is clear that the AC car is far better than non-AC car in terms of exposure to particulate matter. The maximum average concentration inside AC car was found as 47.5 μg/m^3, whereas in non-AC car, the highest concentration was 88.8 μg/m^3, which seems to be quite high than AC car. In addition to this, lowest concentration of $PM_{2.5}$ inside non-AC car was nearly 50% higher than the concentration of particulate matter inside AC car. The average

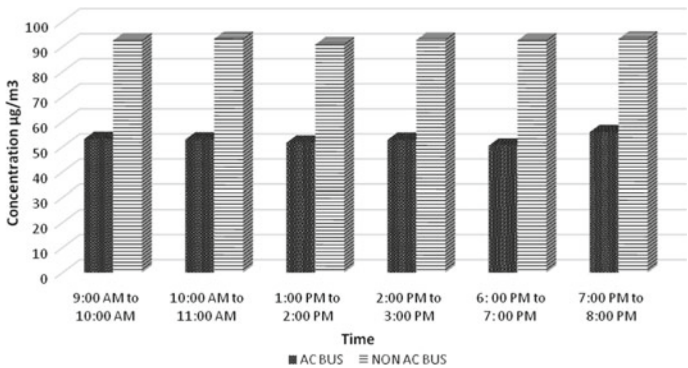

Fig. 4 Concentration of $PM_{2.5}$ in AC-bus and non-AC bus during morning, evening and off-peak hours

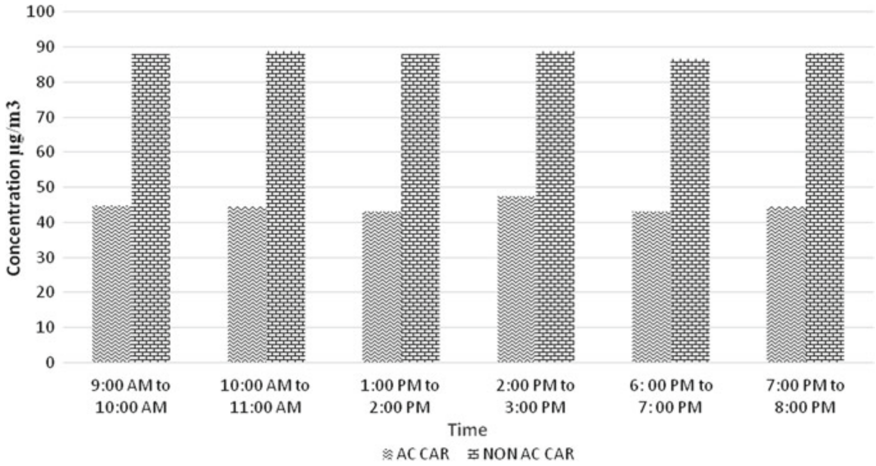

Fig. 5 Concentration of $PM_{2.5}$ in AC-car and non-AC car in during morning, evening and off-peak hours

concentration range of particulate matter was recorded from 43.0 to 47.5 $\mu g/m^3$ and 88.2 to 88.8 $\mu g/m^3$ in AC and non-AC car, respectively.

The monitoring of $PM_{2.5}$ inside running auto-rickshaw along particular transport corridor during working days is presented in Fig. 6. Since auto-rickshaw is a very common mode of travel in Delhi, due to that this mode was also considered to assess the exposure of commuters towards particulate matter. All three major divisions of

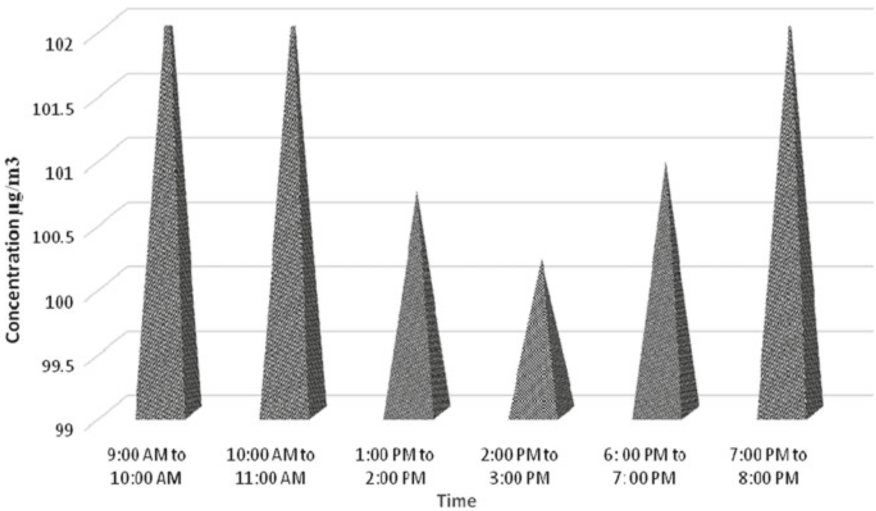

Fig. 6 Concentration of PM $_{2.5}$ in auto-rickshaw during morning, evening and off-peak hours (Yadav et al. 2019)

time interval (morning, evening and off-peak hours) were chosen to conduct the study. After computing the sampled data of auto-rickshaw, quite high range of $PM_{2.5}$ concentrations was observed. No major variation was observed among the average concentration of particulate matter during morning, evening as well as off-peak hours of the day. In comparison to AC bus, non-AC bus, AC car and non-AC car, 95.6%, 10%, 124% and 14.8%, respectively, higher concentration of fine particulate matter was found inside auto-rickshaw. During morning and evening peak hours of the day, quite high concentration of $PM_{2.5}$ was observed inside auto-rickshaw, which may pose hazardous effects on human health of the commuters. During the entire study in auto-rickshaw, it was observed that the reported concentration of $PM_{2.5}$ always exceeded the National Ambient Air Quality Standards (NAAQS).

4 Conclusion

This study investigated commuters' exposure to $PM_{2.5}$, while commuting in different modes of transport like AC bus, non-AC bus, AC car, non-AC car and auto-rickshaw. The measured concentration inside non-AC bus as well non-AC car was found much higher than the permissible limits. Among non-AC vehicles, non-AC bus reported average $PM_{2.5}$ as 92.8 $\mu g/m^3$ in morning as well as evening peak hours and 91.8 $\mu g/m^3$ in off-peak hours, while for non-AC car $PM_{2.5}$ was found to be 88.5 $\mu g/m^3$ in morning peak hours, 88.5 $\mu g/m^3$ in off-peak hours and 87.5 $\mu g/m^3$ during evening peak hours and for auto as 102.3 $\mu g/m^3$ in morning peak hours, 101.5 $\mu g/m^3$ in evening peak hours and 100.5 $\mu g/m^3$ during off-peak hours.

On the other hand, among the AC vehicles, slight variation was observed in the concentration of particulate matter between AC car and AC bus. AC car was found to be the most efficient among the rest with $PM_{2.5}$ detected as 44.6 $\mu g/m^3$ in morning peak hours, 43.8 $\mu g/m^3$ in evening peak hours and 45.3 $\mu g/m^3$ during off-peak hours followed by AC bus with average $PM_{2.5}$ detected as 53 $\mu g/m^3$ in morning, 53.2 $\mu g/m^3$ in evening and 52.2 $\mu g/m^3$ during off-peak hours. The reason behind this slightly higher concentration inside AC bus may be due to frequent opening and closing of doors, large size of engine, large surface area of bus, ridership of more people, frequent boarding and de-boarding of commuters, etc.

Off-peak hour concentration of particulate matter was mostly reported less than those in morning and evening peak hours. Notably, among all commuting modes, auto was the worst with all $PM_{2.5}$ concentrations above 100 $\mu g/m^3$. The study revealed that mode type, location as well as temporal condition affect the exposure of commuters to $PM_{2.5.}$ It is observed that close modes with air-conditioning and segregation have reduced exposure level than open modes. Partially closed modes in mixed traffic lanes such as auto have higher $PM_{2.5}$ concentrations than others.

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System Dynamics Simulation Modeling of Transportation Engineering, Energy, and Economy Interaction for Sustainability

Sandeep Singh and G. Uma Devi

Abstract India's crude oil import is growing phonemically and is likely to grow more, thereby posing a major threat to the economy of the country. A robust policy direction is a need of the hour. Since there is a lack of study in analyzing the interrelationship among the sectors of transportation, energy, and economy, this research work is aimed at performing an in-depth study of these sectors by considering the parameters such as vehicular population, model split, fuel consumption, and fuel costs. System dynamic (SD) simulation models are built using the STELLA software, and these parameters are forecasted for the future years until 2026. In this research work, when the existing trend of growth rate for these sectors was assumed to continue over a horizon year (2026), it was found that a large part of the economy is being spent for meeting the energy demands. So SD simulation models are built in which a scenario of increasing the growth rate of public transportation and at the same time curbing the growth rate of personalized transportation with a modal split value between the both as 70:30 showed a considerable decrease of 29.49% in fuel consumption and 58.04% in fuel cost. This modal split under the desirable scenario has resulted in a savings of 14,629 crore rupees for the horizon year in the road transport sector for Chennai city, India. Finally, this study recommends that in order to achieve sustainability in the city, the model split between public and personalized modes of transportation has to be paid crucial attention.

Keywords Transportation engineering · Energy · Economy · Sustainability

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1 Introduction

The critical relationship between transportation, energy, and economy comes into picture as one takes into account the exact importance of transportation for economic growth in which the energy sector is also involved and plays an equal role similar to the transportation sector and economy sector. The economy sector plays a vital role in performing the balance between the transportation sector and the energy sector. Hence, it could be said that the energy sector drives the transportation sector, whereas the transportation sector drives the economic sector. Altogether, it can be reasoned that the transportation sector and energy sector are like the balancing wheels of each other for driving the economy sector of any country.

Any unbalance in the transportation and energy sector could cause an unforeseen effect in the economy sector. This may result in significant losses to the country in the form of intangible monetary values. The developing countries like India have faced severe issues in the past due to the energy crises across the globe. Overnight fluctuations in the oil prices have never gone without surprising the Indian road users who are majorly dependent on personalized transport like two-wheelers and cars. So keeping these things in mind, the policies framework has to be done, which are accountable and assists in maintaining a balance among these three sectors. However, there are limited studies that have been conducted in the past to examine the interrelationship between the transportation, energy, and economy sectors. Hence, this research work aims to study various energy and economic factors that influence the road transportation sector. This is achieved by developing the system dynamics (SD) simulation models and by testing three different scenarios to potentially reduce the energy and economic losses in the road transportation sector. Finally, this study provided recommendations that are critical and appropriate in reducing the energy and economic losses that ensure prosperity in building up a sustainable transportation system.

2 Review of Literature

Shepherd and Ortolano (1996) evaluated the environmental impacts of policies using the strategic environmental assessment (SEA) for sustainable urban development. Based on four different case studies, this study examined six potentials of SEA to promote sustainability and concluded that SEA is a useful tool to implement sustainability principles in comprehensive planning. Martino et al. (2006) have studied the process of strategic modeling of transportation, economy, and energy scenarios. They have presented the application of two different strategic system dynamics models, namely the ASTRA model (SD model focused on describing the linkages between transportation, economy, and environment) and the MARS model. Both models make use of specific modules to determine travel behavior by running their sub-models iteratively for 25 years. The salient features of both the above models were incorporated, and the final model depicted the future energy supply.

Parikh et al. (2009) studied the future of energy options for India in an interdependent world. They have analyzed the implications of the available domestic energy potential for the energy supply and demand situation in the transportation sector. A quantitative model was built, taking into account the possible forms of energy consumption for transportation of the people and freight. Simulation has been carried out from the base year 2000 up to the year 2050. Demand projection has been carried out based on key factors like economic growth and demographics and their corresponding dynamics. Zheng et al. (2011) studied transportation sustainability that could be transformed into a useful metric for assessing the performance of the transportation system focusing on characterizing and measuring the economic aspect of sustainability concerning transportation in terms of sustainability. The final results of this study described the relationship between urbanity, mode share, and the economic aspects of transportation sustainability.

Maheshwari et al. (2014) attempted to build dynamic models to capture the interdependent behavior of transportation, economic, and, additionally, environmental systems. The results indicated periodic behavior with a phase lag for the performance of transportation and the activity system, which utilized the nonlinear modeling techniques to capture the nominal behavior of all the three systems.

Finally, it can be inferred from the pieces of literature that an inter-relationship-based study involving the sectors of transportation, energy, and economy is limited to some extent and is not critically analyzed. Therefore, this instigates the need for this research work.

3 Study Framework

After various literature reviews, a study framework is designed to analyze the interrelationship between transportation, energy, and economy sectors. The data collection is the initial stage for the study, after which the SD model conceptualization has been carried out in establishing the relationship between the variables of urban transportation, fuel consumption, and fuel cost. The SD simulation models operate based on feedback information; it is used as a principle methodology to forecast future urban vehicular demand in the transportation sector, fuel consumption by vehicles in the energy sector, and fuel cost in the economy sector. This was followed by the development and analysis of SD models for different scenarios (do-minimum scenario, partial efforts scenario, and desirable scenario). The model evaluation and model calibration were carried out by adjusting and fine-tuning the various variables under consideration to reflect the real system. The comparison of results under the three different scenarios was conducted to determine the advantages of the adopted policy measures. Further, based on the obtained results, critical and appropriate recommendations for adopting the best policy measures are suggested. The detailed study framework is shown in Fig. 1.

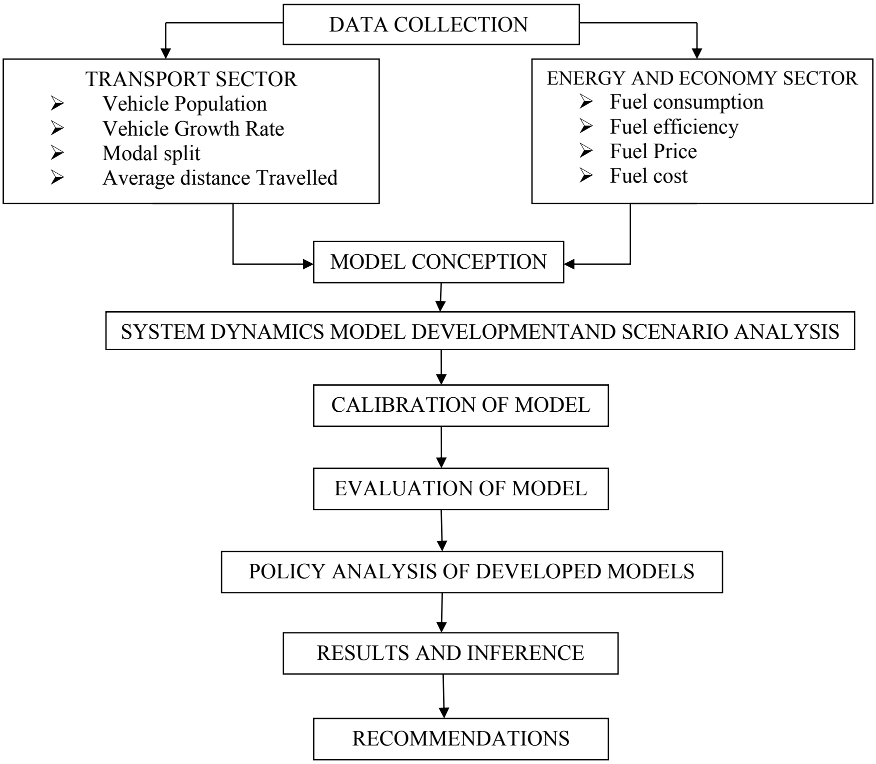

Fig. 1 Study framework

4 Study Area

The study area selected for this study is Chennai city, which is one of the four metropolitan areas in India with a population of around 9 million, with an area of 174 km^2. The Chennai Corporation governs the Chennai city. The Chennai city has a heterogeneous type of traffic condition plying on its roads majorly occupied by personalized vehicles like two-wheelers (TWs) and cars. Other types of vehicles like Metropolitan Transport Corporation (MTC) buses, auto-rickshaws, taxis, private buses (Pvt buses), mini-buses, light commercial vehicles (LCV), and heavy commercial vehicles (HCV) are also plying on the city roads.

Due to the increase in the growth rate of vehicular population, especially personalized vehicles in the Chennai city, there is an increase in the fuel requirement and fuel consumption. This proportional and exponential increment in the vehicles and fuel consumption has eventually resulted in increased fuel costs due to the fuel importing. Hence, it is imperative to study this effect on the economic growth rate through a system dynamics approach aimed at analyzing the impact of the increase

Fig. 2 Map of study area Chennai city. *Source* Maps of India.com

in the vehicular population on energy usage in the transportation sector, which in turn influences the economy of the city. Figure 2 illustrates the map of the study area.

5 Data Collection

5.1 Transportation Sector

Under the transportation sector, the data collected is the classification of vehicles and its composition along with their respective growth rate values, which are further used for scenario analysis under three different scenarios, which are do-minimum scenario, partial efforts scenario, and desirable scenario. The SD models were built

Table 1 Vehicle population in Chennai city

Vehicle class	Year					
	2010	2011	2012	2013	2014	2015
MTC buses	3421	3464	3527	3654	3798	4257
Auto-rickshaw	49,062	63,340	66,679	68,599	70,933	73,854
Taxis	1259	1268	1354	1475	1517	1628
Pvt buses	2702	2906	2962	2991	3015	3043
Mini-buses	2095	2217	2355	2406	2491	2575
LCV	11,836	12,736	19,123	21,469	33,136	41,248
HCV	12,846	17,928	22,165	33,571	40,833	49,124
Motor cycles	13.71	15.63	17.06	19.42	21.18	22.67
Scooters	3.33	4.03	4.69	4.90	5.10	5.75
Mopeds	4.97	6.15	6.27	6.39	6.45	6.66
TWs (in lakhs)	22.01	25.81	28.91	30.53	32.79	35.09
Cars (in lakhs)	4.82	5.80	5.98	6.15	6.59	6.87
Total (in lakhs)	27.61	32.64	37.60	38.81	40.11	43.72

Source Statistics of transportation Department, Chennai (www.tn.gov.in)

using these data, and the forecasting of the same values was carried out until the horizon year 2026. The different classes of vehicles and their vehicle population for the Chennai city considered for this study are shown in Table 1.

Table 1 shows that the TWs, which includes motor cycles, scooters, and mopeds, with a vehicular population of 35.09 lakhs and cars with 6.87 lakhs share the highest proportion among the vehicle class in the year 2015. However, the public transportation system consisting of the MTC buses shares one of the lowest vehicle populations with just 4257 vehicles in the year 2015. Also, it was noticed that the TWs and the cars continue to grow exponentially each year in comparison to MTC buses.

5.2 Energy Sector

Under the energy sector, the average distance traveled by the various classes of vehicles in km/day, the efficiency of fuel in km/day, and the consumption of fuel in L/veh/year by the different class of vehicle are taken into account and used in the model simulation process which is shown in Table 2.

Table 2 Efficiency of fuel and consumption of fuel by vehicle classes

Vehicle class	Average distance travelled (km per day)	Efficiency of fuel (km per L)	Consumption of fuel (liters per vehicle per year)
Bus	151	4.1	13,415
Auto-rickshaws	96	21	1669
Taxis	21	13	534
Pvt buses	111	5.0	11,863
Mini-buses	22	8.7	897
LCV	51	14	1330
HCV	55	4.33	4637
Cars	24	12.9 (petrol)	684
		15.6 (diesel)	652
TWs	18	53	124

Source Report of the Expert Group, Government of India Report

5.3 Economy Sector

In India, one among the rapidly growing city is the city of Chennai, Tamil Nadu, which have a different form of economic pattern. In 2015, Chennai city had a GDP of $66 Billion, i.e., 403,260 crore rupees ranking fifth in the cities of India based on GDP. As per the estimates of the Confederation of Indian Industries (CII), it is estimated that the city's economy will grow up to $100 billion by the year 2025. Table 3 indicated the annual GDP and the GDP growth rate of Chennai city.

Table 3 Annual GDP and GDP growth rate of Chennai city

Year	GDP (crore rupees)	Growth rate (%)
2012–2013	348,338	6.94
2013–2014	374,498	7.51
2014–2015	403,260	7.68

Source Central Statistical Organization and State Directorate of Statistics

6 Scenario Analysis and Model Results

6.1 Scenario I-Do-Minimum Scenario (When the Existing Trend Is Allowed to Continue)

In this scenario, the existing growth rates of the different classes of vehicles like MTC buses, auto-rickshaws, taxis, LCVs, HCVs, Pvt buses, mini-buses, cars, and TWs are considered to continue up to the horizon year 2026. Based on this scenario model, as depicted in Fig. 3, the values of fuel consumption and fuel cost by each class of vehicles have been simulated.

It could be observed from Table 4 that if the existing trend is allowed to continue, the number of cars and TWs will reach 18.47 lakh and 84.61 lakh, respectively, in 2026. Whereas the public transport vehicles constituting the MTC buses have increased only to a fleet size of around 6693 in 2026. The growth of the vehicle population has an incidental increase in the level of fuel consumption and the respective fuel cost by each class of vehicles. The consumption of fuel for different vehicle classes in liters per day and its respective fuel cost in rupees per day is shown in Tables 5 and 6, respectively.

Under the do-minimum scenario with the growth in the number of vehicles between the public and personalized transport, fuel consumption is also seen to increase. The fuel consumed by cars is 29.51 lakh liters per day, and TWs are 20.86

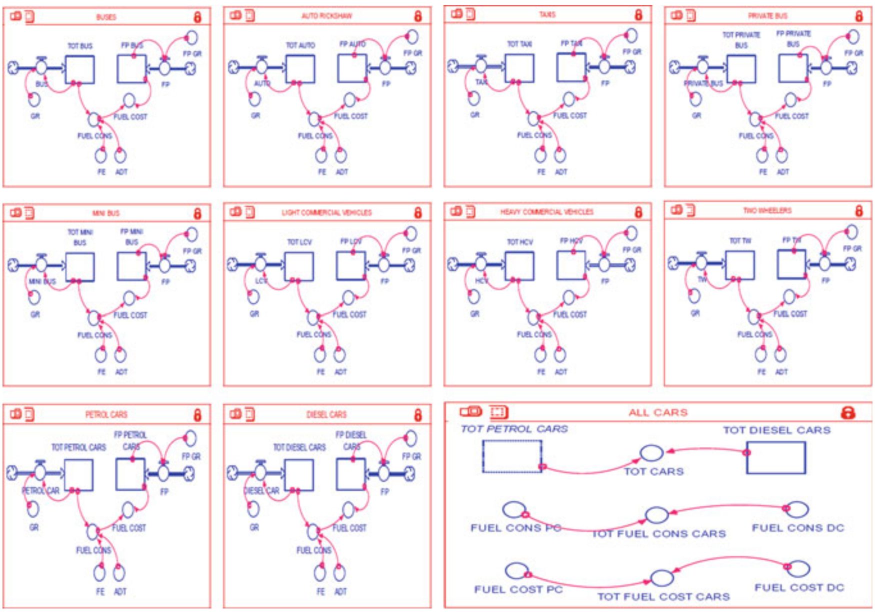

Fig. 3 System dynamics simulation models for do-minimum scenario

Table 4 Results of scenario I–vehicular population

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt buses	Mini-buses	Cars	TWs
2015	4257	73,854	1628	41,248	49,124	3043	2575	687,598	3,509,159
2016	4436	78,211	1939	49,126	50,057	3442	2858	752,232	3,801,472
2017	4622	82,826	2309	58,510	51,008	3892	3173	822,942	4,118,135
2018	4816	87,713	2750	69,685	51,978	4402	3522	900,299	4,461,175
2019	5019	92,888	3276	82,995	52,965	4979	3909	984,927	4,832,791
2020	5229	98,368	3901	98,847	53,972	5631	4339	1,077,510	5,235,363
2021	5449	104,172	4646	117,726	54,997	6369	4816	1,178,796	5,671,468
2022	5678	110,318	5534	140,212	56,042	7203	5346	1,289,602	6,143,902
2023	5916	116,827	6591	166,993	57,107	8147	5934	1,410,825	6,655,689
2024	6165	123,719	7850	198,888	58,192	9214	6587	1,543,443	7,210,107
2025	6424	131,019	9349	236,876	59,297	10,421	7312	1,688,526	7,810,709
2026	6693	138,749	11,135	282,119	60,424	11,787	8116	1,847,248	8,461,341

Table 5 Results of scenario I–fuel consumption by each class of vehicle

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt bus	Mini-buses	Cars	TWs
	Liters per day								
2015	148,454	337,618	5931	150,261	623,977	106,118	6511	1,098,520	865,272
2016	154,689	357,538	7063	178,960	635,832	120,020	7228	1,201,781	937,349
2017	161,186	378,632	8412	213,142	647,913	135,743	8023	1,314,748	1,015,430
2018	167,956	400,972	10,019	253,852	660,224	153,525	8905	1,438,334	1,100,016
2019	175,010	424,629	11,933	302,338	672,768	173,637	9885	1,573,538	1,191,647
2020	182,361	449,682	14,212	360,084	685,550	196,383	10,972	1,721,450	1,290,911
2021	190,020	476,214	16,926	428,860	698,576	222,109	12,179	1,883,266	1,398,444
2022	198,001	504,310	20,159	510,772	711,849	251,206	13,519	2,060,293	1,514,935
2023	206,317	534,064	24,010	608,330	725,374	284,113	15,006	2,253,961	1,641,129
2024	214,982	565,574	28,596	724,521	739,156	321,332	16,657	2,465,833	1,777,835
2025	224,011	598,943	34,058	862,904	753,200	363,427	18,489	2,697,622	1,925,928
2026	233,420	634,281	40,563	1,027,719	767,511	411,036	20,523	2,951,198	2,086,358

Table 6 Results of scenario I–fuel cost for each class of vehicles

Year	MTC buses	Auto-ricshaws	Taxis	LCV	HCV	Pvt bus	Mini-buses	Cars	TWs
	Rupees per day								
2015	5,690,252	18,342,464	322,202	5,759,488	23,917,035	4,067,521	249,586	50,173,571	47,009,367
2016	6,308,714	20,366,766	400,052	7,298,561	25,931,232	4,894,790	294,771	57,758,487	53,395,122
2017	6,308,714	20,366,766	400,052	7,298,561	25,931,232	4,894,790	294,771	57,758,487	53,395,122
2018	7,754,603	25,110,240	616,725	11,720,443	30,482,794	7,088,307	411,162	76,565,353	68,886,788
2019	8,597,435	27,881,443	765,737	14,852,427	33,049,933	8,529,955	485,599	88,167,416	78,244,373
2020	9,531,873	30,958,481	950,752	18,821,351	35,833,266	10,264,812	573,512	101,538,105	88,873,093
2021	10,567,873	34,375,106	1,180,471	23,850,868	38,851,000	12,352,510	677,341	116,948,638	100,945,618
2022	11,716,474	38,168,794	1,465,693	30,224,393	42,122,876	14,864,813	799,966	134,712,060	114,658,076
2023	12,989,915	42,381,159	1,819,830	38,301,076	45,670,296	17,888,078	944,792	155,189,744	130,233,234
2024	14,401,762	47,058,407	2,259,533	48,536,042	49,516,466	21,526,227	1,115,837	178,798,879	147,924,123
2025	15,967,061	52,251,844	2,805,476	61,506,038	53,686,545	25,904,317	1,317,849	206,021,152	168,018,144
2026	17,702,489	58,018,436	3,483,328	77,941,927	58,207,811	31,172,841	1,556,432	237,412,767	190,841,737

lakh liters per day, and that by MTC buses is just 2.33 lakh liters per day in 2026. Finally, the total fuel that would be consumed by all the vehicles in the year 2026 would reach 298.81 crore liters per annum.

Under the do-minimum scenario with the growth in the number of vehicles and fuel consumption between the public and personalized transportation, the fuel cost for each vehicle class is also seen to increases. The fuel cost incurred by cars is 23.74 crore rupees per day, and TWs are 19.08 crore rupees per day, and that of MTC buses is just 1.77 crore rupees per day in 2026. The total fuel cost that would be incurred by all the vehicles in the year 2026 would be 24,679 crore rupees per annum.

6.2 Scenario II-Partial Efforts

In the scenario of partial efforts, the growth rate values of MTC buses, auto-rickshaws, taxis, LCVs, HCVs, Pvt buses, mini-buses, cars, and TWs have been altered as per the government policies which targets for a 50:50 modal split between public and personalized transport mode for the horizon year 2026. The simulation process was carried out for each class of vehicles based on their incremental growth rate values from which the total quantity of fuel consumption and fuel cost was estimated. To achieve the target of 50:50 modal split between public and personalized transport modes, the growth rate of the former one has been increased to reach 11.10%, and on the other hand at the same time, the growth rate of later one is restrained from being nearly half of the already existing value of 4.16% and 4.73%, cars and TWs, respectively. Figure 4 depicts the developed system dynamics simulation models for the partial scenario.

The simulated value of the vehicle population for each class of vehicle-based on the values mentioned above is shown in Table 7.

It could be observed from Table 7 that if minimal efforts are undertaken based on the government's policy and by altering growth rate of an existing trend, the number of cars and TWs will reach 10.45 lakh and 45.40 lakh, respectively, from 18.47 lakh cars and 84.61 lakh TWs (scenario I values) in the year 2026, whereas the public transport vehicles constituting the MTC buses have increased only to a fleet size of around 12,767 from 6693 (scenario I value) in the horizon year 2026. This happens so because of the consideration of the hypothesis that one single MTC bus can replace 20 cars and 40 TWs. This decrease in the vehicular population with respect to the number of cars and TWs happens due to a modal split value of 50:50 between public and personalized transport mode in the transportation system which has lead to decrease in consumption of fuel and fuel cost which is shown in Tables 8 and 9, respectively.

In accordance with the alteration of growth rate in the number of vehicles between the public and personalized transportation, the consumption of fuel by each class of vehicles in the transportation system gets varied correspondingly. From Table 8, it can be observed that the fuel consumed by cars decreases to 16.70 lakh liters per day from the scenario I's 29.51 lakh liters per day and the fuel consumed by TWs

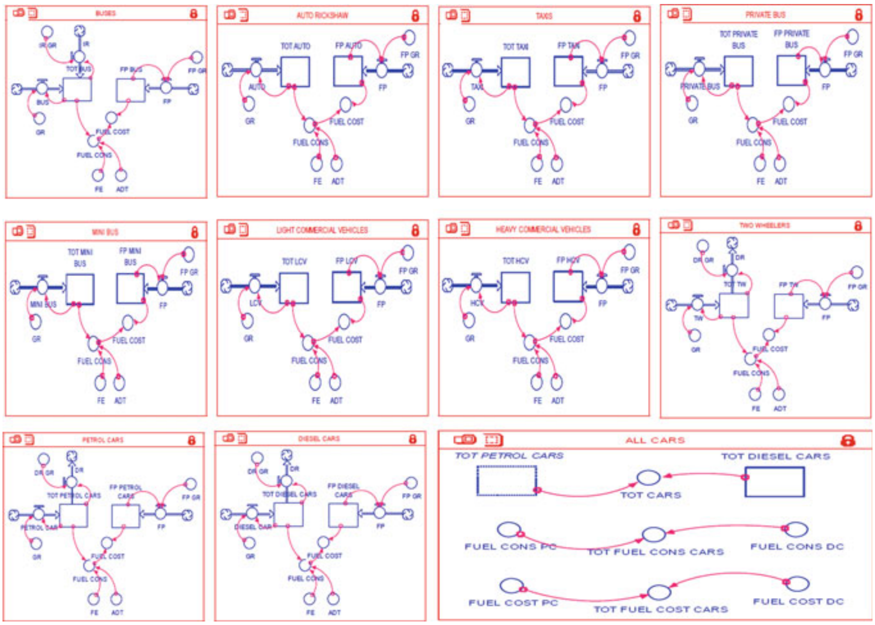

Fig. 4 System dynamics simulation models for partial scenario

decreases to 11.19 lakh liters per day from the scenario I's 20.86 lakh liters per day and the fuel consumed by MTC buses increases to 4.45 lakh liters per day from the scenario I's 2.33 lakh liters per day in 2026.

The total fuel consumption by personalized transportation is 27.89 lakh liters per day, whereas that of public transportation is just 4.45 lakh liters per day. So the total saving in fuel consumption is 23.44 lakh liters per day.

From the above values, it can be said that the fuel consumption of MTC buses has seen an increase in it, but when we take into account the values of fuel consumption per person traveling in MTC Bus with respect to that of persons using cars and TWs combined, it can be said that the fuel consumption of the former transport mode is seen to be much lesser than that from the latter one. When these resultant values are compared with the scenario I results, it can be concluded that the total fuel consumed by all the vehicles in scenario II decreases to 224.25 crore liters per annum from the scenario I's 298.81 crore liters per annum in 2026, which shows a reduction of 24.95% of fuel consumption by all the vehicles in the scenario I.

In accordance with the alteration in the growth rates of the number of vehicles and fuel consumption between the public and personalized transport modes, the fuel cost by each mode of transportation also gets varied correspondingly.

From Table 9, it can be observed that the fuel cost incurred by cars decreases to 13.42 crore rupees per day from the scenario I's 23.74 crore rupees per day and the fuel cost incurred by TWs decreases to 10.24 crore rupees per day from the scenario I's 19.08 crore rupees per day and the fuel cost incurred by MTC buses increases to

Table 7 Results of scenario II–vehicular population

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt buses	Mini-buses	Cars	TWs
2015	4257	73,854	1628	41,248	49,124	3043	2575	687,598	3,509,159
2016	4704	78,211	1939	49,126	50,057	3442	2858	714,302	3,592,326
2017	5198	82,826	2309	58,510	51,008	3892	3173	742,045	3,677,464
2018	5744	87,713	2750	69,685	51,978	4402	3522	770,868	3,764,620
2019	6347	92,888	3276	82,995	52,965	4979	3909	800,813	3,853,842
2020	7013	98,368	3901	98,847	53,972	5631	4339	831,922	3,945,178
2021	7750	104,172	4646	117,726	54,997	6369	4816	864,243	4,038,678
2022	8563	110,318	5534	140,212	56,042	7203	5346	897,821	4,134,395
2023	9462	116,827	6591	166,993	57,107	8147	5934	932,706	4,232,380
2024	10,456	123,719	7850	198,888	58,192	9214	6587	968,950	4,332,688
2025	11,554	131,019	9349	236,876	59,297	10,421	7312	1,006,604	4,435,372
2026	12,767	138,749	11,135	282,119	60,424	11,787	8116	1,045,724	4,540,491

Table 8 Results of scenario II–fuel consumption by each class of vehicle

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt bus	Mini-buses	Cars	TWs
	Liters per day								
2015	148,454	337,618	5931	150,261	623,977	106,118	6511	1,098,520	865,272
2016	164,042	357,538	7063	178,960	635,832	120,020	7228	1,141,150	885,779
2017	181,266	378,632	8412	213,142	647,913	135,743	8023	1,185,437	906,772
2018	200,299	400,972	10,019	253,852	660,224	153,525	8905	1,231,447	928,262
2019	221,331	424,629	11,933	302,338	672,768	173,637	9885	1,279,245	950,262
2020	244,571	449,682	14,212	360,084	685,550	196,383	10,972	1,328,902	972,784
2021	270,250	476,214	16,926	428,860	698,576	222,109	12,179	1,380,491	995,838
2022	298,627	504,310	20,159	510,772	711,849	251,206	13,519	1,434,085	1,019,440
2023	329,983	534,064	24,010	608,330	725,374	284,113	15,006	1,489,764	1,043,601
2024	364,631	565,574	28,596	724,521	739,156	321,332	16,657	1,547,609	1,068,334
2025	402,917	598,943	34,058	862,904	753,200	363,427	18,489	1,607,704	1,093,653
2026	445,223	634,281	40,563	1,027,719	767,511	411,036	20,523	1,670,137	1,119,573

Table 9 Results of Scenario II–fuel cost for each class of vehicle

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt bus	Mini-buses	Cars	TWs
	Rupees per day								
2015	5,690,252	18,342,464	322,202	5,759,488	23,917,035	4,067,521	249,586	50,173,571	47,009,367
2016	6,690,143	20,366,766	400,052	7,298,561	25,931,232	4,894,790	294,771	54,830,083	50,457,478
2017	7,865,735	22,614,472	496,711	9,248,912	28,115,056	5,890,312	348,136	59,927,067	54,158,506
2018	9,247,902	25,110,240	616,725	11,720,443	30,482,794	7,088,307	411,162	65,506,965	58,131,003
2019	10,872,944	27,881,443	765,737	14,852,427	33,049,933	8,529,955	485,599	71,616,379	62,394,880
2020	12,783,538	30,958,481	950,752	18,821,351	35,833,266	10,264,812	573,512	78,306,486	66,971,510
2021	15,029,861	34,375,106	1,180,471	23,850,868	38,851,000	12,352,510	677,341	85,633,495	71,883,834
2022	17,670,908	38,168,794	1,465,693	30,224,393	42,122,876	14,864,813	799,966	93,659,146	77,156,473
2023	20,776,040	42,381,159	1,819,830	38,301,076	45,670,296	17,888,078	944,792	102,451,267	82,815,858
2024	24,426,805	47,058,407	2,259,533	48,536,042	49,516,466	21,526,227	1,115,837	112,084,379	88,890,356
2025	28,719,084	52,251,844	2,805,476	61,506,038	53,686,545	25,904,317	1,317,849	122,640,372	95,410,415
2026	33,765,601	58,018,436	3,483,328	77,941,927	58,207,811	31,172,841	1,556,432	134,209,237	102,408,716

3.37 crore rupees per day from the scenario I's 2.33 crore rupees per day in 2026. The total fuel cost incurred by personalized transportation is 23.66 crore rupees per day, whereas that of public transportation is just 3.37 crore rupees per day. So the total saving in the fuel cost is 20.29 crore rupees per day.

From the above values, it can be said that the fuel consumption of MTC buses has seen an increase in it, but when we take into account the values of fuel consumption per person traveling in MTC Bus with respect to that of persons using cars and TWs combinedly, it can be said that the fuel consumption of the former transport mode is seen to be much lesser than that from the latter one. When these resultant values are compared with the scenario I results, it can be concluded that the total fuel cost incurred by all the vehicles in scenario II decreases to 18,275 crore rupees per annum in scenario II from the scenario I's 24,679 crore rupees per annum in the horizon year 2026, which shows a reduction of 28.94% of fuel cost incurred by all the vehicles in the scenario I.

6.3 Scenario III-Desirable Scenario

In the desirable scenario, the growth rate values of MTC buses, auto-rickshaws, taxis, LCVs, HCVs, Pvt buses, mini-buses, cars, and TWs were altered as per the government policies which targets for a 70:30 modal split between public and personalized transport mode for the horizon year 2026. The simulation process was carried out for each class of vehicles based on their incremental growth rate values from which the total quantity of fuel consumption and fuel cost was found. Figure 5 depicts the developed SD simulation model for the desirable scenario.

To achieve the target of 70:30 modal split between public and personalized transport modes, the growth rate of the public transportation was increased to reach 17.34%, and on the other hand at the same time, the growth rate of cars and TWs is restrained to 2.33% and 3.19%, respectively. The simulated value of the vehicle population based on the values mentioned above is shown in Table 10.

The simulated value of the vehicle population for each class of vehicle-based on the values mentioned above is shown in Table 10.

It could be observed from Table 10 that if minimal efforts are undertaken based on the government's policy and by altering growth rate of the existing trend, the number of cars and TWs will reach 8.20 lakh and 39.66 lakh, respectively, from 18.47 lakh cars and 84.61 lakh TWs (scenario I values) in 2026, whereas the public transport vehicles constituting the MTC buses have increased only to a fleet size of around 16,393 from 6693 (scenario I value) in the horizon year 2026. This happens so because of the consideration of the hypothesis that one single MTC Bus can replace 20 cars and 40 TWs.

This decrease in the vehicular population with respect to the number of cars and TWs happens due to a modal split value of 70:30 between public and personalized transport mode in the transportation system which has lead to decrease in consumption of fuel and fuel cost which is shown in Tables 11 and 12, respectively.

Table 10 Results of scenario III–vehicular population

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt buses	Mini-buses	Cars	TWs
2015	4257	73,854	1628	41,248	49,124	3043	2575	687,598	3,509,159
2016	4812	78,211	1939	49,126	50,057	3442	2858	698,665	3,548,462
2017	5440	82,826	2309	58,510	51,008	3892	3173	709,931	3,588,204
2018	6149	87,713	2750	69,685	51,978	4402	3522	721,400	3,628,392
2019	6951	92,888	3276	82,995	52,965	4979	3909	733,076	3,669,030
2020	7857	98,368	3901	98,847	53,972	5631	4339	744,963	3,710,123
2021	8882	104,172	4646	117,726	54,997	6369	4816	757,065	3,751,677
2022	10,040	110,318	5534	140,212	56,042	7203	5346	769,387	3,793,696
2023	11,349	116,827	6591	166,993	57,107	8147	5934	781,932	3,836,185
2024	12,829	123,719	7850	198,888	58,192	9214	6587	794,705	3,879,150
2025	14,502	131,019	9349	236,876	59,297	10,421	7312	807,712	3,922,597
2026	16,393	138,749	11,135	282,119	60,424	11,787	8116	820,955	3,966,530

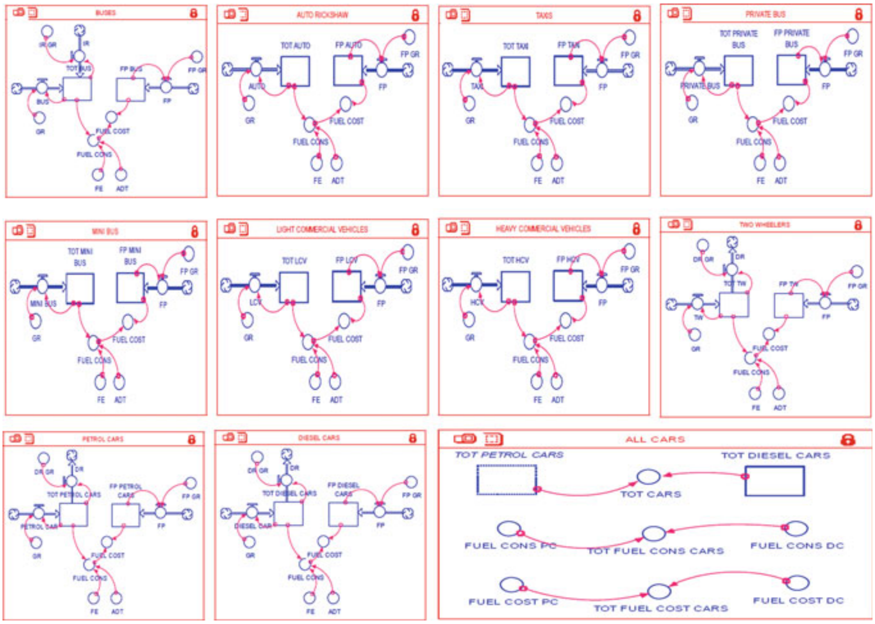

Fig. 5 System dynamics simulation models for desirable scenario

In accordance with the alteration of growth rate in the number of vehicles between the public and personalized transportation, the consumption of each class of vehicles of the transportation system gets varied correspondingly. It can be observed from Table 11, that the consumption of fuel by cars decreases to 13.13 lakh liters per day from the scenario I's 29.51 lakh liters per day and the fuel consumed by TWs decreases to 9.78 lakh liters per day from the scenario I's 20.86 lakh liters per day and the fuel consumed by MTC buses increases to 5.71 lakh liters per day from the scenario I's 2.33 lakh liters per day in 2026. The total fuel consumption by personalized transportation is 22.91 lakh liters per day, whereas that of public transportation is just 5.71 lakh liters per day.

So the total saving in fuel consumption is 17.20 lakh liters per day. From the above values, it can be said that the fuel consumption of MTC buses has seen an increase in it, but when we take into account the values of fuel consumption per person traveling in MTC bus with respect to that of persons using cars and two-wheelers combinedly, it can be said that the fuel consumption of the former transport mode was seen to be much lesser than that from the latter one.

When these resultant values were compared with the scenario I result, the total fuel consumed by all the vehicles in scenario III decreases to 210.67 crore liters per annum from the scenario I's 298.81 crore liters per annum in 2026, which shows a reduction of 29.49% of fuel consumption by all the vehicles in the scenario I.

Table 11 Results of scenario III–fuel consumption by vehicles

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt buses	Mini-buses	Cars	TWs
	Liters per day								
2015	148,454	337,618	5931	150,261	623,977	106,118	6511	1,098,520	865,272
2016	167,813	357,538	7063	178,960	635,832	120,020	7228	1,116,310	874,963
2017	189,695	378,632	8412	213,142	647,913	135,743	8023	1,134,423	884,763
2018	214,432	400,972	10,019	253,852	660,224	153,525	8905	1,152,863	894,672
2019	242,394	424,629	11,933	302,338	672,768	173,637	9885	1,171,638	904,692
2020	274,002	449,682	14,212	360,084	685,550	196,383	10,972	1,190,754	914,825
2021	309,732	476,214	16,926	428,860	698,576	222,109	12,179	1,210,218	925,071
2022	350,121	504,310	20,159	510,772	711,849	251,206	13,519	1,230,037	935,432
2023	395,776	534,064	24,010	608,330	725,374	284,113	15,006	1,250,217	945,909
2024	447,386	565,574	28,596	724,521	739,156	321,332	16,657	1,270,766	956,503
2025	505,725	598,943	34,058	862,904	753,200	363,427	18,489	1,291,692	967,216
2026	571,671	634,281	40,563	1,027,719	767,511	411,036	20,523	1,313,001	978,048

Table 12 Results of scenario III–fuel cost by vehicles

Year	MTC buses	Auto-rickshaws	Taxis	LCV	HCV	Pvt buses	Mini-buses	Cars	TWs
	Rupees per day								
2015	5,690,252	18,342,464	322,202	5,759,488	23,917,035	4,067,521	249,586	50,173,571	47,009,367
2016	6,843,926	20,366,766	400,052	7,298,561	25,931,232	4,894,790	294,771	53,698,729	49,841,362
2017	8,231,502	22,614,472	496,711	9,248,912	28,115,056	5,890,312	348,136	57,472,849	52,843,965
2018	9,900,402	25,110,240	616,725	11,720,443	30,482,794	7,088,307	411,162	61,513,602	56,027,454
2019	11,907,666	27,881,443	765,737	14,852,427	33,049,933	8,529,955	485,599	65,839,925	59,402,727
2020	14,321,892	30,958,481	950,752	18,821,351	35,833,266	10,264,812	573,512	70,472,102	62,981,338
2021	17,225,593	34,375,106	1,180,471	23,850,868	38,851,000	12,352,510	677,341	75,431,870	66,775,535
2022	20,718,006	38,168,794	1,465,693	30,224,393	42,122,876	14,864,813	799,966	80,742,514	70,798,307
2023	24,918,491	42,381,159	1,819,830	38,301,076	45,670,296	17,888,078	944,792	86,428,987	75,063,424
2024	29,970,605	47,058,407	2,259,533	48,536,042	49,516,466	21,526,227	1,115,837	92,518,020	79,585,484
2025	36,047,013	52,251,844	2,805,476	61,506,038	53,686,545	25,904,317	1,317,849	99,038,261	84,379,969
2026	43,355,387	58,018,436	3,483,328	77,941,927	58,207,811	31,172,841	1,556,432	106,020,404	89,463,288

In accordance with the alteration in the growth rates of the number of vehicles and fuel consumption between the public and personalized transportation, the fuel cost by each mode of transportation also gets varied correspondingly.

It can be observed from Table 12, that the fuel cost incurred by cars decreases to 10.60 crore rupees per day from the scenario I's 23.74 crore rupees per day and the fuel cost incurred by TWs decreases to 8.94 crore rupees per day from the scenario I's 19.08 crore rupees per day and the fuel cost incurred by MTC buses increases to 4.35 crore rupees per day from the scenario I's 2.33 crore rupees per day in the year 2026. The total fuel cost incurred by personalized transportation is 19.54 crore rupees per day, whereas that of public transportation is just 4.35 crore rupees per day. So the total saving in the fuel cost is 15.19 crore rupees per day.

From the above values, it can be said that the fuel consumption of MTC buses has seen an increase in it, but when we take into account the values of fuel consumption per person traveling in MTC Bus with respect to that of persons using cars and TWs combinedly, it can be said that the fuel consumption of the former transport mode was seen to be much lesser than that from the latter one. When these resultant values were compared with the scenario I results, the total fuel cost incurred by all the vehicles in scenario III decreases to 10,353 crore rupees per annum from the scenario I's 24,679 crore rupees per annum in 2026, which shows a reduction of 58.04% of fuel cost incurred by all the vehicles in the scenario I.

7 Recommendations

This study recommends stringent legislative policy as similar to the one implemented in New Delhi Capital city of India: odd–even concept of plying of vehicles on the road on alternate odd–even dates of the days in a week. Similar legislative policies are to be framed and implemented with immediate effect that curbs the usage of the personalized mode of transport and enhances the public mode of transport. In this study, the recommended policy is restricting the usage of the personalized mode of transport for certain specified short distance travels during peak hour or certain period or on a particular non-working day in a week which can be implemented which not only reduces the plying of the personalized mode of transport vehicles on the city roads but also enhances the usage of public mode of transport vehicles including rails, trams, etc. On the other hand, the fleet size of the urban public transportation like the MTC buses has to be increased not only with greater efficiency and reliability but also with promising mobility, accessibility, and connectivity, to achieve the policy as mentioned earlier of the government.

These policies can eventually lead to a considerable reduction in fuel consumption and fuel cost in the city, making it a place with reduced economic losses due to fuel imports. Also, fuel emissions from vehicles and the impact of pollutants on the environment can be reduced, thereby attaining sustainable transportation systems that have considerable contributions toward achieving the Sustainable Development Goals (SDG).

Finally, this study also recommends that besides, implementing the the recommended mitigation measures to the 3E systems—'Engineering', 'Energy', and 'Economy', it is always advisable and better to consider additional 3E's also, i.e., the Education, Ethical values, and Enforcement into the 3Es' systems for a prosperous and sustainable development of the city.

8 Conclusions

The significant contribution in this study is a novel approach to understand the dynamics of the three interdependent sectors, using the concepts derived from the system dynamics (SD) technique.

The findings of the study suggest that the performance of SD simulation models are useful in evaluating the inter-disciplinary and interdependent parameters of the different sector of transportation, energy, and economy. SD simulation models are useful to understand the three systems in unison and formulate appropriate policies that conserve resources without hindering growth, ultimately ensuring a healthy environment to provide a better and sustainable life for future generations.

Finally, it could be concluded from the analysis of the study that not only the quantitative indicator like the vehicular population, fuel consumption, fuel cost, and GDP of the city has to be considered for representing the sustainable growth of the city but, also, the qualitative indicators like the value system and quality of life of the people are to be considered for representing the real, sustainable growth and prosperity of the city for achieving Sustainable Development Goals (SDG).

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Quantifying Travel Time Reliability of Air-Conditioned Public Buses in Urban Area: A Case Study of Kolkata

Saptarshi Sen and Sudip Kumar Roy

Abstract Public transport plays a major role in transporting people from one place to another for work trips in India. It is a preferred mode for work trips, mainly due to the low travelling cost. Whenever the trip makers travel by public transport mode with higher travelling cost, they expect a more reliable service. If the expectation is not fulfilled, they may like to shift their mode choice from public transport to a service with better travel time reliability, such as para-transit and personal vehicle. This modal shift will affect the traffic flow characteristics of different roadways and also adversely affect the environment. So, this study focuses on estimating the travel time reliability of roadway public transport especially air-conditioned public buses of Kolkata and adopt measures to improve it. As case study, one of the AC buses of Kolkata (Route: AC9) originating from Jadavpur 8B Bus Stand and terminating at Karunamoyee Bus Stand has been selected. The actual travel time has been compared with the scheduled travel time for different times of the weekdays over a week. Moreover, to highlight the difference between peak and off-peak hour, the travel time reliability measures of the AC9 during peak and off-peak hours of the day have been estimated separately as reliability buffer index, on-time performance, travel time measure, and punctuality index. A comparison of different alternative modes on the basis of travelling cost is discussed. Finally, some factors which affect the travel time reliability of buses in Kolkata have been discussed and some measures to improve the present reliability condition of the buses are recommended.

Keywords Public transport · AC bus · Reliability · Travel time · Punctuality · On-time performance

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1 Introduction

In most of the countries in the world, public transport system plays a vital role in transporting people from one place to the other all throughout the day. In India, public transport of all forms (including metro railway) is preferred by the commuters, especially for work trips, due to its low travelling cost. In some of the Indian cities, including Kolkata, metro railway dominates the modal share of public transport due to its highly reliable service. But, since the introduction of the air-conditioned (AC) buses along various routes in Kolkata, there has been a tendency to shift the mode towards AC buses especially in those routes where metro railway is not yet functional. Even if the AC bus fare is higher than the conventional non-AC buses, people mainly prefer these buses due to the comfortable journey it provides. Along with the comfort level, a very important parameter that plays a vital role in selecting the mode of transport during their work trips is the service quality of the buses. Service quality mainly includes the travel time reliability, on-time performance and headway regularity. When these service qualities are not maintained by the AC buses, the commuters who can afford and are willing to pay more for reducing their travel time, tend to shift to a more reliable mode such as personal vehicle, cab and para-transit. This modal shift will increase the traffic volume, mainly during the peak hour, making it difficult to manage and control traffic. This leads to increased traffic congestion and ultimately increases the travel time at the route network level. Hence, it is necessary to study the service reliability characteristics of the AC buses so that proper steps can be taken to maintain the desired level of reliability. In this study, the main focus remains on the estimation of travel time reliability of AC bus. The objectives of the study can be classified as:

- To discuss the socio-economic characteristics of the passengers travelling in an AC bus and their perception towards the AC bus service
- To estimate the travel time reliability of AC bus during peak hours and off-peak hours using different proposed methods in previous literatures
- To discuss some ways to improve the reliability in order to attract more commuters towards public transport.

Many researches regarding travel time reliability have been done in the last couple of decades. Some of the researchers have identified the factors that affect the reliability of buses. Congestion created by the heterogeneity and huge traffic volume on the road (Georgiadis et al. 2014; Paulley et al. 2006; Yaakub and Napiah 2011) is a major factor that affects reliability. The time of the day also adversely affects the traffic flow which gives rise to congestion, thus increasing the travel time (Carrasco 2011; Inta and Muntean 2015; Lee et al. 2012). Route length is also considered as a factor for unreliability of buses (Chen et al. 2009; Hu and Shalaby 2017; Lee et al. 2012). This is because, higher the route length, more will be the resistance to the flow due to higher number of signalized intersection and congestion. The passenger volume is considered as one of the factors for delay in travel time since it increases the dwell time (Liu and Sinha 2006; Hu and Shalaby 2017). There are several methods applied by different researchers to quantify reliability. In this study, four methods have been adopted to assess the reliability of the AC bus service. These four methods have been extracted from previous works by researchers and from service quality manual of other countries and are described in the next section.

2 Methodology

The reliability of a bus can be assessed from three perspectives, i.e. travel time reliability, headway regularity and on-time performance. In this study, the reliability of the AC bus is assessed from the perspective of travel time measure and on-time performance, and the difference between the peak and off-peak hours is highlighted. In 2017, Hu and Shalaby ranked the reliability measures on the basis of ease of data availability and ease of calculation. From the rank list, four methods having rank below 10 were chosen for this study. The methods are discussed as follows:

2.1 Travel Time Measures

Travel time of a bus indicates the in-transit travel time from the origin terminal to the destination terminal. The travel time perspective of reliability measurement includes the measure of deviation of actual in-transit travel time from the scheduled in-transit travel time or from mean travel time. The methods proposed in previous literatures which are adopted in this study are:

- Coefficient of Variation of Travel Time ($C_{\mathrm{v,TT}}$) (Mazloumi et al. 2008): It is defined as the ratio of standard deviation (σ) of in-transit travel time to the mean (μ) in-transit travel time (refer Eq. (1)). Lower the variation from the mean travel time, more reliable will be the bus service; i.e. lower the value of $C_{\mathrm{v,TT}}$, more reliable is the service. The value lies from 0 (if there is no variation from mean travel time) to 1 (if there is a deviation equal to the mean travel time).

$$C_{\mathrm{v,TT}} = \frac{\sigma}{\mu} \tag{1}$$

  This method focuses on the deviation from the mean travel time and is the easiest way to determine the reliability of a bus service. Along with the calculation, the data collection for this method is also easy.
- Reliability Buffer Index (RBI) (FHWA): This method is a travel time measure of ascertaining bus service reliability. It is defined as the ratio of the difference between the 95th percentile travel time and the mean travel time to the mean travel time (refer Eq. (2)). The value is expressed in percentage.

$$\text{RBI}(\%) = \frac{\text{95th Percentile Travel Time} - \text{Mean Travel Time}}{\text{Mean Travel Time}} \times 100\% \quad (2)$$

Higher the difference between the 95th percentile travel time and the mean travel time, higher will be the RBI value and the bus service will be less reliable.

- Punctuality Index based on Routes (PIR) (Chen et al. 2009): This index denotes the probability of the adherence of the actual travel time (TT_{actual}) to the scheduled travel time ($TT_{scheduled}$). A threshold limit is maintained. If the deviation of the actual travel time from the scheduled travel time falls within the prescribed threshold (δ_1 and δ_2), then the service is said to be reliable (refer Eq. (3)).

$$\text{PIR} = P\{\partial_1 < |\text{TT}_{\text{actual}} - \text{TT}_{\text{scheduled}}| < \partial_2\} \quad (3)$$

In the previous literature, the acceptable threshold limit is considered as 10% (Chen et al. 2009) of the $TT_{scheduled}$. If the deviation from $TT_{scheduled}$ lies within the threshold limits, higher will be the reliability of the service. In this study, different threshold limits are set and the changes in PIR values have been determined.

These three methods are adopted in this study as a part of the travel time measure perspective for estimating the reliability of the AC bus service.

2.2 On-Time Performance (OTP)

On-time performance is another aspect of quantifying bus service reliability. The method adopted in this study is the one proposed by Washington Metropolitan Area Transit Authority (WMATA) to determine the service quality of transit systems in urban area. It indicates the deviation of the bus arrival time from the scheduled arrival time at the destination terminal. A threshold limit of deviation is set as 2 min early and 7 min late by WMATA. The on-time performance-based reliability of bus service is defined as Eq. (4).

$$\text{OTP} = \frac{\text{No. of Trips with arrival time deviation within WMATA threshold limit}}{\text{Total No. of Trips Observed}} \quad (4)$$

The OTP value lies within 0 and 1. OTP value close to 1 indicates reliable bus service. It can be expressed in percentage to quantify the reliability.

3 Study Area—Location and Route

Kolkata is one of the metropolitan cities of India and is the largest in Eastern India with a population of nearly 4.5 million (according to census report, 2011). Out of these people, almost 40% are workers and commute regularly to their work place from their residence. A huge population of Kolkata resides in the southern part such as Garia, Jadavpur, Santoshpur and travels to Salt Lake regularly for work. Out of the many bus routes from south Kolkata to the Salt Lake area via Eastern Metropolitan Bypass, AC9 (air-conditioned bus) and S9 (non air-conditioned bus) bus route has a huge demand all throughout the day, mainly for work trips. In this study, the focus is mainly on AC buses, so the analysis is based mainly on the data acquired from AC9 buses.

The AC9 route is a round trip route where the origin terminal is 8B bus stand at Jadavpur, and the destination terminal is the Karunamoyee bus stand at Salt Lake (refer to Fig. 1). The AC9 bus plies through two densely populated residential area of South Kolkata, viz. Jadavpur and Santoshpur. Some of the other important junctions in this route include Rashbehari Connector, Science City and Beliaghata on the Eastern Metropolitan Bypass (EM Bypass). These junctions are important route interchange location, and thus, AC9 also acts as a feeder to these junctions. The major part of the route passes through commercial areas but also has some residential complex. The destination, i.e. a part of Salt Lake and Karunamoyee, has a huge employment density with many government offices, such as BikashBhavan, UnnayanBhavan, SechBhavan to name a few, as well as many private company headquarters. Not only does this area has offices but also has some residential zones which also contribute to the demand of the AC9 bus.

4 Data Collection and Extraction

Primary survey was conducted to collect the data daily from 4 AC9 buses at 4 different time of the day, i.e. morning peak, morning off-peak, afternoon off-peak and evening peak, for seven days by randomly selecting the seven days over a month. No bus of the same scheduled departure time was surveyed during these 7 days. The time range for each category is given in Table 1.

On-board technique was adopted to acquire the data. In this method, enumerators travelled in the bus along with the other passengers to carry out the survey. The following data were obtained:

- Travel Time data: Travel time within two consecutive stops was noted down from the start of the journey at the origin terminal till the end of the journey at the destination terminal by enumerators using a digital watch. The dwell time at each stop and the delay due to signalized intersection and congestion were also observed using a stop watch. Running time, delays due to congestion, traffic signal and dwell time were collected along with the total journey time from this technique.
- Passenger volume data: An enumerator counted the number of passengers boarding or alighting the bus at each stop throughout the trip manually. This gave us an idea of the total bus users during the different time of the day.

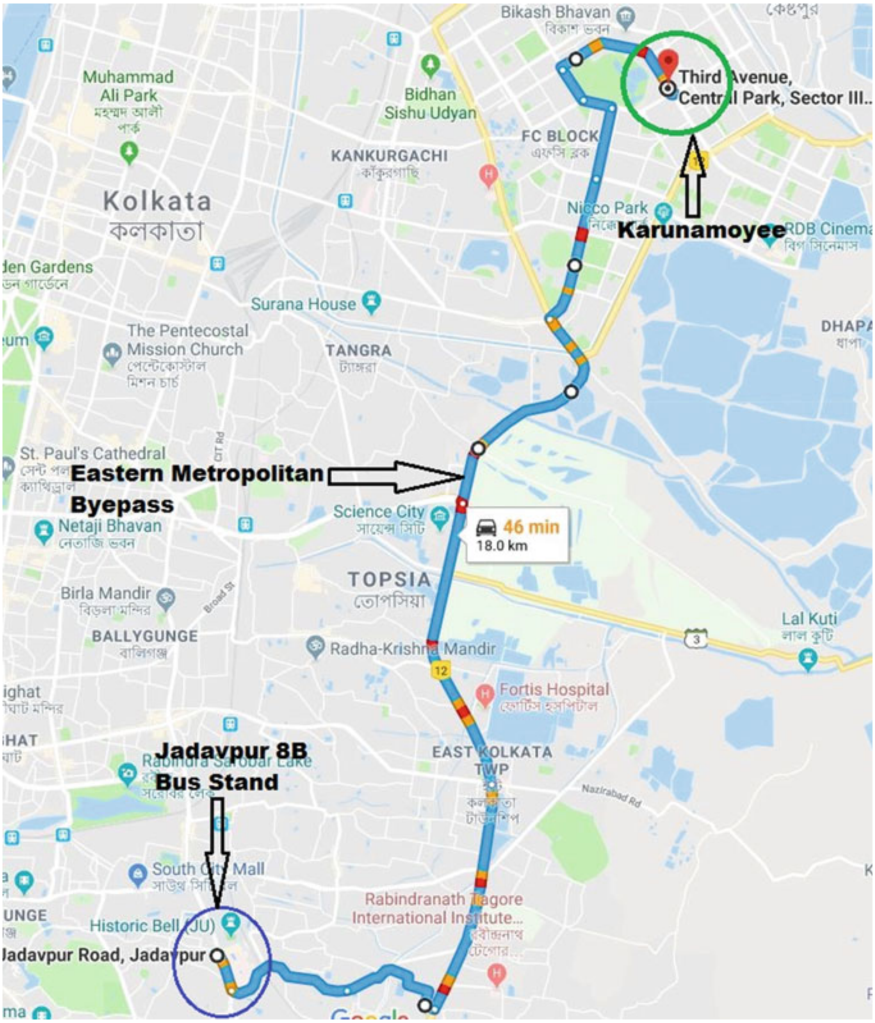

Fig. 1 AC9 route. *Source* Google Map

Table 1 Characteristics of the time of the day and the number of buses surveyed

Characteristics	Morning peak	Morning off-peak	Afternoon off-peak	Evening peak
Time of the day	8 am to 11 am	11 am to 12 noon	12 noon to 5 pm	5 pm to 8 pm
Total no. of buses surveyed	7	7	7	7

- Socio-economic characteristics of passengers: Questionnaire survey was carried out to acquire the socio-economic data of the commuters. A set of questions was prepared, and the answers were collected from the passengers. It included questions on personal details and household details, and the remaining majority part of the questionnaire included questions related to the perception on the service quality of AC9 bus.

The data related to travel time and passenger count obtained from the primary survey were uploaded to MS Excel software, manually. The questionnaire survey data were rigorously scrutinized. After rejecting the incomplete and ambiguous data entry by the respondents, 350 responses were selected to carry out the analysis.

5 Results

The results of the analysis are separately presented under separate heads including socio-economic characteristics of commuters and reliability estimation. A glimpse of the socio-economic characters of the commuters is provided which have a significant effect on their mode choice during work trips. Then the reliability of the AC9 service is quantified using different proposed methods separately for peak and off-peak hours.

5.1 Socio-economic Characteristics of Commuters

The number of passengers travelling in AC9 during peak hours is usually two times more than the off-peak hour as shown in Figs. 2 and 3, respectively, for peak and off-peak hour. The total number of passengers travelling daily in AC9 buses is nearly 2200 (*Source* West Bengal Transport Corporation).

The major purpose of the commuters to travel by AC9 bus is mostly official work and educational. Most of the commuters travel by AC9 bus to reach their workplace or educational institutes during the morning peak hour and return to their home from workplace during the evening peak hour. As shown in Figs. 4 and 5, on an average, 70% of people travel by AC9 for work purpose during both peak and off-peak hours followed by educational purpose which is about 20%.

From the purpose of travelling, it is clear that the age distribution of the commuters lie within the range of 18–60 years of age as shown in Fig. 6. In this figure, it shows that the age of more than 75% commuters lie within the age group of 18–45 during both peak and off-peak hours. So it is evident that the young professionals travel to Salt Lake in AC9 bus more often all throughout the day.

The gender distribution of the commuters is shown in Figs. 7 and 8, respectively, for peak and off-peak hours. The distribution indicates that nearly 40% of the commuters are female. One of the main reasons to attract the female commuters towards AC buses is the comfort level.

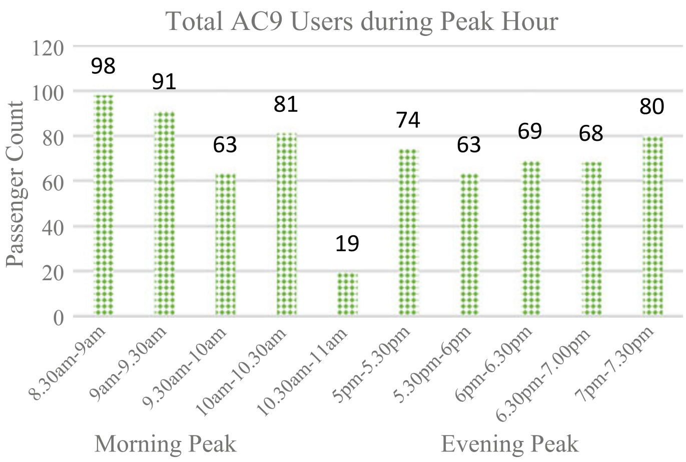

Fig. 2 AC9 passenger count during peak hour

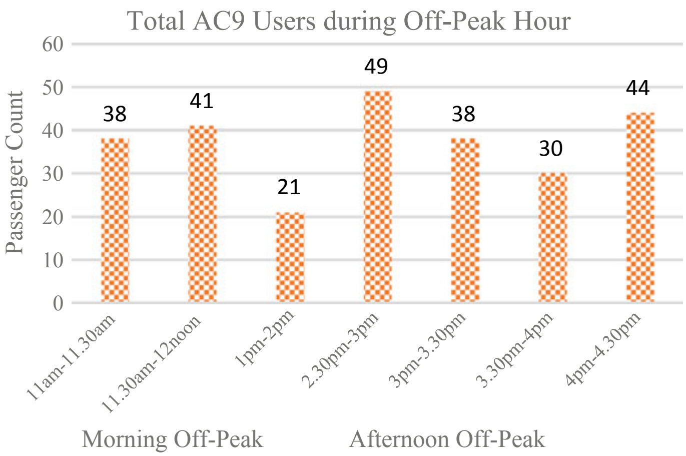

Fig. 3 AC9 passenger count during off-peak hour

The monthly income of the commuters of AC9 is shown in Fig. 9. It shows that more than 70% of the commuters have a monthly income in the range of INR 10,000–INR 100,000. This indicates that they can afford the travelling cost of AC9 which is two times higher than a non-AC bus in the same route (as shown in Table 5).

Almost 60% of the commuters own a vehicle (either 2-wheeler or 4-wheeler), yet the daily commuters prefer AC9 (almost 75% on an average, as shown in Fig. 10); due to not only the comfortable journey, it provides at such affordable price, but also there is no need to search for parking space near their destination.

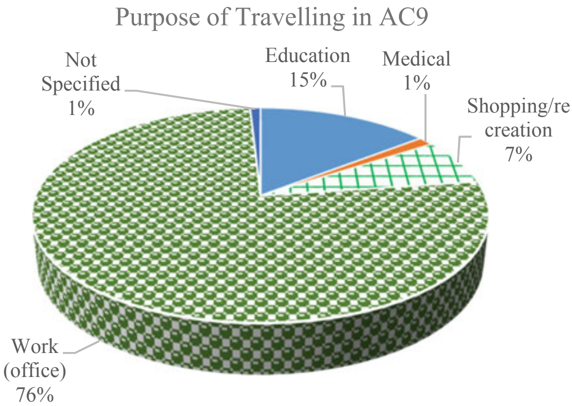

Fig. 4 Purpose of travelling in AC9 (peak hour)

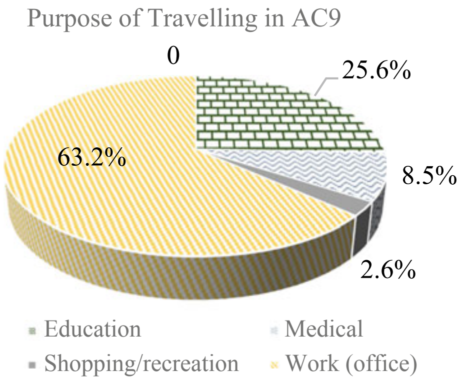

Fig. 5 Purpose of travelling in AC9 (off-peak hour)

But, the preference of AC9 will remain as long as the reliability is maintained. Reliability in terms of headway regularity, travel time variation and on-time performance must be maintained so that the mode choice is not shifted towards para-transit mode or cab or personal vehicles. Thus, it is necessary to quantify and maintain the desired service reliability.

5.2 *Quantifying Reliability*

Reliability can be determined from three perspective, viz. travel time measure, headway regularity, and on-time performance. In this study, only travel time measure and on-time performance perspective are considered to quantify reliability.

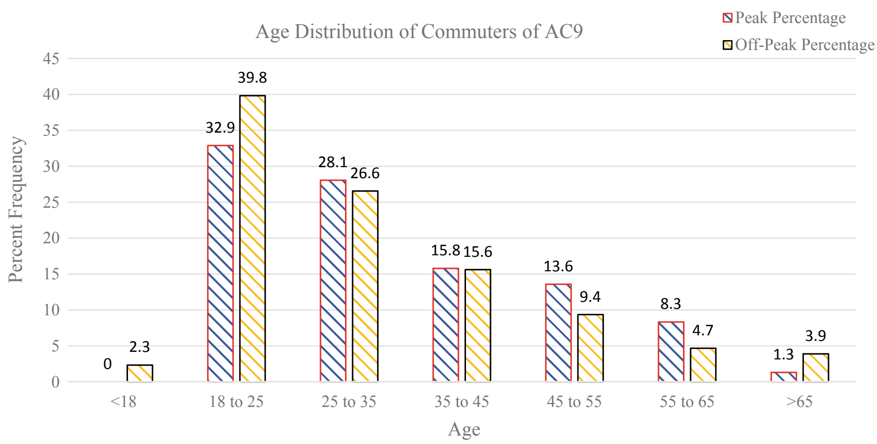

Fig. 6 Age distribution of the AC9 users

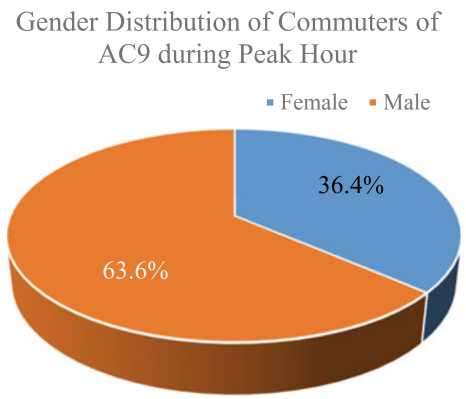

Fig. 7 Gender distribution during peak hour

5.2.1 Travel Time Measure

The travel time measure includes the measure of deviation of the actual in-vehicle travel time from the scheduled in-vehicle travel time. The operating speed of the AC9 bus as collected from the operator [West Bengal Transport Corporation (WBTC)] is 20 kmph. At this operating speed, the total time taken by the bus to cover the distance of 18 km from the origin terminal to the destination terminal is 54 min.

In Figs. 11 and 12, the actual in-vehicle travel time is compared with the scheduled in-vehicle travel time graphically for different times of the day during peak and off-peak hours, where the scheduled travel time is fixed at 54 min.

From the two figures, it is observed that during the peak hours, the actual travel time variation from the scheduled travel time is higher than the off-peak hours. The

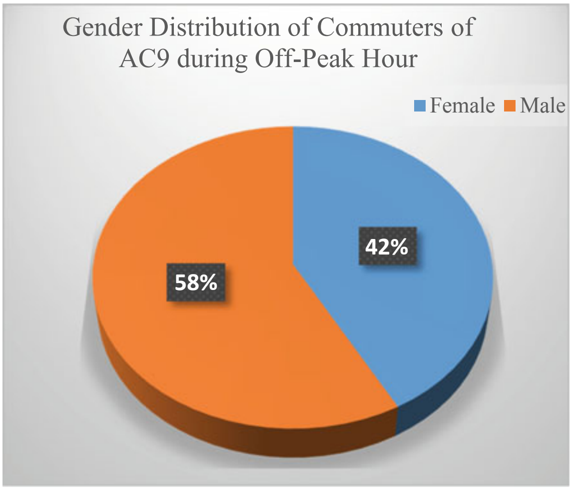

Fig. 8 Gender distribution during off-peak hour

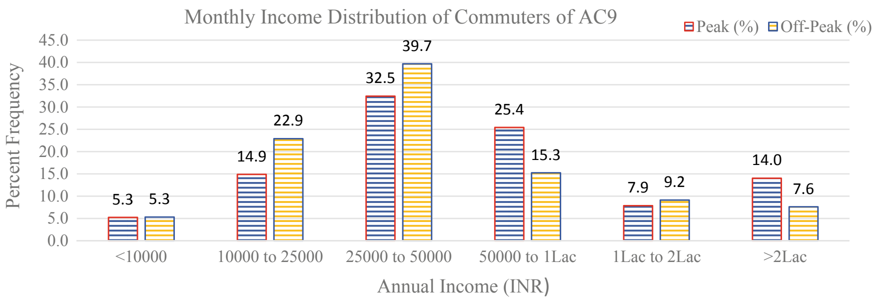

Fig. 9 Monthly income distribution of AC9 users

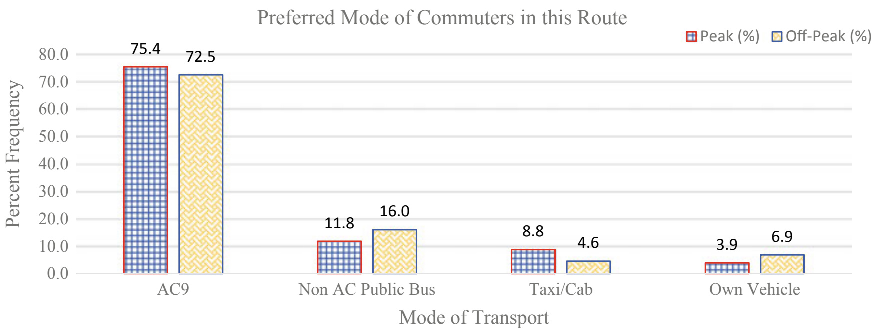

Fig. 10 Mode preference of commuters in Jadavpur to Karunamoyee route during peak and off-peak hour

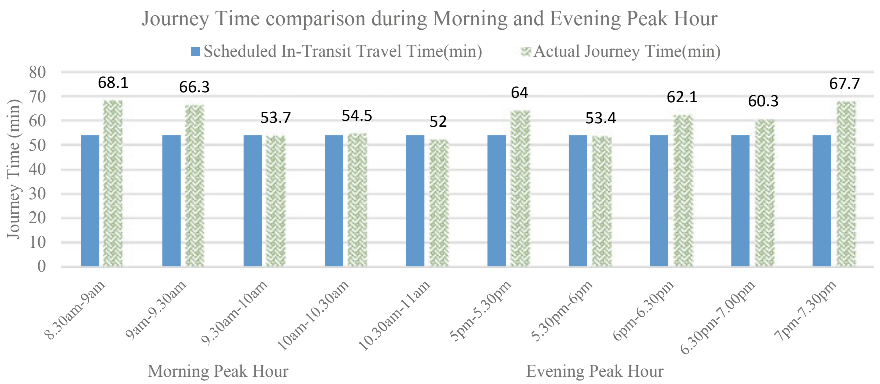

Fig. 11 Comparison of actual travel time with scheduled travel time during morning and evening peak hour

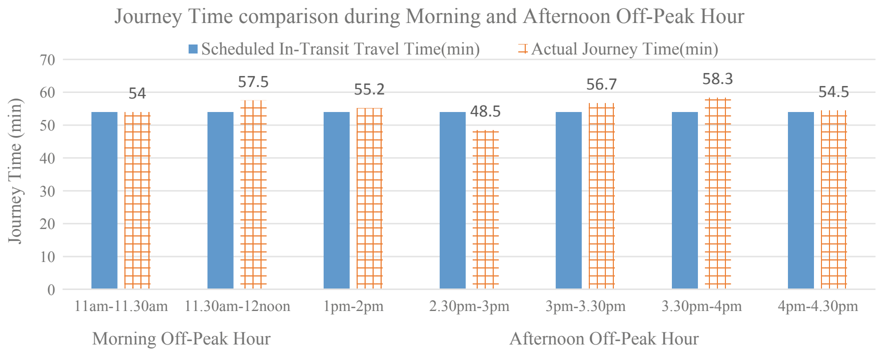

Fig. 12 Comparison of actual travel time with scheduled travel time during morning and afternoon off-peak hour

reliability is quantified in this study by three different methods (as described in Methodology section) under the travel time measure category.

- *Coefficient of Variation of Travel Time ($C_{v,TT}$)*: Using Eq. (1), the coefficient of variation of travel time is determined which is a direct measure of reliability. Higher the value, less reliable is the service. Table 2 shows the ($C_{v,TT}$) values for peak and off-peak hours. Here the value during peak hour is inevitably higher than off-peak hour but overall it is not a very high value. This indicates that the travel time variation is quite less for AC9 bus which makes the bus reliable.
- *Reliability Buffer Index (RBI)*: Eq. (2) is used here to calculate the RBI %. RBI value indicates the buffer time that has to be added to the travel time so that the commuters reach the destination on time 95% of the times he travels by AC9 bus. Table 3 shows the RBI values of AC9 for Peak and Off-Peak Hours.

Table 2 Result comparison during Peak and Off-Peak Hours for Method 1 ($C_{v,TT}$)

Travel time measure: method 1	Scheduled travel time (min)	Mean travel time (min)[a]	Standard deviation (min)[a]	Coefficient of variation ($C_{v,tt}$)
Peak hour	54	61	6	0.09
Off-peak hour	54	55	3	0.05

[a]The travel time values are rounded off to the next minute

Table 3 Result comparison during peak and off-peak hours for method 2 (RBI)

Travel time measure: method 2	Scheduled travel time (min)	Mean travel time (min)[a]	95th percentile travel time (min)[a]	RBI (%)
Peak hour	54	61	68	10.6
Off-peak hour	54	55	58	5.4

[a]The travel time values are rounded off to the next minute

This RBI value indicates that a buffer time of almost 11% and 6% of the mean travel time has to be added to the commuters' journey time for the travel during peak hour and off-peak hour, respectively.

- *Punctuality Index Based on Routes (PIR)*: This index is determined by using Eq. (3). The probability of travel time deviation within the threshold limit is determined. In this study, three different δ_1, δ_2 values are considered, viz. 5%, 10%, 20% and the PIR values for corresponding threshold limits are determined.

From Table 4, it is observed that the PIR value during peak hour remains at 0.4 up to 10% threshold limit. But when the threshold limit is increased to 20%, the PIR value is increased to 0.7. This means that for higher threshold values, the reliability is higher during peak hour as because the travel time is higher during peak hours, whereas, during off-peak hour, the PIR value reaches 1 at 20% threshold value. So, as far as the travel time reliability is concerned, AC9 has an overall good reliability.

Table 4 PIR result comparison during peak and off-peak hours for method 3 (PIR)

Travel time measure: method 3	Scheduled travel time (min)	Peak hour			Off-peak hour		
		Threshold limit δ_1, δ_2 (min) (5%)	Threshold limit δ_1, δ_2 (min) (10%)	Threshold limit δ_1, δ_2 (min) (20%)	Threshold limit δ_1, δ_2 (min) (5%)	Threshold limit δ_1, δ_2 (min) (10%)	Threshold limit δ_1, δ_2 (min) (20%)
Values (min)[a]	54	3	6	11	3	6	11
PIR		0.4	0.4	0.7	0.63	0.88	1

[a]The time values are rounded off to the next minute

Table 5 On-time performance result comparison during peak and off-peak hours (OTP)

No. of buses surveyed	Peak hours			Off-peak hours		
	14			14		
	2 min early	7 min late	OTP	2 min early	7 min late	OTP
	0	6	0.43	0	12	0.86

5.2.2 On-Time Performance (OTP)

This indicates the adherence of the arrival time of the bus to the scheduled arrival time (refer to Eq. (4)). There are various authorities who have proposed various limits up to which the arrival of the buses may be treated as on-time. In this study, the proposal of Washington Metropolitan Area Transit Authority (WMATA) has been adopted to determine the on-time performance of AC9. The proposal suggests that a service can be treated as on time if it arrives 2 min early or 7 min late from the scheduled arrival time. The analysis shows (refer to Table 5) that OTP of AC9 buses are 0.43 during peak hour, whereas they are 0.86 during the off-peak hour.

6 Recommendations

The result shows that the travel time reliability is quite high during off-peak hours than during peak hours. The main reasons for the unreliability during peak hours are:

- Huge traffic volume during the peak hours creates congested flow which increases the travel time.
- Due to high passenger volume during the peak hours, the dwell time is also increased which increases the travel time.

Even though the reliability of the AC9 bus is lower during the peak hours, Fig. 13 shows that the users are satisfied with the service. More than 75% commuters, from

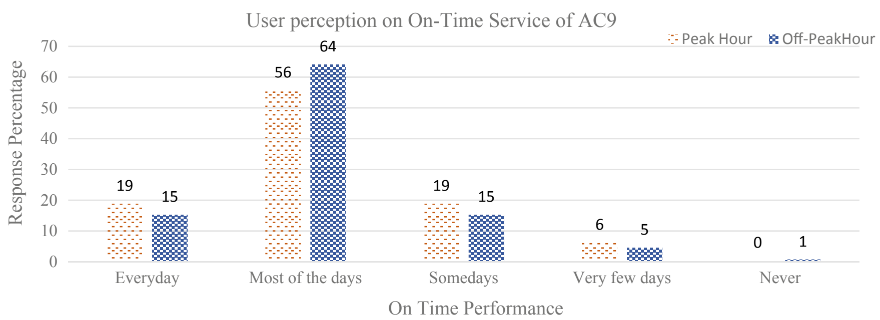

Fig. 13 User perception on the service quality of AC9

Table 6 Comparison of travelling cost of alternate modes along the route

Bus type	Travelling price (INR)	Distance (km)	Avg. price (INR/km)
AC9	15	4	2.47
	20	10	
	30	18	
Non-AC public bus	7	4	1.07
	9	10	
	10	18	
App cab sharing[b]	50–60	4	8.35–10.2
	70–90	10	
	100–120	18	
Own vehicle (fuel cost only)[a]	40–50	4	9.57–11.53
	90–110	10	
	175–200	18	

[a]Only petrol cost as on 27th April, 2018
[b]Peak hour cost as on 27th April, 2018

their daily experience, have mentioned that the bus reaches the destination on time during both peak and off-peak hours.

The reason behind the perception of users on the service quality of AC9 is that the journey is quite comfortable and affordable. Also, the unreliability of the bus is not too high which satisfies the users. But, if the reliability reduces, i.e. the travel time variation increases further, then the passengers will tend to shift to other reliable modes such as App Cab on a sharing basis or personal vehicle. The travelling cost comparison of alternate modes is presented in Table 6.

The cost comparison shows that the average per km cost of App Cab or own vehicle is within INR 10. So, if the reliability of AC9 bus reduces too much, then the passengers who have their own vehicle or can afford to travel in cab may tend to shift to those modes of transport for their work trips.

Thus, to maintain the present service condition or to improve the service, the following measures may be adopted:

- Exclusive bus lanes on the shoulder side of the carriageway may be provided throughout the major arterials and should be given priority at signalized intersections.
- The control and management system should be improved, in terms of signal designing.
- Dedicated bus stops should be provided at proper locations and should be well maintained. Boarding and alighting of passengers from buses at places other than dedicated bus stops should be prohibited and penalized.
- The AC buses should not be allowed to stop at minor bus stops unless there is a need for any passenger to alight from the bus. So, the buses which will stop at a particular bus stop should be mentioned as information to the passengers.

- Expected time of arrival at different bus stops should be displayed inside the bus by incorporating live traffic and GPS system in the AC buses, so that the passengers may reschedule their work depending on the expected time of arrival. Moreover, a display board at each major bus stop may be provided with GPS connectivity in order to display the arrival time of the next AC bus. Both of these measures will help the passengers (on and off the bus) in managing time and will attract more users.

7 Concluding Remarks

The present study aims at quantifying the travel time reliability of the air-conditioned buses in Kolkata separately for peak and off-peak hours. As a case study, AC9 bus was selected as it connects densely populated residential area in South Kolkata with a high-density employment zone in Salt Lake (Fig. 1). A large number of passengers travel by AC9 (almost 2200 passengers daily) (Figs. 2 and 3) with almost equal gender distribution (Figs. 7 and 8). The socio-economic characters of the commuters suggest that the young age group (Fig. 6) travelling for work or educational purpose (Figs. 4 and 5) avail the AC bus service more often. So, it is necessary for the bus service to be reliable and on time in order to keep the faith intact, of the existing passengers, and to attract more new passengers.

The travel time reliability is estimated from two perspectives, viz. travel time measure and on-time performance. Using Eqs. (1), (2) and (3), the travel time measures of reliability were estimated separately for peak and off-peak hours. In all the three methods, it is seen that the reliability of AC9 during peak hour is less than the off-peak hours by almost two times (Tables 2, 3 and 4). In the PIR method (Table 4), the reliability during peak and off-peak hours increases to 0.7 and 1, respectively, when the threshold limit is increased to 20% of the scheduled time. The on-time performance of AC9 is estimated using Eq. (4) and the threshold limit prescribed by WMATA was adopted. The result pointed out (Table 5) that the on-time service of AC9 is less than 0.5 during peak hours, whereas it is 0.9 during off-peak hours.

The reliability estimated is lower during peak hours, yet more than 75% (Fig. 13) of the commuters perceive the bus to be reliable and on-time and their preferred mode remains AC9 (Fig. 10). This may be due to the fact that the service is very comfortable at an affordable travelling price (Table 6) which dominates the users' mode choice behaviour. But if the reliability indices reduce further during the peak hours, there may be a tendency for the high income group commuters (Fig. 9) to shift to a reliable mode of transport such as App Cab or personal vehicle. This will adversely affect the transportation system of Kolkata and make the traffic management difficult. Moreover, it will increase the route travel time and further affect the reliability of the buses.

The unreliability of the buses during the peak hours may be attributed to the huge traffic volume during peak hours which increases the travel time and also due to high passenger volume which increases the dwell time. This may be improved by adopting some measures like providing exclusive bus lanes, providing dedicated bus stops for AC buses, prohibiting boarding and alighting at places other than dedicated bus stops, not allowing the AC buses to stop at all the minor bus stops unless there is a need for commuters to alight from the bus.

This study is limited to AC9 bus route of Kolkata. The other AC buses in Kolkata may be studied in order to identify the service quality conditions and adopt measures to improve it. In this study, travel time reliability perspective is considered. The headway perspective of service quality of buses may be included in further research. The travel time reliability measures, adopted in this study, are from previous literatures which may be altered in future research and the reliability may be estimated.

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Development of Nomogram for Travel Characteristics

M. S. Brahma Pooja, Naidu Mahalakshmi Villuri, and CSRK Prasad

Abstract Trip length is the length of a trip measured in the distance or in time. In this paper, different parameters are taken and their influence on the trip length is studied for various cities in India. The overall goal of this study is to estimate the trip length by generating models. These generated models are used to develop ready-made nomograms. In this study, the development of models for trip length with relationship between variables such as dependent (trip length) and independent (socio-economic and land-use data) variables by using regression analysis. In this study, Microsoft Excel software is used for developing models. PyNomo software is for development of nomograms for quick estimate. Among all socio-economic and land-use data parameters average monthly household income, total population, total literates, commercial area (%), industrial area (%) and intermediate public transportation vehicles are correlated to trip length.

Keywords Nomograms · Dependent variable (trip length) · Independent variables (socio-economic characteristics and land-use data)

1 Introduction

Transportation is the movement of persons, animals, cargo from one location to another location. In the civil engineering world, transportation engineering plays an important role. In India, with growing of population, urban areas are also increasing day by day. By this, trip lengths of the road users are increasing. For development of

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urban areas, transportation planning process is used for forecasting the travel demand and planning of road network.

Trip length is the length of the trip measured in the distance or time. Trip length is mainly used in the various areas of transportation and environmental aspects. Trip distribution is the second component in the traditional four-step transportation forecasting planning process. This trip distribution process is used to predict the number of trips originate and end from origin to destination zones. In that the trip length is used as the main component. It is difficult to find the trip length by conducting regular surveys. The cost and time are taken for these regular surveys very high. Alternate solution by using simple linear regression analysis and nomograms is the need of the day.

To obtain the trip length, linear regression analysis and nomograms concepts are used. There are two main objectives for this research:

- To formulate different models for trip length for various categories of cities.
- To develop the nomograms by using the models for various categories of cities in the PyNomo software.

1.1 Regression Analysis

Regression analysis is a method used in the statistical data analysis. It is the study of relationship between one dependent variable and two or more independent variables to forecast or to estimate the values. The general model for linear regression analysis is

$$Y = aX + b$$

Here, Y is the dependent variable which is the predict value and X is the independent variable and b is the intercept and a is the slope.

The model of the regression shows analysis of variance (ANOVA) which is in statistical model.

1.2 Nomograms

A nomogram is a diagram which allows the fast calculation for a specific formula represents in graphical method. Nomograms are also called as an alignment chart or nomograph. These graphical calculators were invented in 1880 by 'Philbert Maurice d'Ocagne'. Nomogram consists of a set of numbered scales, for each variable in the formula. A straight line is placed across the known values to find out the unknown value. This line is called as index line or isopleths. So, if there is a chance to know the two variables, it is easy to find the third variable. The distances of nomogram along and between the scales depend upon the range of calculation the nomogram covers. Nomograms are different types.

The simplest nomogram is single scale, which contain numbers on both sides of the scale. This is called conversion scale. This nomogram provides conversion between two variables. This paper shows single-scale nomogram as a first example.

The most common type of multi-scale nomogram consists of three parallel straight scales. In this nomogram, the equation contains three variables. Example for this type of nomogram is discussed later in the paper.

To develop the nomogram, PyNomo software is useful. PyNomo is a sub-module of Python software or library to build PDF nomograms. PyNomo is a powerful, free software package for drawing precision nomograms. It is written by Leif Roschier. The output from PyNomo is in vector form in a PDF or EPS files.

2 Literature Review

Kadyali and Lal (2007) and Mathew and Krishna Rao (2007b) documented the process of travel demand forecasting essentially consist of four stage model transportation. In this, the definition of trip length was explained, i.e., trip length is the length of a trip measured in the distance or in time. Giannopoulos (1977) described the impact of distribution of land uses upon trip characteristics specially on trip length. This paper discusses about the relationship between the trip length classified by mode and purpose and the distance of one trip end from the city centre. By this a quantitative expression of relation between transport and land use is obtained. Analyzed the expression by examining the data from the Greater Athens area, London. It is concluded that the trip length distributions are connected to income and other economic or social parameters in an urban area. Moniruzzaman et al. (2013) explored about the factors that influence the use of different modes of transportation and related trip length for seniors in Montreal Island. By doing this survey, the geographical variability in travel behaviour of seniors is found. It is concluded that by increasing the age, trip length will decrease. Probably, it increases walking but not the length of walking trips. Mathew and Krishna Rao (2007a) explained about the travel demand modelling in transportation engineering which implies the procedure for predicting the travel decision of people by means of an appropriate system of zones. It aims at explaining where the trips go, what mode the people choose, and which routes they prefer. It provides a zone-wise analysis of the trips followed by distribution of trips, split the trips, mode wise based on the selection of the travellers and finally assigns the trips to the network. This process helps to understand the effects of future developments in the transport networks on the trips. Doerfler (2009) described about the PyNomo software and how nomograms were developed in this paper. The paper described about the different types of nomograms based on the format of the equation. By using any complicated or general for of equation, it can produce the output which plotted as a nomogram. Evesham (1986) discusses about the origin and development of nomography. The most common type of consists of three parallel scales are constructed in such a way that each represents a variable from a given three variable relationships. A straight line passing through particular points on any two of the scales which shows the results in the middle scale which was the third scale of the three variable scale nomogram.

3 Study Area

Cities which are densely populated and have a good share of trips per day have been selected in this study. Data is selected from 46 Indian cities. Data related to these 46 cities about various parameters and land-use parameters have been obtained using various sources and surveys. In this study, the land-use parameters are taken as a percentage. Parameters of the study area which are independent variables are listed below in Tables 1 and 2. And the trip length, the dependent variable listed in Table 3.

Excluding walk trips: The total number of trips made daily by public and private transport in the area from the exclusive of the walk.

Including walk trips: The total number of trips made daily by public and private transport in the area with a walk.

Average density of the city (persons/km^2): Ratio of the total population of city to the size of the city.

Total population (in lakh): Total population in the study area.

Total literates (in lakh): Total literates in the study area aged above 7 years who can read and write.

Average monthly household income (₹): Average monthly HH income as per comprehensive development plan/master plan.

Table 1 Socio-economic parameters

Socio-economic parameter	
Average density of the city (persons/km^2) (ADC)	Average monthly expenditure on transport to total income (₹) (AMET)
Total population (in lakh) (TP)	Average monthly household income (₹) (HI)
Total literates (in lakh) (LI)	Private transport vehicles two wheeler (2W)
Private transport vehicles taxi and cars (4 W)	

Codes for parameters are given in the parenthesis

Table 2 Land-use parameters

Land-use parameters	
Administrative municipal area (%) (AMA)	Recreational area (%) (REC)
Residential area (%) (RES)	Public and semi-public area (%) (PASPA)
Commercial area (%) (CA)	Transport area (%) (TA)
Industrial area (%) (IA)	Water bodies and coastal area (%) (WB)
Agricultural area (%) (AA)	

Codes for parameters are given in the parenthesis

Table 3 Trip length consider for study area

Trip length
Excluding walk trips
Including walk trips

Average monthly expenditure on transport to total monthly income (₹): Ratio of expenditure on transport to the total income as per comprehensive mobility plan.

4 Methodology

To generate models from linear regression analysis, the socio-economic parameters and land-use parameters are considered. According to the city topography and the location of that city, data of the selected parameters are dissimilar for every city. The cities are categorized based on two characteristics. First one is population. Population have different sizes for every city. So, the population is classified into three groups. First one, whose population is less than 10 lakhs, indicated as P1. The second one, whose population ranges from 10 to 40 lakhs, indicated as P2. The third one, the population of the city which is greater than 40 lakhs, indicated as P3. The categorization based on population is shown in Fig. 1.

From the growing of population, the area of the city also increases day by day. The change in the city area may give changes in the behaviour of traffic. So, the second city character is taken as the area. The area again classified into three groups. Initially, small cities which have area less than 300 km^2 indicated as A1. Next, the medium range areas which have 300–1000 km^2 indicated as A2. Finally, the large

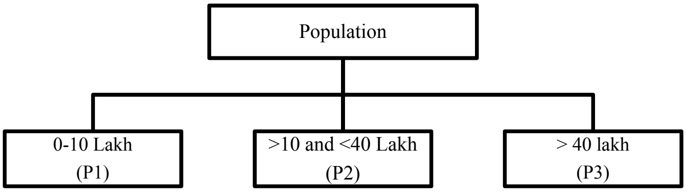


Fig. 1 Population-based city category

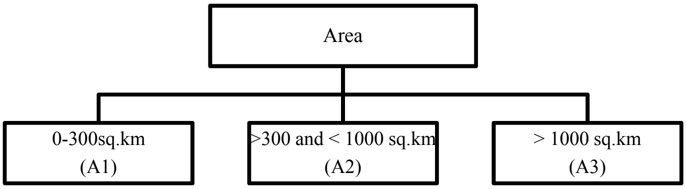


Fig. 2 Area-based city category

cities which have area greater than 1000 km^2 indicated as A3. The categorization based on area is shown in Fig. 2.

4.1 Step by Step Methodology for This Work Is Given Below

- By this categorized components, the selected socio-economic parameters and land-use parameters of cities are considered as inputs.
- From this selected data, the regression analysis is carried out.
- These parameters are now used to bring out the relation with trip length by using simple linear regression analysis.
- The main intent of this work is to estimate the trip length from the selected cities.
- From framed regression models, nomograms are developed.

5 Generation of Regression Models

To generate the regression model, socio-economic parameters and land-use parameters are independent variables, and trip length is the dependent variable. By doing the data analysis in excel, the model is generated. With the consideration of the *t*-stat and *p*-value the model are developed.

- Here, *t*-stat is to determine whether the slope of the regression line differs notably from zero.
- *P*-value is the null hypothesis which should be less than significant level.

Table 4 Models for trip length (km) for excluding walk trips with a single independent variable by category population

Models for trip length for excluding walk trips by category population wise								
	P1			P2			P3	
V	AMA	4.362 (3.66)	*V*	RES	0.054 (2.75)	*V*	TP	0.039 (6.58)
	Intercept	−208.647 (−3.54)		Intercept	4.835 (9.65)		Intercept	4.889 (7.93)
F	R^2	0.81	*F*	R^2	0.79	*F*	R^2	0.86
	F-test	13.42		*F*-test	7.57		*F*-test	43.37

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value
The model from P1 is (from above table) read as
Trip length in km (excluding walk trips) = 4.362 * AMA − 208.647

- R^2 shows closeness of independent, dependent data are fitted in the model (desirably equal to 1).

To develop the nomograms, the regression is done by taking as two variables and three variables. Here, two variables mean one independent variable and one dependent variable. Three variables mean two independent variables and one dependent variable. Some of the best regression models for trip length for excluding and including walk trips by their category wise are listed below.

5.1 *Models for Trip Length for Excluding Walk Trips by Category Wise*

See Tables 4, 5, 6 and 7.

5.2 *Models for Trip Length for Including Walk Trips by Category Wise*

See Tables 8, 9, 10 and 11.

6 Development of Nomograms

To develop nomograms, PyNomo script is useful. The writing of script is very easy. Awareness of algebra is required for organizing the formula in order to create a

Table 5 Models for trip length (km) for excluding walk trips with a two independent variable by category population

Models for trip length for excluding walk trips by category population wise					
	P1			P2	
V	TP	0.97 (17.15)	*V*	ADC	0.000025 (9.79)
	CA	1.98 (15.80)		RES	0.006 (12.97)
	Intercept	−3.587 (−6.87)		Intercept	0.612 (31.54)
F	R^2	0.99	*F*	R^2	0.99
	F-test	208.64		*F*-test	61.17

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value
The model from P1 is (from above table) read as
Trip length in km (excluding walk trips) = 0.97 * TP + 1.98 * CA − 3.587

Table 6 Models for trip length (km) for excluding walk trips with a single independent variable by category area

Models for trip length for excluding walk trips by category area wise					
	A1			A2	
V	HI	0.000171 (2.48)	*V*	AA	1.32 (9.45)
	Intercept	3.59 (2.97)		Intercept	−0.44 (−0.57)
F	R^2	0.60	*F*	R^2	0.98
	F-test	6.17		*F*-test	89.31

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value
The model from A1 is (from above table) is read as
Trip length in km (excluding walk trips) = 0.000171 * HI + 3.59

standard type of equation that PyNomo supports. Some of the examples for two variables and three variable nomograms of are shown below:

For two variables models the nomograms will be as below:

- The independent variable is administrative municipal area (%) and the dependent variable is trip length in km (excluding walk trip) is shown in Fig. 3.
- The independent variable is average monthly household income (Rs.) and the dependent variable is trip length in km (excluding walk trip) is shown in Fig. 4.

For three variables models the nomograms will be as below:

Table 7 Models for trip length (km) for excluding walk trips with a two independent variable by category area

Models for trip length for excluding walk trips by category area wise		
	A1	
V	LI	0.157 (5.51)
	CA	2.396 (11.45)
	Intercept	−0.173 (−0.27)
F	R^2	0.97
	F-test	65.86

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value. The model from A1 is (from above table) read as
Trip length in km (excluding walk trips) = 0.157 * TL + 2.396 * CA − 0.173

Table 8 Models for trip length (km) for including walk trips with a single independent variable by category population

Models for trip length for including walk trips by category population wise					
	P1			P3	
V	AMA	0.171 (2.91)	*V*	ADC	0.000215 (4.73)
	Intercept	−3.012 (−1.09)		Intercept	1.467 (2.94)
F	R^2	0.36	*F*	R^2	0.95
	F-test	8.47		*F*-test	22.40

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value
The model from P3 is (from above table) is read as
Trip length in km (including walk trips) = 0.000215 * ADC + 1.467

- Here, the independent variable is total literates and average monthly household income and the dependent variable is trip length in km (excluding walk trip) (Fig. 5).
- Here, the independent variable is administrative municipal area and recreational area (%) and the dependent variable is trip length in km (including walk trip) (Fig. 6).

Table 9 Models for trip length (km) for including walk trips with a two independent variable by category population

Models for trip length for including walk trips by category population wise					
	P1			P3	
V	AMA	0.267 (3.57)	*V*	TP	0.039 (8.41)
	REC	0.777 (3.57)		TA	0.062 (1.7)
	Intercept	−9.513 (−2.1)		Intercept	3.832 (5.67)
F	R^2	0.48	*F*	R^2	0.93
	F-test	6.69		F-test	38.98

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are t-stat value
The model from P1 is (from above table) is read as
Trip length in km (including walk trips) = 0.267 * AMA + 0.777 * REC − 9.513

Table 10 Models for trip length (km) for including walk trips with a single independent variable by category area

Models for trip length for including walk trips by category area wise					
	A1			A2	
V	AMA	0.169 (3.28)	*V*	REC	1.108 (2.67)
	Intercept	−3.058 (−1.26)		Intercept	1.697 (1.41)
F	R^2	0.41	*F*	R^2	0.70
	F-test	10.77		F-test	7.17

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are t-stat value
The model from A2 is (from above table) read as
Trip length in km (including walk trips) = 0.169 * AMA − 3.058

7 Conclusion

Nowadays transportation is increasing rapidly due to urbanization, planning for the safe and convenient facility is becoming a challenge, to overcome these problems, this is an effort to provide a little help to the transportation planning area. For that purpose, some models were suggested to estimate trip length (km) for excluding and including walk trips.

- Total literates and commercial area of the city are in relation with trip length in km (excluding walk trips) which gives the R^2 value is 0.97.

Table 11 Models for trip length (km) for including walk trips with a two independent variable by category area

Models for trip length for including walk trips by category area wise		
	A3	
V	AMA	0.32 (3.19)
	REC	0.23 (3.1)
	Intercept	−16.04 (−3.07)
F	R^2	0.83
	F-test	5.24

Note: *V* stands for variable, *F* stands for model features
The values shown in brackets are *t*-stat value
The model from A1 is (from above table) is read as
Trip length in km (including walk trips) = 0.32 * AMA + 0.23 * REC − 16.04

- Best parameters for trip length for excluding walk trips are administrative municipal area (%), agricultural area (%), and recreational area (%) where as the regression values are 0.81, 0.98, and 0.79.
- Total population and recreational area (%), total population and transport area (%) are showing good relation with trip length in km (excluding walk trips) and the regression values are 0.85 and 0.93.
- Best parameters for trip length for including walk trips are average density of the city (persons/km^2), recreational area (%), private transportation vehicle are 0.95, 0.70, and 0.97, respectively.
- With these parameters the trip length for any developing city can be easily establish without doing any survey which is time consuming, reducing human effort, and economic.
- Hence, by the regression analysis the results obtained gives the approximate equal values for trip length (excluding and including walk trips in km).

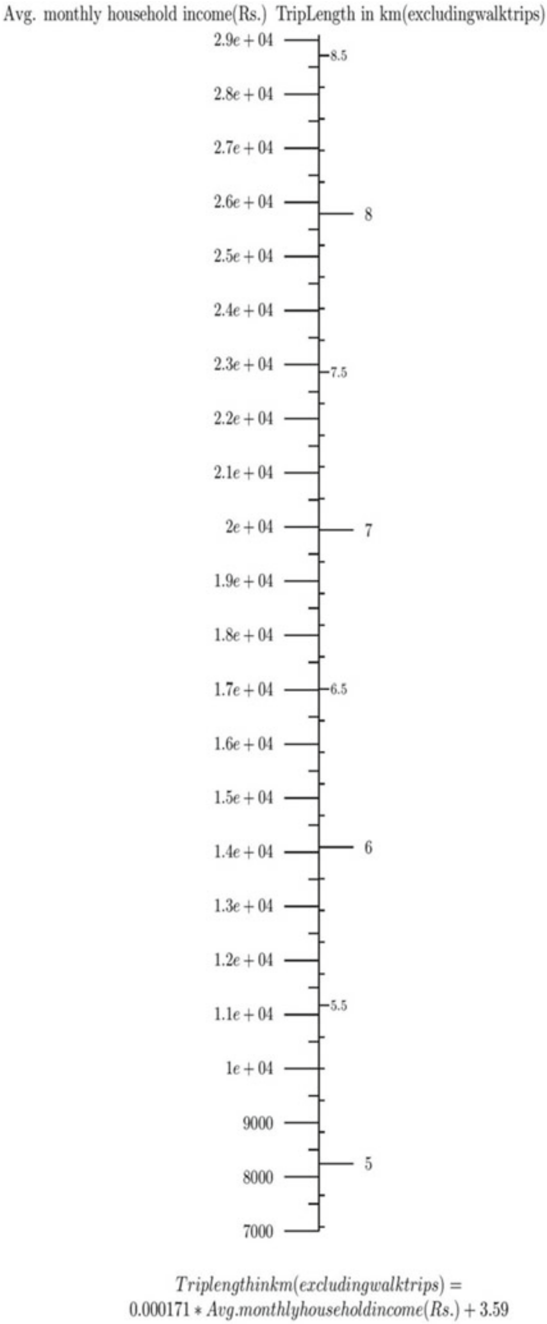
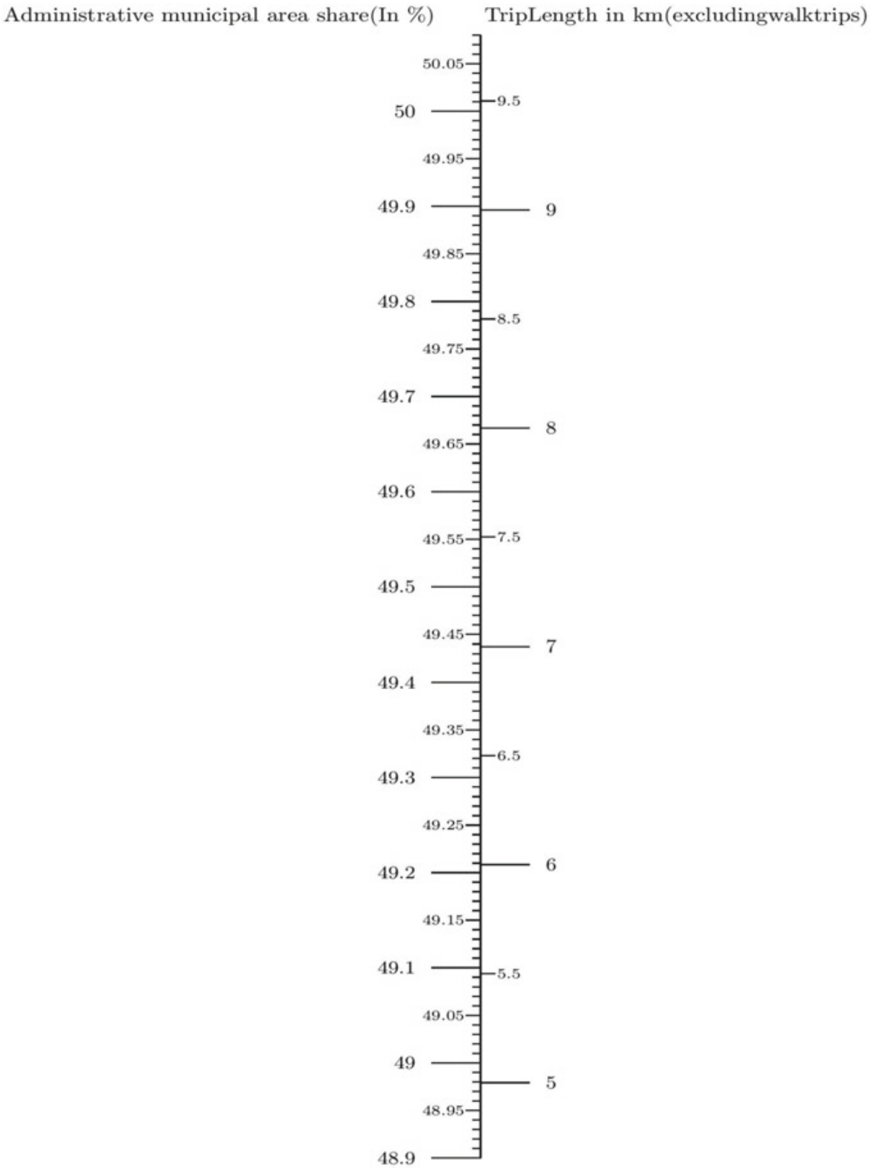


Fig. 3 Avg. monthly household income (Rs.) versus trip length in km (excluding walk trips)



$$TripLengthInkm(ExcludingWalkTrips) = 4.362 * AdministrativeMunicipalArea(Share) - 208.647$$

Fig. 4 Administrative municipal area (%) versus trip length in km (excluding walk trips)

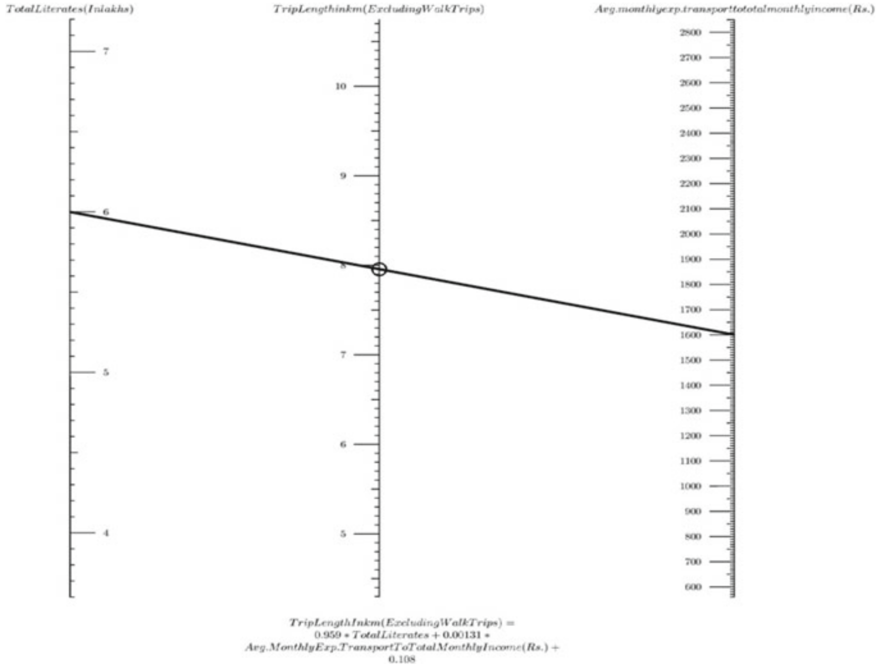


Fig. 5 Total literates and avg. monthly exp. transport to total monthly income (Rs.) versus trip length in km (excluding walk trips)

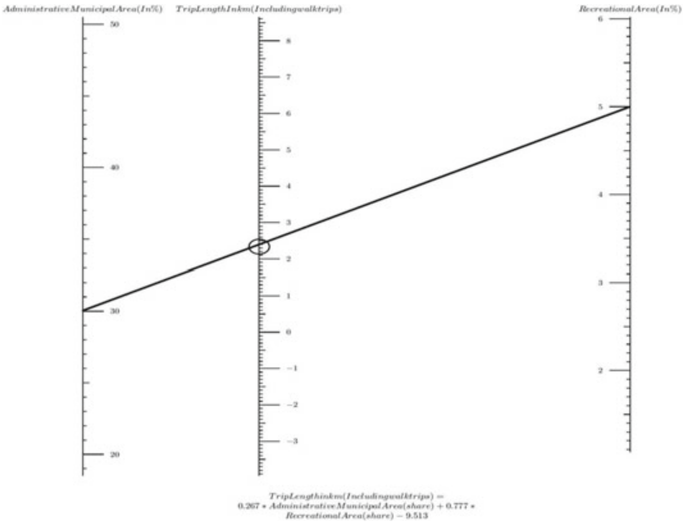


Fig. 6 Administrative municipal area (%) and recreational area (%) versus trip length in km (including walk trips)

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A Comparison Between Equilibrium Model and a Day-to-Day Model for Transit Rider's Route Choice with Calibrated Information Parameter

V. M. Ashalakshmi, S. Padma, Bino I. Koshy, and Neelima Chakrabarty

Abstract Route choice behaviour of transit riders is the key factor that affects the performance of a transit network. One of the major uses of technology in transit assignment is giving prior information to the riders about the trip which will influence the choice of the rider. A day-to-day model is proposed here which will give the variation of route choice with the provision of static information and is compared with the deterministic user equilibrium (DUE) model results. A factor for the importance of information is introduced to get the effect of information on transit rider's route choice. Sensitivity of the model is analysed by varying demand and information. A laboratory experiment along with a field survey is conducted to get the factor for the importance of information. Two key factors that affect route choice are considered: (i) experience acquired in the route on the previous day and (ii) information provided for the users about the route section. Multiple linear regression analysis is carried out to get the importance of information. The coefficients for information varies between 0.13 and 0.49.

Keywords Route choice · Transit assignment problem (TAP) · Deterministic user equilibrium (DUE) model · Day-to-day model

1 Introduction

From the earlier times, Traffic Assignment Problem (TAP) has been a major study area for transportation planners. It is generally believed that the flows in the network are in the state of user equilibrium (UE) defined by Wardrop (1952) in his first

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principle. It states that a stable condition is reached only when no traveller can improve his travel time by unilaterally changing routes. It means the travel time on all used paths is equal, and (also) less than or equal to the travel time that would be experienced by a single vehicle on any unused path. Wardrop assumes that each passenger is well aware of the travel time that is likely to occur in all the routes and choose the route with minimum travel time. But in the real scenario, this assumption is generic as each person has their own perception of travel time based on which decisions are taken.

Information technology is developing day by day which can be integrated with TAP for better results. Advanced traveller information system (ATIS) is a branch of intelligent transportation system (ITS) which will help the planners to improve the traffic condition on the road network. The penetration of ATIS in the route choice behaviour of trip makers is found to have a significant effect on the route choice, and thereby link flows (Yang and Meng 2001). Before introducing any information system to a transportation system, its economic viability has to be checked. The system is said to be effective if there is a relevant improvement in the transit rider's route choice due to the provision of information. Modelling a transit rider's route choice and incorporating information to it is hardly a simple task. This paper focuses on the effect of static information on the transit rider's line choice using a DUE and a day-to-day model.

1.1 Objectives

- To study the line choice behaviour of transit riders, without any information about the transit network.
- To study the effect of information in line choice.
- To find out factor for importance of information in transit rider's route choice.
- To study the variation in route choice with the variation of factor for importance of information.
- To compare the route choice behaviour in DUE model and day-to-day model.

2 Literature Review

2.1 Deterministic User Equilibrium (DUE)

Beckmann et al. (1956) were the first to rigorously formulate Wardrop's conditions mathematically which is used for nearly half-century. The basic assumption of Wardrop is that users are aware of cost of travel that would be possibly experienced in all routes, which is not true. Therefore, passenger's perception also has to be incorporated (Sheffi 1985).

Frequency of transit services is an important attribute that varies transit assignment from route assignment for car as a mode. Frequency-based transit assignment was initially carried out by Spiess and Florian (1989) and "route section" concept, for incorporating congestion in the model, was introduced by De Cea and Fernandez (1993). Wie et al. (1995) stated that passengers adjust their departure time based on the travel time and waiting time, and he proposed a discrete-time formulation for route departure equilibrium problem.

For getting real-world conditions, dynamic nature of world has been introduced into the models. DE. Boyce et al. (1995) considered instantaneous dynamic behaviour of automobile drivers which can be modelled using static transit assignment techniques. Cantarella (1997) extended the study into modelling the elastic demand. Hamdouch et al. (2011) introduced seat availability in transit assignment for modelling.

2.2 Day-to-Day Models for Traffic Assignment Problem

Earlier most of the studies were based on the equilibrium flow values but later researchers felt the importance of considering the evolution of these equilibrium values. Cascetta and Cantarella (1991) were one of the first to introduce within-day dynamics and day-to-day dynamics in traffic assignment modelling. Travellers make their decisions based on experience in previous days known as learning process (Yang and Zhang 2009; Seetharaman 2017). Weighted average of experience is taken over the memory length.

For day-to-day models, link-based approach gives good results as compared to path-based approach (X. He et al. 2010). Cantarella and Watling (2016) introduced discrete-time system and continuous-time system in modelling which will give the dynamic nature more clearly. Seetharaman (2017) studied the impact of passenger trade-off between travel time and uncertainty on passenger's route choice was assessed. All these studies show that the day-today models give a distribution of stationary flows with its mean value equating to the user equilibrium.

2.3 Effect of Information on Route Choice

After the implementation of ATIS like technologies, the flow pattern in network is found to be varying. The provision of real-time information is found improve the traffic conditions on the road and traveller's adjust their departure time based on the information provided and experience attained in the network (Hu and Mahmassani 1997). The perception of passengers gets updated based on the previous day's experience and information provided based on which they choose route for their present trip. The impact of ATIS or real-time information on route choice is studied by various researchers (Yang and Meng 2001; Avineri and Prashker 2006; Paolo Delle

Site 2018). The information dissemination happens through various devices (Variable Message Signs, Countdown Systems etc). Other than these information devices, social communication will also have an impact on route choice. Zhang et al. (2018) proposed a model wherein the experiences of fellow travellers or 'friends' was used as a source of information in revising the route choice behaviour of individuals.

3 Modelling for Transit Assignment Problem in Informed Scenario

De Cea and Fernandez (1993) use route section approach to incorporate congestion effect. Route section is the portion of route between two consecutive transfer nodes. The same route section can serve two or more routes. The cost function in De Cea and Fernandez (1993) consists of an additional waiting time which stands for "congested" condition that occurs when transit services have limited capacity.

3.1 Example Network

Equilibrium model proposed by De Cea and Fernandez (1993) for transit assignment in the congested condition is coded for a dummy network having three nodes as shown in Fig. 1.

A is the origin node and B is the destination node, x and y are transfer points and 4 lines are serving the network. $S1$, $S2$, $S3$, $S4$, $S5$, $S6$ represent the route sections and $L1$, $L2$, $L3$, and $L4$ represent the line of service through the network. Details of different lines are given in Table 1.

The in-vehicle travel time and waiting time for each route sections is given in Table 2.

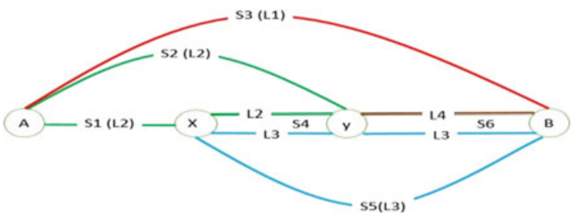


Fig. 1 Network Diagram (De Cea and Fernandez 1993)

Table 1 Details of network

Line	Nodes	In-vehicle travel time (min)	Frequency (veh/h)
$L1$	A-B	25	10
$L2$	A-x	7	10
	x-y	6	
$L3$	x-y	4	4
	y-B	4	
$L4$	y-B	10	20

Table 2 Route section data for the network

Route Section	$S1$	$S2$	$S3$	$S4$	$S5$	$S6$
In-vehicle travel time (min)	7	13	25	5.4	8	9
Waiting time (min)	6	6	6	4.3	15	2.5
Total time (min)	13	19	31	9.7	23	11.5
Capacity	50	50	50	70	20	120

3.2 General Assumptions

There will be many alternative routes that a traveller may choose in the network, but as per Wardrop's first principle each person is assumed to choose the shortest path available in the network. That is, he/she selects that particular route which minimizes his/her total travel time (in-vehicle travel time + waiting time).

External congestion due to other traffic is neglected, and in-vehicle travel time in each route section is assumed to be static. But due to the assumption of the limited capacity of transit services if passenger volume increases, the waiting time increases which is reflected by total travel time increase. This additional time due to congestion is accounted with the waiting time assuming when volume is high people may not be able to board the transit service and they will wait for the next service. This congestion is modelled through a BPR formulation of DeCea and Fernandez (1993).

The passengers are assumed to have a memory length of one which means that each passenger decides their route based on the previous day's experience and information provided on the current day. Passenger inter-arrival and inter-arrival time of buses are assumed to have an exponential distribution, and the information provided for the passengers is assumed to be reliable, and the factor of importance of information is assumed to vary between 0 and 1.

3.3 Factor for Importance of Information

Different people give different importance to information. It depends on several attributes such as the behaviour of people, educational level, reliability of information, etc. *A* factor, θ, is introduced in this study which represents the importance given to the information. This factor can have value between zero and one. If θ is equal to one, then the decision is made only based on the experience, and if θ is zero, the decision is made only based on the information. A value of θ near to zero means the travellers give more importance to information, and a value near to one means the travellers give more importance to experience.

3.4 Cost Function

Similar to De Cea and Fernandez (1993) transit stops are considered to work as a complex queueing system in which congested condition arises resulting in increase of waiting time for transit services. Here the in-vehicle travel time and total waiting time are considered as experience of passengers as it is the actual time of travel experienced. For route section *s*, the experience will be,

$$e_s = t_s + \left(\frac{\alpha}{f_s}\right) + \beta \times \Psi_s \times \left(\frac{V_s + V^s}{K_s}\right) \tag{1}$$

where t_s be the in-vehicle travel time, α is the factor that depends on distribution of inter-arrival time of buses and passengers (as inter arrival time of buses were assumed to be exponential α will be equal to 1), f_s is the total frequency of buses in the route section, β is the parameter whose calibrated value is taken as 10 (De Cea and Fernandez), V_s is the number of passengers that compete with V^s, V^s is the total number of passengers boarding the route section at a node, and K_s is the practical capacity of route section. The functional form which Ψs should take should be strictly monote, namley, power form. Therefore, the second term in the equation represents the waiting time and the third term stands for additional waiting time due to congestion.

The cost function consists of experience of the passenger in the previous trip and information provided in the current day. The cost function for route section s is given below,

$$\text{Cost of travel } \mathrm{ca}_s = \theta \times e_s + (1 - \theta) \times \inf_s \tag{2}$$

3.5 DUE Model for Transit Assignment

Traffic assignment is one of the major steps in urban transportation planning. The number of passengers using each route is forecasted to know the performance of the network and also to know the need for a new facility.

3.5.1 Algorithm of DUE Model

Step 0: (initialization) Assign the traffic to the minimum cost path which gives a volume matrix having flow through each route section $\{V_s^1\}$. Set $n = 1$.

Step 1: (update) Update the cost of travel through each route section based on the new flow value.

$$\text{Experience } e_s^n = \text{ca}_s^n + k_s \times V_s^n \tag{3}$$

$$\text{Cost of travel } \text{ca}_s^{n+1} = \theta \times e_s^n + (1 - \theta) \times \inf_s \tag{4}$$

Step 2: (direction finding) Perform all-or-nothing assignment based on current set of cost of travel, $\{\text{ca}_S^n\}$. This yields an auxiliary flow patterns, $\{y_S^n\}$

Step 3: (move) Find the new pattern by setting

$$V_s^{n+1} = V_s^n + \frac{1}{n} \times \left(y_s^n - V_s^n\right) \tag{5}$$

Step 4: (convergence check) If V_S^{n+1} and V_S^n are sufficiently close, stop; otherwise set $n = n + 1$ and go to step 1.

3.5.2 Sensitivity Analysis

Sensitivity analysis of model on example network helps to evaluate the performance of model under various input parameters. In real network, a wide range of parameter values exists. In the model, two parameters considered are demand and θ. Demand-in an *O-D* pair can have values in a wide range depending on the network characteristics and working status of people. θ can have any value between 0 and 1 and depends on educational qualification, gender and age group of people travelling.

Differing Demand

To test the sensitivity of route flow to varying demand, the model is run for four values of demand and the results are shown in Fig. 2.

The results indicate that when demand increases the flow through route sections also increases, and for higher demand, the route sections having high travel time are also used by passengers.

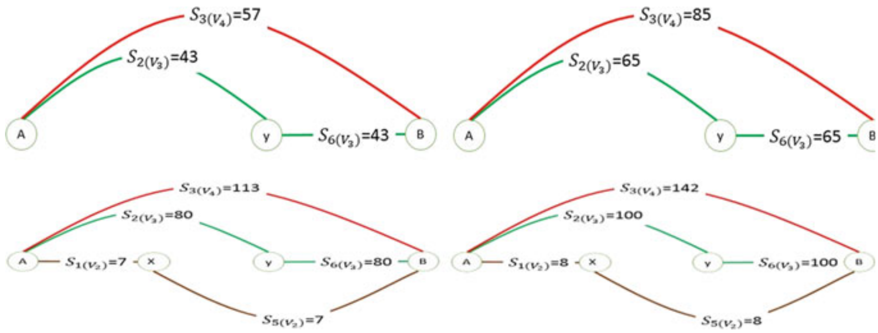


Fig. 2 DUE Route section flow for demand 100, 150, 200 and 250

In real world, the demand is dynamic and the model proposed uses static demand. But the model can be used for any value of demand.

Differing θ values

θ is the coefficient for effect of information. The value of θ varies between 0 and 1. A large θ would ensure that passengers give more importance to experience compared to information. If θ is zero all the passengers choose the minimum informed travel time path, the effect of congestion is not taken into account in information. This will result in high congestion in the minimum path, and all other paths are left idle; as the current model doesnt account for dynamic information. The model assumes static information through out the simulation process as well as during DUE. This is a limitation of the study. Route section flows for varying values of θ and keeping all other parameters constant are shown in Fig. 3.

The information is given based on free-flow travel times, and choice 3 is the route having minimum free-flow travel time (Fig. 4).

These graphs highlight that when θ is very small passengers make their decisions based on the information and choose the routes which have minimum informed travel time. As θ value increases, passengers give importance to congestion also which will result in flows through route sections which were idle when θ is zero. For small variation in θ, flow values will not change.

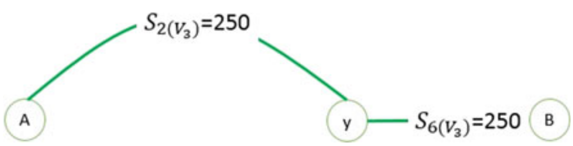


Fig. 3 DUE Route section flows for $\theta = 0$

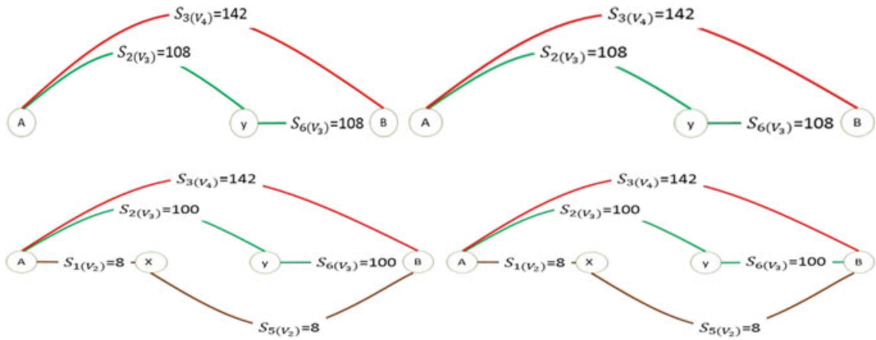


Fig. 4 DUE Route section flows for θ 0.25, 0.5, 0.75, and 1

3.5.3 Convergence Test

DUE model assigns the flow to different route sections, and the equilibrium flow is obtained. The MSA algorithm converges after 40 iterations to an equilibrium solution.

Simulation runs are carried out for different values of information factor and flows are obtained.

3.6 A Day-to-Day Model for Transit Assignment

The passengers often change their route choice whenever they feel the travel time in current route is higher than the other routes. These dynamic conditions are better explained using day-to-day models. In this model, the position of passenger at each time interval is obtained. The time step is set to be 324 s as the minimum travel time route section has a travel time of 5.4 min.

3.6.1 Algorithm of Day-to-Day Model

Step 0: Input passenger arrival matrix (par) and bus arrival matrix (bar). Set iteration starting time $t_i = 0$ and a time frame of 324 s. Set $n = 1$

Step 1: Select passengers having arrival time within the time frame. That is, if $t_i \leq \text{par} < t_i + 324$, a new matrix pb is created with these passenger arrivals,

Step 2: Waiting time of each passenger is calculated based on their arrival time and bus arrival time at each node.

$$W_g^s = \text{bar}_l^s - \text{par}_g^s \quad \forall g \in \text{pb} \tag{6}$$

where W_g^s is the waiting time of gth passenger at route section s, bar_l^s is the arrival time of line l at route section s, par_g^s is the arrival time of gth passenger at route section s.

Total waiting time of choice, ch

$$W_g^{\mathrm{ch}} = \sum_s W_g^s \quad \forall g \in \mathrm{pb} \tag{7}$$

Step 3: Information matrix formation based on the waiting time

$$\text{Informed travel time} = \text{In-vehicle travel time} + \text{waiting time}$$

$$\mathrm{Inf}_g^{\mathrm{ch}} = W_g^{\mathrm{ch}} + \delta + \sum_s c^s \quad \forall g \in \mathrm{pb} \tag{8}$$

where δ is the random term.

Step 4: (update) Update the cost of travel time through each route section based on the information and experience matrix

$$\mathrm{cost}_g^{\mathrm{ch}} = \theta \times \mathrm{e}_g^{ch} + (1 - \theta) \times \mathrm{Inf}_g^{\mathrm{ch}} \quad \forall g \ \in \ \mathrm{pb} \tag{9}$$

Step 5: (assign flow) Assign the passengers to the choice having minimum cost of travel.

Volume of passengers in the time frame t,

$$v_s^{\mathrm{ti}} = \sum_g v_s^t \, , \forall g \ \in \ \mathrm{pb} \tag{10}$$

Step 6: Congestion cost can be found as follows: capacity of route section,

$$K_s^t = \sum_l b_l \times b_a \quad \forall l \in s \tag{11}$$

where b_a is the capacity of buses in passenger per bus line. Here it is taken as 5 passenger/bus line in order to introduce the congestion effect.

$$\text{Additional waiting time} = 10 \times \frac{(V^s + V_s)}{(K_s^t)} \tag{12}$$

V^s is the number of passengers boarding the route section at the node,$\mathrm{V_s}$ is the number of passengers that compete with V V^{s}.

Step 7: (update total travel time) total travel time of passenger g,

$$tt_g^n = W_g^n + \text{Additional waiting time} + \sum_s c^s \tag{13}$$

where W_g^n is the total waiting time gth passenger in the selected choice.

Step 8: (update experience) the experience of passenger is updated based on the position of passenger

$$\text{pos}_g^{\text{ti}} = \text{t}_i + 324 \quad \forall g \ \in \ \text{pb} \tag{14}$$

$$e_g^n = \left\{ tt_g^n \text{ if pos}_g^{\text{ti}} \geq tt_g^n \right\} \quad \forall \text{g} \ \in \ \text{pb} \tag{15}$$

$$\text{If pos}_g^{\text{ti}} \geq tt_g^n, \quad \text{pos}_g^{\text{ti}} = tt_g^n \quad \forall g \ \in \ \text{pb} \tag{16}$$

Step 9: (stopping criteria) If $\text{pos}_g^n = \text{tt}_g^n \ \forall g \in \text{par}$, set $n = n + 1$ and go to step 0, otherwise set $= t_i + 324$ and go to step 1. Stop when $n = N$, where N is the number of days for which iteration have to be done.

3.6.2 Sensitivity Analysis

Differing Demand

To test the sensitivity of route flow to varying demand, the model is run for four values of demand and the results are shown in Fig. 5.

Similar to DUE model as flow increases, route section flows also increases. Here travellers choose all the four choices when the demand is more than 200 and only choice 1 is left idle for lower demand values.

Differing θ values

In day-to-day model as the arrival time of passengers and that of buses vary in each day, the waiting time of passengers also varies. In DUE, average waiting time is taken but in day-to-day model waiting time will vary from passenger to passenger (Fig. 6).

In DUE model, only one route (having minimum informed travel time) has flow, whereas in day-to-day model, all the route sections have flow (Fig. 7).

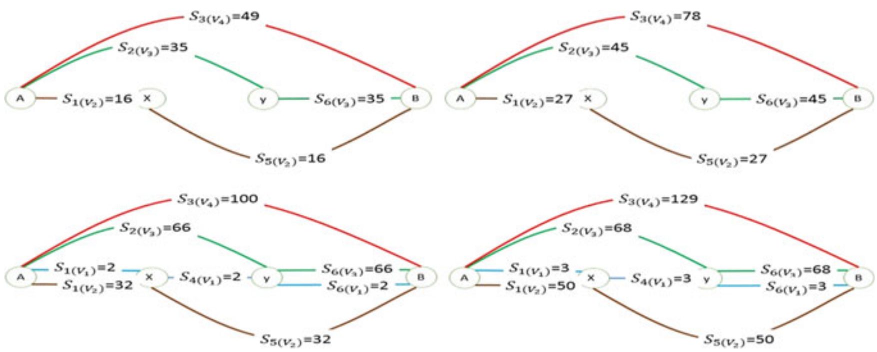


Fig. 5 Day-to-day average route section flow for demand 100, 150, 200, 250

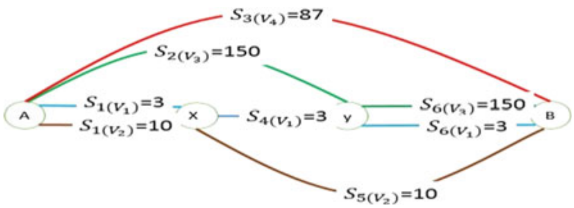


Fig. 6 Day-to-day average route section flows for $\theta = 0$

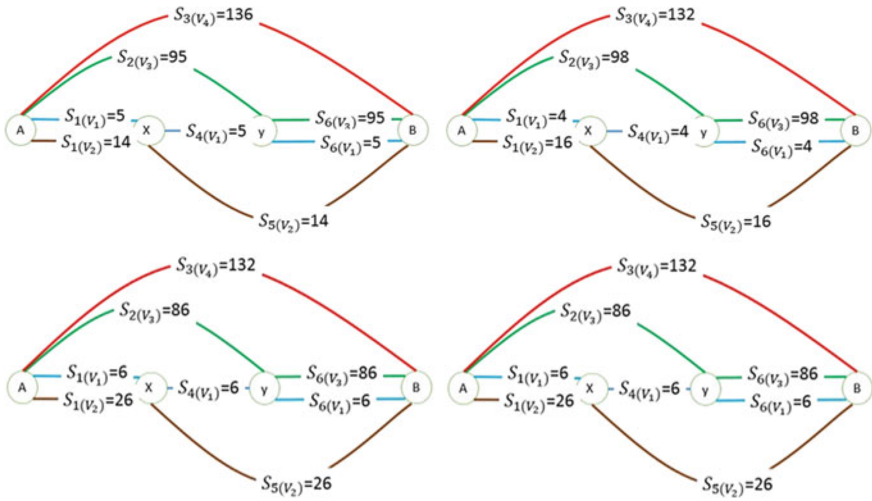


Fig. 7 Day-to-day average route section flows for $\theta = 0.25, 0.5, 0.75, 1.0$

For θ less than 0.25, more passengers choose choice 3 (*A*-*y*-*B*) compared to choice 4 (*A*-*B*) as choice 3 is the route having minimum informed travel time. The flow values of choice 1, choice 2 and choice 3 increase with increase in the value of θ, whereas flow in choice 3 decreases.

3.6.3 Convergence Test

A burning period of 20 is given for the model, and simulation runs are carried out for different values for factor of information. In day-to-day model, each route has a flow, but the average flow pattern resembles with the flow pattern from DUE. Simulation is run for different values of factor of information, and the results are compared with DUE model flows.

An example for flow is given in Fig. 8.

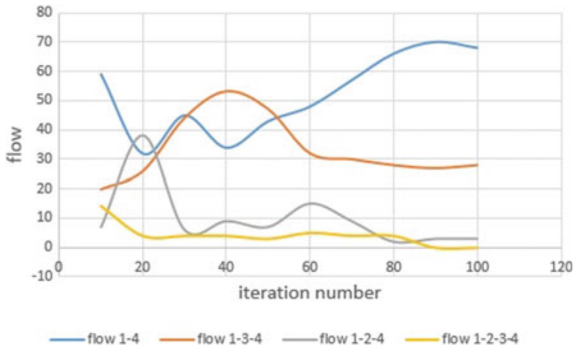


Fig. 8 Flow through route sections in day-to-day model for demand 100 and $\theta = 0$

Fig. 9 Image of study area

4 Data Collection

Calibrated θ value is required for the proposed methodology. A field survey was conducted along with an experiment in simulator to get the importance of information. Revealed preference survey is carried out in selected participants. The details of the survey are given in this section.

4.1 Study Area Characteristics

Delhi Transport Corporation (DTC) is the main public transport operator of Delhi, and it is one of the largest CNG-powered bus service operators in the world. DTC services in Delhi have vastly distributed network of buses and operates on many routes in Delhi and neighbouring states. The study area is bounded by 4 bus stops, namely Ashram Chowk, Badarpur Border, Mehrauli and AIIMS. Five *O-D* pairs were selected within this study area by conducting pilot survey in such a way that each *O-D* pair has two transit services serving. Home-to-workplace trips were considered mainly; hence, such *O-D* pairs were chosen. Trial runs were carried out in *O-D* pairs to know the travel time and travel time variance of transit lines serving each *O-D* pair. The selected *O-D* pairs and their details are given in Fig. 10 and Table 3, respectively (Fig. 9).

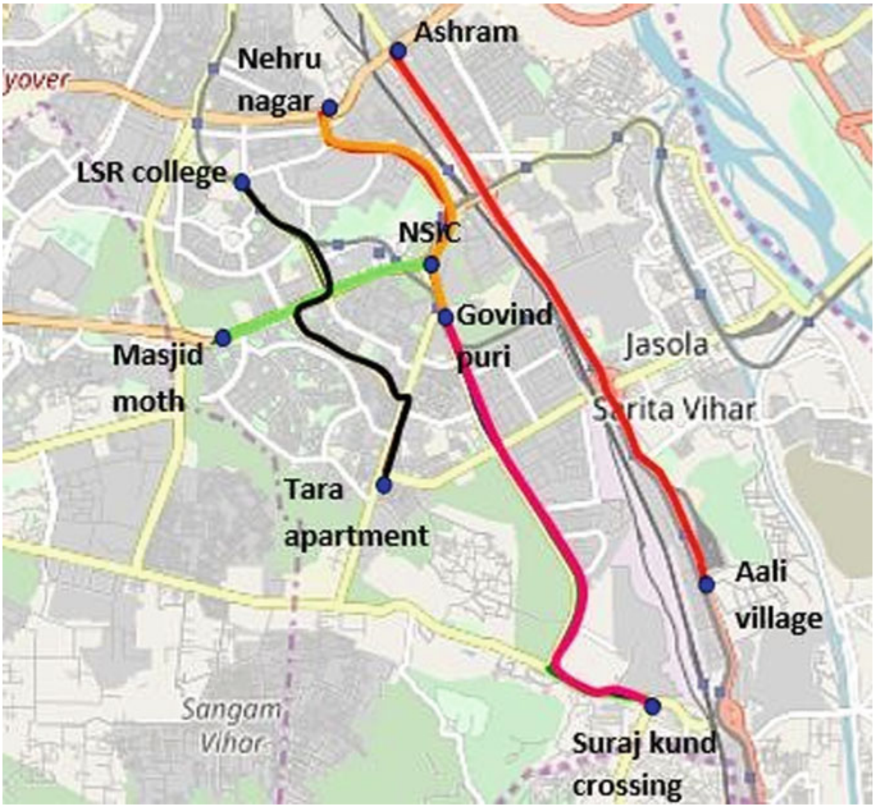


Fig. 10 Image of *O-D* pairs selected for the survey

Table 3 Details of O-D pairs and their travel times

Location	Bus lines	Mean (min)	Variance	Standard deviation	Travel time (min)
Nehru Nagar to Govind Puri Metro	429	16	12	3.46	16 ± 3.46
	469	14.3	1.34	1.15	14.3 ± 1.15
Ashram to Aali Village	405	35	109	10.44	35 ± 10.44
	479	32	40	6.32	32 ± 6.32
NSIC to Masjid Moth	427	11.5	12.5	3.53	11.5 ± 3.5
	534A	12	28	5.29	12 ± 5.29
Surajkund Crossing to Govind Puri Metro	433	21	4	2	21 ± 2
	511A	23.67	32.3	5.68	23.67 ± 5.68
Tara Apartments to Lady Shri Ram College	425	37.67	46.24	6.8	37.67 ± 6.8
	445A	35.67	0.64	0.8	35.67 ± 0.8

4.2 Participants

As most of the commuters in Delhi are well aware of available routes for their trip and corresponding travel time, their responses won't give much information about the effect of information provided. Hence, commuters from some different places were considered for this survey. Fifty-one participants were divided into five groups so that each *O-D* pair consists of 10 participants and again divided randomly between two groups: informed passengers and non-informed passengers. Travel time in each transit service is given to the informed passengers, whereas no travel time information is given to the non-informed passengers. The participant details about their socio-economic characteristics were collected along with their daily trip details, and they were requested to do their trial runs in the field. Each participant has to choose any transit service for their trip for each trial and have to travel from origin to destination. A minimum of 10 trial runs is made by the participants in the morning peak period (8.00 am to 12.00 pm) and in the evening peak period (4.00 pm to 8.00 pm). After each trial, their perceived travel time and waiting time were collected. In order to get the congestion level in the network, the number of buses in which the participants were not able to board before each trial was also collected.

4.3 Laboratory Experiment

Zen driving simulator is used for the laboratory experiment. Normally, this simulator is used to find out drivers behaviour at different conditions. Here it was used to find out the route choice behaviour of public transit passengers. To relate with the transit service, the speed of simulator was controlled to be at 20 km/h and the vehicle is stopped at the bus stops on the way for approximately 5 s. Simulator was run several times to induce different situations to get the travel time variability and to get travel time in each routes. Each participant in the survey was seated near the car body where he was able to see all the traffic scenarios in the projected screen similar to the real trip in bus. Before starting the survey, an imaginary scenario was described that he/she has to make a trip from his/her home to workplace, for which they have two routes and buses serving. Here also the participants were divided into two groups: informed participants and participants without information. Each time the participant is asked to choose a route, and simulator is run through that route. At the end of the trial, perceived travel time of participant is noted down. For each participant, 4 trial runs are made.

5 Data Analysis

5.1 Preliminary Analysis

The data collected is analysed using the statistical software SPSS, the Statistical Package for the Social Sciences. By analysing the personal data, 69% of participants were male participants, whereas in the survey conducted among daily commuters 51% were male participants. 57% of participants have education only in matriculate level, 33% of them are illiterate and only 10% are graduates. 44% of daily commuters are illiterate, and 30% have matriculate level of education which shows that in educational qualification of transit riders are low. The information using behaviour of both participants and daily commuters is given in Figs. 11, 12, 13, 14, 15, 16, 17, respectively.

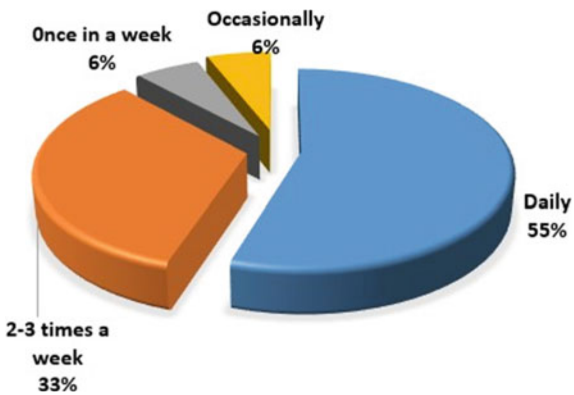


Fig. 11 Frequency of using bus (participants)

Fig. 12 Use of information (participants)

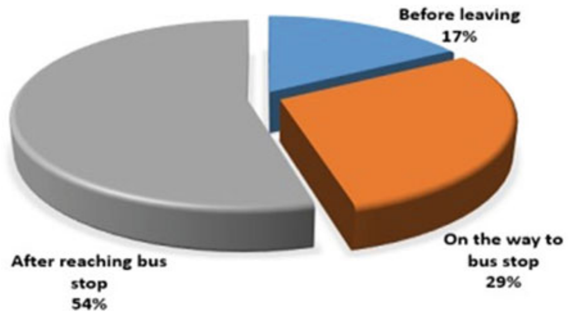


Fig. 13 Reason for not using information (participants)

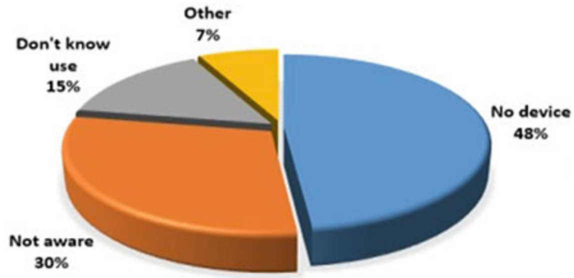


Fig. 14 Preferred type of information (Participants)

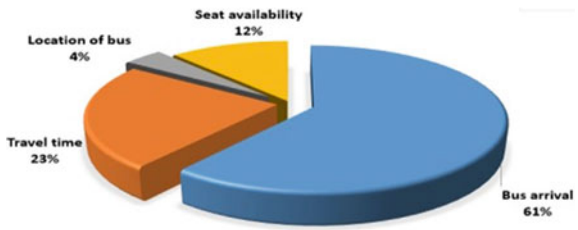
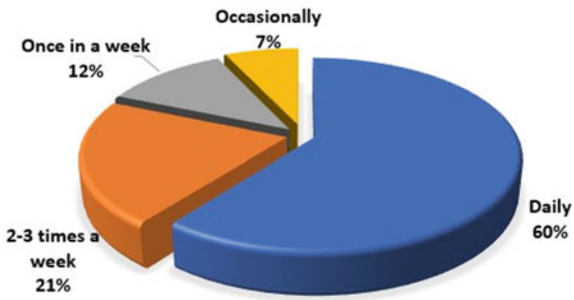


Fig. 15 Frequency of using bus (Daily commuters)



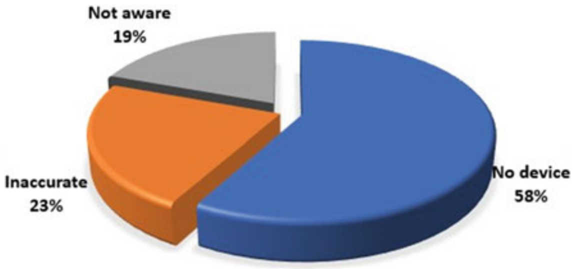


Fig. 16 Reason for not using information (daily commuters)

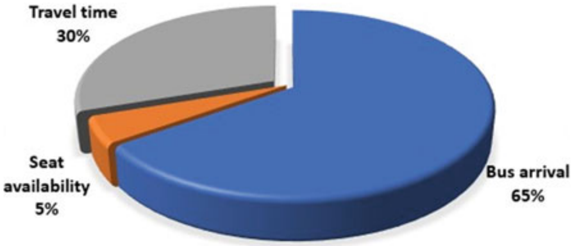


Fig. 17 Preferred type of information (daily Commuters)

5.2 Regression Analysis of Field Data

Binary logistic regression models were prepared for both informed and non-informed participants for simulator as well as field scenarios. For informed participants, if they choose the minimum travel time transit service in information, 1 is given. Otherwise 0 is given. The data is again segregated into two, participants of age group 18-30 and participants of age group 30–50. The probability of choosing informed path can be obtained from the following equations.

$$Logit(P) = -0.192 * G - 0.412 * I + 0.951 * E \tag{17}$$

$$Logit(P) = 0.134 * G + 0.405 * I - 0.539 * E \tag{18}$$

Equation (17) can be used for age group 18–30 and (18) can be used for age group 30–50. Equation (17) will predict the probability of choosing informed path with an accuracy of 48.3% and the other route with 76.1% accuracy. The overall accuracy of prediction is 61.4%. Equation (18) will predict the probability of choosing informed line with 70% of accuracy but for those who are choosing against the information prediction accuracy is very low, 24%. The model has an overall prediction accuracy of 49.1%.

The perception variance is also modelled using multi-linear regression analysis for both informed and without information scenario in the field. The variation in perceived travel time is modelled as follows.

$$\text{Perception variance} = 2.82 * A + 1.6 * G - 2.22 * E + 1 * I \tag{19}$$

$$\text{Perception variance} = 1.72 * A + 2.77 * G - 0.54 * E + 0.72 * I \tag{20}$$

Equation (19) represents the perception variance of informed participants and (20) represents that of non-informed participants. Both of the equations imply that people with higher educational qualification have low perception variance. Female passengers have more perception variance compared to male passengers, especially in without information scenario.

5.3 Regression Analysis of Data Collected from Experiment in Simulator

Similar to the field survey, the data collected from the experiment conducted using simulator is also divided into two: participants with information and participants without information. For passengers without information, Eq. (21) can be used to find out perception of travel time.

$$\text{Perception} = 6.07 * A + 0.76 * G - 3.73 * E + 2.72 * I \tag{21}$$

For informed participants, the perception is influenced by the information provided and updated by experience. Equation (22) represents the effect of experience and information on perception.

$$\text{Perception} = 0.13 \times \text{experience} + 0.51 \times \text{information} \tag{22}$$

The equation implies that information has more effect on perception than experience, provided the information is reliable. Therefore, information provided in the route section will influence the route choice and will have a major impact on network flows.

5.4 Effect of θ on Route Choice

By giving various values of θ both the models are run to get the effect of information. When θ is equal to 1, there is no effect of information; that is, passengers make their choice only based on the experience. Passengers do not consider their experience on the path when θ is zero and make their choice based on information.

Value of θ varies between 0.13 and 0.49. 5 values are considered to find the variation of flow, 0.13, 0.23, 0.33, 0.43 and 0.49. Flow in each route section is given in the following figures for both DUE and day-to-day models. DUE model gives same flow for both the values of θ 0.33 and 0.43

From the table, as θ varies the flow through routes also vary. If $\theta \leq 0.23$ ($(1-\theta) = 0.77$) passengers choose the optimum path having minimum free-flow travel time and if $\theta > 0.23$ the effect of congestion is also considered.

5.5 Comparison of DUE and Day-to-Day Models

The DUE models are traffic assignment models in which equilibrium flow values through each route section are obtained. In day-to-day model, traffic assignment is done each day and the evolution of equilibrium flow is observed. The dynamic nature of traffic and flow variations can be clearly observed in day-to-day models. A burning period (in day-to-day models, a number of simulation run values in the starting are excluded because it is believed that a number of runs are needed to the network to adapt with the given criteria) of 20 is taken in day-to-day model. Hence for comparison purpose, average of flow values in 100 days excluding the burning period is taken as day-to-day flow values.

From Figs. 18 and 19 by comparing the flow values of day-to-day and DUE models, it is clear that the flow pattern in both the models are same. In DUE models, the traffic is assigned to the paths having minimum travel time and all other paths do not have any flow. In other words, all the used paths have minimum travel time and all the paths having higher travel time is left unused. But in day-to-day models, people shift their choice from one route to other until the equilibrium is attained. It is also observed that there does not exist an equilibrium condition in day-to-day models. Whenever a passenger feel that the current choice has more travel time as compared to other paths, they change their choice. Still as the flow patterns are the same, equilibrium sits within the framework of day-to-day dynamic analysis.

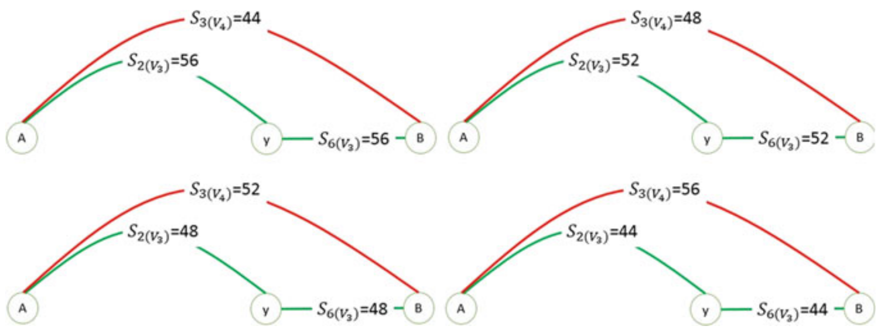


Fig. 18 Variation of flow in DUE model with $\theta = 0.13, 0.23. 0.33, 0.49$

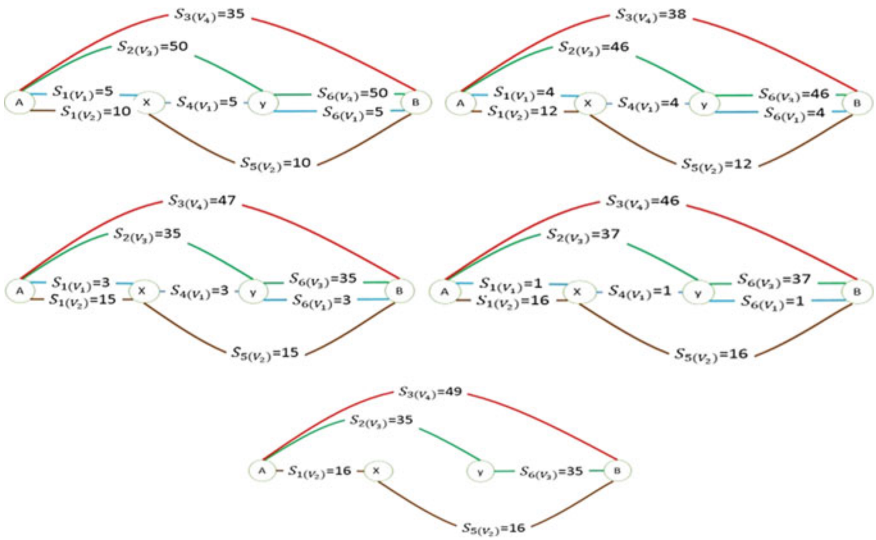


Fig. 19 Variation of flow in DUE model with θ 0.13, 0.23. 0.33, 0.43, 0.49

6 Conclusions

From the field survey, less than 50% of the passengers did not use information for their travel purpose and major reason for not using is that they do not have suitable device and inaccuracy of information. 60% of passengers feel that information about bus arrival time is very essential. This shows the importance of reliable information in route choice. A DUE transit assignment model and a day-to-day model are coded in MATLAB for an example network. The day-to-day model represents the dynamic world condition however the information in the current model was not assumed to be dynamic and did not vary with the time step. The models are sensitive to difference in demand and θ values. The calibrated value of θ differs between 0.13 and 0.49. If $\theta \leq 0.23$ passengers choose the path having minimum free-flow travel time and if $\theta > 0.23$, the effect of congestion is also considered in route choice. By comparing DUE model and day-to-day model, both the models have same flow pattern but day-to-day model has no equilibrium value and the flow values do not converge.

Regression analysis results show that educated people tend to use informed path more and have less perception variance. As age increases perception, variance also increases and female passengers have more perception error as compared to male passengers. When comparing informed passengers and passengers without information, illiterate passengers of age group 30–50 have high perception error if they are informed. For all other groups of passengers, passenger's perception error decreases with provision of information.

In this study, only the route choice of passengers is assumed to be dynamic. The demand is taken as static. But in reality, the demand is also dynamic which is not accounted for in the model. The effect of real-time information is also not considered which is a scope for future work. The memory length of the participants is taken as one, which means that travellers react according to the recent temporal experience. The memory length and learning process of participants can be incorporated in future studies.

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Alternate Vehicle Usage Controlling Policies and Their Effect on Vehicular Pollution—Case Study of Delhi

Shivani Verma, Ravindra Kumar, and N. P. Melkania

Abstract Vehicular pollution is mainly responsible for the air pollution in metropolitan cities such as Delhi which has been nominated as most polluted city of the world. This paper is based on the study of prediction of reduction in emission of different air pollutants mainly carbon monoxide (CO), volatile organic compound (VOC), oxides of nitrogen (NO_x), oxides of sulfur (SO_x), and particulate matter (PM) under different scenarios in Delhi during 2017–2028. The effectiveness of various scenarios which are Business-As-Usual (BAU) existing conditions in current environment and three other reduction scenarios, namely odd–even policy (OEP), electronic vehicle penetration (EVP), and integrated scenario (IS), is evaluated in order to emission reduction over two intervals which are short-span (2023) and long-span (2028) by International Vehicular Emissions Model up to 2028. It is found that, there are huge increment observed in the Delhi's vehicle population, and it would reach about 40% up to 2028 in comparison with 2017. By comparing all scenarios, it is found that EVP scenario is more effective in case of CO. CO would reduced ~27.47% up to 2023 and ~41.81% up to 2028 while OEP scenario can reduce emissions more effectively for VOC, NO_x, SO_x, and PM and would reach to 26.51 5, 27.7%, 35.63%, and 30.29% in short-span while 24.54%, 29.23%, 35.98%, and 29.78% in long-span as compared to BAU scenario. Overall OEP scenario is performed best for other criteria pollutant except CO compared to other controlling scenarios.

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Keywords Air pollution · Vehicular emission forecasting · International vehicle emissions model (IVE) · Driving characteristics · Vehicle kilometer traveled (VKT) · Emission factor

1 Introduction

Our atmosphere is surrounded by a thick atmospheric layer which is known as air. It is a gaseous mixture containing nitrogen ~78%, carbon ~0.04%, oxygen ~21%, and argon ~0.9%, and remaining are other trace gases including helium, hydrogen, etc. and water vapor which is vary from location to location in the range of 1–5%.

Air pollution is the contamination of air or in a broad way, we can say that addition of unwanted and undesirable substances which have an adverse effect on human, plant and animals as well as on the environmental properties termed as air pollution. The substances which are responsible for the air pollution are known as air pollutants, and these common air pollutants are oxides of nitrogen (NO_x), oxides of sulfur (SO_x), particulate matter ($PM_{2.5}$ and PM_{10}) both volatile organic compounds (VOCs), carbon dioxide (CO_2), carbon monoxide (CO), ozone (O_3), etc. The cause of air pollution can be natural and anthropogenic as well. The natural air pollution sources are volcanic eruption, natural forest fire, thunderstorm, atmospheric chemicals reactions (commonly secondary aerosols), etc., and anthropogenic sources are thermal power plants (majorly), mining mills, vehicles, various chemicals industries, biomass burning, fossils fuel burning, etc.

Vehicles are the major contributors to the air pollution; more the vehicle population leads more the air pollution, because they produce a significant amount of nitrogen oxides, hydrocarbons, volatile organic compounds, and other air pollutants. As we know that India is a developing country, which means more industrialization, urbanization happens which lead to the more transportation in developing megacities. Delhi surrounded by the Uttar Pradesh and Haryana. Delhi is a well-known commercial, social, cultural, and political center in India. This is the one reason of vehicular emission in Delhi, and as we know that it touches the boundaries of U P and Haryana; therefore, commercial vehicles come in and out from the city, which also contributes into the vehicular emission in Delhi. As per the Delhi Statistical Handbook, the number of registered vehicles crosses one crore up to 2017 in Delhi (Delhi Statistical Hand Book 2017). Emission from this huge numbers of vehicles named Delhi among the most polluted city in the world. These air pollutants cause various health issues in human such as heart diseases, breathing problems, allergic reactions, eye and throat irritation, cancer and plants as well and cause chlorosis (loss of green color in plants), necrosis, etc., and they affect biotic components as well as monuments by erosion. It has been observed that there are multiple sources are present which are responsible for the pollutant emission, namely vehicles, power plants, industries, road dust, area sources (includes wood burning or kerosene burning for the cooking purpose, cause air pollution area-wise), etc. Therefore, it must introduce such policies by which we control the vehicular pollution emission.

1.1 Need for the Study

In the current environment urbanization, industrialization, and at all time increasing vehicular emission with the various anthropogenic activities such as biomass domestic wood burning, combustion of fossils fuels produces gaseous pollutants and particulate matter both. The short-term effect of PM on human such as lung inflammatory reactions, adverse effects on cardiovascular systems while long-term exposure causes lung dysfunction, reduce life, etc. The other smaller pollutants such as $PM_{2.5}$ can penetrate deep into the lungs and cause respiratory diseases, while the gaseous pollutants such as NO_x, SO_x, HCs and CO have different health issues.

It is estimated that about half of the NO_x and CO coming from the vehicular emissions with the small fraction of HCs. It is well known that Delhi is named as most polluted city of the world by WHO, and major contribution to this pollution is vehicular emission which is accountable for the 10,000–30,000 annual deaths (https://www.firstpost.com/india/air-pollution-causes-30000-deaths-annually-in-delhi-fifth-leading-cause-of-death-in-india-2547). This much air pollution causes about 6, 20,000 premature deaths by heart stroke, lung cancer, heart failure, pulmonary diseases.

So, how can we cope with this dreadful situation? For this, we should take some steps by introducing some control measures to reduce vehicular emission. Therefore, a comprehensive study has been proposed to select a best-suited scenario (policy) in order to reduce vehicular emission in future in Delhi.

1.2 The Objective

The objective of our study is to find that scenario which reduced more the emission of pollutants which are carbon monoxide (CO), volatile organic compound (VOC), oxides of nitrogen (NO_x), oxides of sulfur (SO_x), and particulate matter (PM) in Delhi.

2 Literature Review

There are several studies which have been done for the analysis of different control policies in order to reduce vehicular emission, and some of them are mentioned below. All the authors conclude that the major source of air pollution is vehicular emission. Many authors considered some common air pollutants which are nitrogen oxides (NO_x), carbon monoxide (CO), volatile organic compounds (VOCs), particulate matter (PM), etc., and it was found that half of the NO_x and CO coming from the vehicular emissions with the small fraction of HCs. The development of effective controlling policies for the eradication, reduction and look-out of atmospheric pollution is extremely important at local and national level.

Guo et al. (2016) created five controlling scenarios with base year scenario, namely these are higher emission vehicles elimination (HVE), new energy vehicles promotion (NEP), emission standards updating (ESU), LDV population regulation (LPR), and integrated scenario (IS) and studied the reduction in vehicular emission under these scenarios as compared to BAU as base year scenario in Beijing-Tianjin-Hebei (BTH) region during 2011–2020 (Guo et al. 2016). While Xing et al. (2011) investigated in his study on emission of different air pollutants in future from anthropogenic sources in China. The considered controlling scenarios in this study are Policy (1) (reclamation of energy competence with better execution of environmental provision), Policy (2) (reclamation of energy competence with uncompromising environmental provision), Policy (0) (reclamation of energy competence with existing environmental provision), and REF [0] (existing control provisions and execution status) from 2005 to 2020 (Xing et al. 2011), and Zhang et al. (2014) investigated the trend of vehicular emission from on-road mobile sources of past in Beijing, China and evaluate best-suited vehicular emission reduction policies until 2020 under Clean Air Action Plan of China (Zhang et al. 2014).

Jain et al. (2016) considered 2007 as a base year scenario (BAU) and 2021 as year of impact analysis and evaluate the impact of integrated mass rapid transit system (IMRTS) and other polices on the vehicular emission in Delhi from 2007 to 2021 and created three substitute scenarios for the commercial vehicles, namely ALT-1, 2, and 3 for introduction of BS-V norms, full phase of IMRTS implementation (bus rapid transit system and 4 phase of Delhi Metro) and ALT-2 with increasing the parking fee from 10–20 Rs. to 60–200 Rs., respectively, and two alternative scenarios for the good vehicles which are introduction of BS-V norms and diesel particulate filter (DPF) in HDV's were added (Jain et al. 2016). Mahommadiha et al. (2018) consider 2013 as a base year and 2028 as year for impact analysis and researches the pattern of emission of air pollutants from vehicles in Tehran and evaluate the impact of different scenarios on pollutants emission from traffic division. They generated eight scenarios one of these business-as-usual (BAU), elimination of carburetor equipped vehicles (ECU), new and energy motorcycles (NEM), high emission standards (HES), vehicle catalyst replacement (VCR), fuel quality enhancement (FQE), diesel particulate filter (DPF), total scenarios aggregation (TSA) (Mahommadiha et al. 2018). Guttikunda et al., evaluated on-road vehicle exhaust emissions for a 40-year horizon (1990–2030), and they found that the most reduction in emissions between 1998 (Goel et al. 2015).

3 Study Area

Delhi is the capital of our country India. The geographical expansion of Delhi is 1483 square kilometer of which 783 km is referred as rural area and 700 km as urban area (Goyal et al. 2013) along with latitudinal expansion of 28 ° 24′ 17″

and 28° 53′ 00″ North and longitudinal expansion of 76° 50′ 24″ and 77° 20′-37″ East. Delhi surrounded by the Uttar Pradesh and Haryana. Delhi is a well-known commercial, social, cultural, and political center in India. This is the one reason of vehicular emission in Delhi and as we know that it touches the boundaries of U P and Haryana; therefore, commercial vehicles come in and out from the city, which also contributes into the vehicular emission in Delhi. The climate of Delhi is extremely dry with hot summer and temperature rises up to 45° C. The rainfall pattern in Delhi is 715 mm, of which 3/4 portions falls in July to September. According to the census 2011, the population has an increase of 21% from 13.9 to about 16.76 million. More the human population leads to the increase in vehicle population. The geographical expansion of Delhi is shown in Fig. 1.

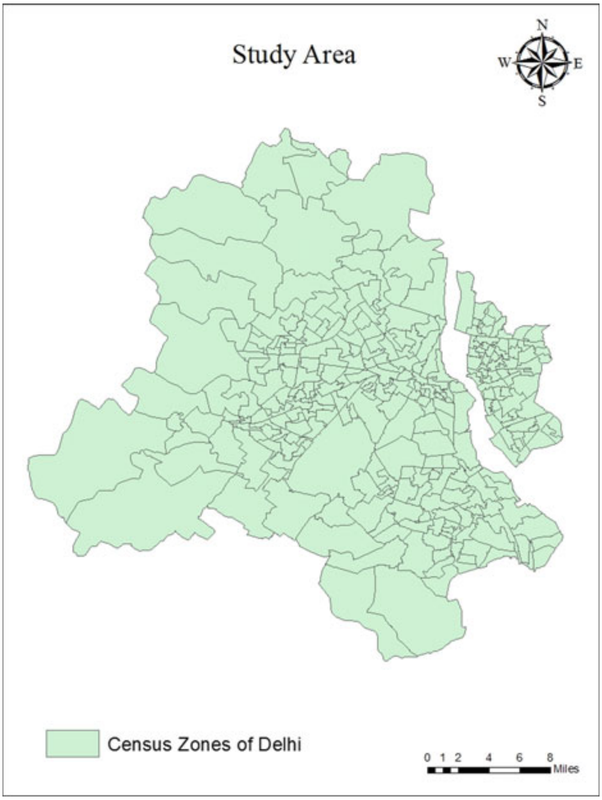

Fig. 1 Map of study region (Delhi)

4 Research Techniques

In this paper, we estimated the emission reduction of criteria pollutants under different control scenarios using IVE model for scenarios effectiveness analysis in Delhi region during 2017–2028. The pollutants include CO, VOC, NO_x, SO_x, and PM. Various steps have been taken for the present study, and these are shown in Fig. 2.

4.1 Vehicle Population (VP)

It is found that the growth of vehicle population is dominantly affected by economic development of that region. Figure 3 shows the total number of registered vehicles in Delhi which shows that the vehicle population is rapidly growing from last decade, and it was nearly 5,185,410 as observed in 2007 to about 104,53,067 in 2017 (Delhi

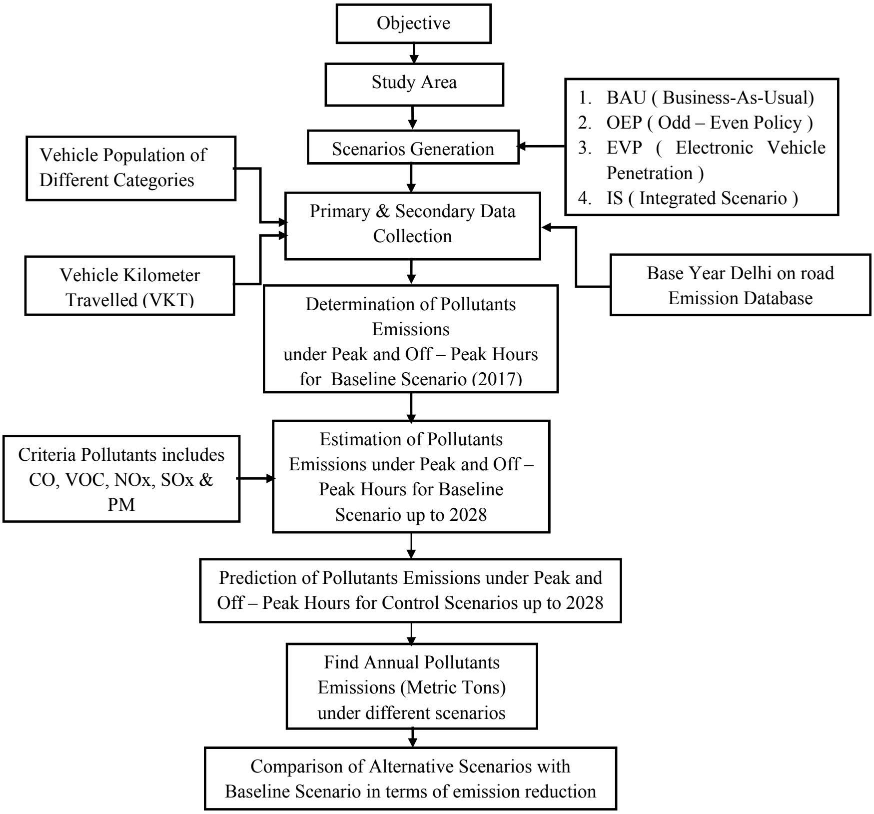

Fig. 2 Methodology adopted in this study

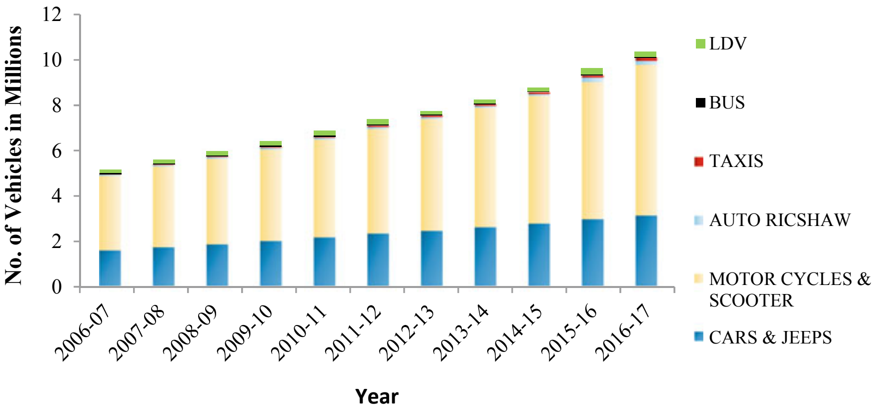

Fig. 3 Trend of growth of vehicles population of different categories and their distribution during 2006–2017 time periods in Delhi (Delhi Statistical Hand Book 2017)

Statistical Hand Book 2017), in which motorcycles and scooters with 64.17% have maximum number among all other vehicles categories, while car and jeeps, auto rickshaw, taxis, buses and light-duty vehicles (LDVs) have 30.16%, 1.66%, 1.42%, 0.36%, and 2.21%, respectively, in total number of registered vehicles in Delhi. To calculate vehicular emission by total vehicle population, we should know the fraction of vehicles who meet with specific emission standards (Bharat Stage I–Stage IV); therefore, they can match with the emission factor of vehicles, because emission factor (EF) value is affected by the level of emission standards with which vehicle meet.

4.2 Vehicle Kilometer Traveled (VKT)

Vehicular emission is directly proportional to the distance traveled by vehicle. There are various secondary factor which affect the emission of that vehicle, and these are age of the vehicle, fuel type, engine technology, emission standard, etc.; therefore, vehicle kilometer traveled is one of the most important parameters in terms of vehicular emission as compared to the other variables such as speed of vehicles, road grade, and traffic.

4.3 Base Year Emission of Delhi

In this paper, we consider 2017 as base year and 2018 as year for the estimation of reduction in vehicular emission. In base year, we study about the existing conditions in current environment which include total number of vehicles present in 2017, fuel

Table 1 Emission of pollutants from different sources in Delhi

Source	Emission rate of pollutants (kg/day)				
	PM_{10}	SO_x	NO_x	CO	HC
Industrial	32,479	264,399	360,526	23,771	4765
Area source	27,730	2608	15,332	217,800	66,700
Vehicular emission	9750	720	84,200	217,800	66,700
Road dust emission	77,275	–	–	–	–
Total	147,234	267,727	460,058	374,123	131,433

Source Air Quality Monitoring Emission Inventory & Source Apportionment Studies for Delhi, 2008

standard in which they run, emission standards with which they meet, and existing status about emission rate of different pollutants which are emitted from different sources such as industries, area sources, vehicles, and road dust. An emission inventory is the dataset which include the list of all resources that possibly contribute into air pollutant emission. An emission inventory has been prepared by Air Quality Monitoring Emission Inventory & Source Apportionment Studies for Delhi were developed with NEERI in 2008 with five major pollutants including PM_{10}, SO_2, NO_x, CO, and HC (hydrocarbons). According to this dataset, the pollutant emission from different sources is shown in Table 1 (NEERI 2008).

As per Delhi Statistical Handbook 2017, six main categories were taken for Delhi vehicles including cars and jeeps, motorcycles and scooters, bus, taxis, auto rickshaw and light-duty vehicles (LDV). In all of the above, passenger cars contribute largely in vehicle population (64.17%) and VKT (42.69%). CO and NO_x largely emitted by cars (47.16%) and 39.07%, respectively, HC and PM by two-wheelers 60.4% and 38.56%, respectively.

4.4 *Emission Factor by International Vehicle Emissions (IVE) Model*

International Vehicular Emissions Model is used for the assessment of emission of pollutants from vehicles. International Sustainable Systems Research Center (ISSRC) and University of California at Riverside (URC) developed this model and US Environmental Protection Agency (EPA) funded this for the estimation of emission of pollutants by mobile sources such as vehicles. The IVE is considered as an important tool for the creation of an emission inventory of vehicles and for the prediction of pollutant emission over a period of time. It is an efficient tool because it parameters related to vehicles as well as environmental parameters such as humidity and temperature.

IVE consists of three main tab/window to create emission inventories (ISSRC 2008).

1. Location Tab: Information regarding region which is affected, e.g., Delhi.
2. Fleet Tab: Information regarding fleet technology such as engine technology, emission standards, age of the vehicles.
3. Correction Tab: It contains correction factor for each parameter.

The input data is given by two input files containing all the information required to run the model. The two input files are the location file which includes specific information about the location and the vehicle activity, and the fleet files including data about the vehicle fleet characterization.

4.4.1 Location Input File and Vehicle Activity Study

This file plays an important role in the estimation of vehicular emission by including (1) the vehicle kilometer traveled and driving manner, and (2) start pattern means cold start or hot start, soak pattern performed by the vehicle. The real-world driving cycle data is used which is collected from V-box, global positioning system (GPS) technology for this region. For the estimation of vehicular emission, it is necessary to know information about the driving pattern, road grade, traffic conditions, and environmental conditions of that area. Once all these information collected, then we input these into location file of an IVE model; this contains areas for these information in location input file.

There is a term which is necessary for the calculation of vehicular emission known as vehicle-specific power (VSP) which includes driving pattern of the vehicle such as distance, time, altitude, revolution per minute (RPM), velocity and road grade. It contains 60 bins (ISSRC 2008). Equation 1 represents the equation which is used for the calculation of VSP in IVE model.

$$\mathrm{VSP} = V(1.1 \cdot a + 9.81 \sin(a \tan(\mathrm{grade})) + 0.132) + 0.000302(v + v \cdot w)^2 \cdot v \tag{1}$$

where

V	vehicle velocity (m/s)
a	acceleration (m/s^2)
Road grade	grade of road (rad).

If the value of road grade is very small (<10%), then sin(a tan(grade)) can be replaced by 1 or close to one. Then the above equation can be modified as below. Equation 2 represents the simplified form of above equation (ISSRC 2008).

$$\mathrm{VSP} = v \cdot (1.1 \cdot a + 9.81 \cdot \mathrm{grade} + 0.132) + 0.000302 \cdot v^3 \tag{2}$$

VSP is depending upon the engine stress. In all 60 bins, first two row of 20 bins for low stress, next 20 bins for medium stress, and rest 20 bins for high stress. To improve the vehicular emissions, a factor developed known as vehicle stress. To calculate vehicle stress, we must have vehicle RPM with the average of power in 15 s before the time of interest. The equation for said calculation is shown in Eq. 3 (ISSRC 2008).

$$\mathrm{STR} = \mathrm{RPM} + 0.08 * \mathrm{PreaveragePower} \tag{3}$$

where

RPM	Vehicle RPM stands for revolution per minute (RPM), and it is measured by using GPS. The minimum accepted value of RPM is 900.
PreaveragePower	It is average of power before the 15 s from the time of interest. In above equation, 0.08 is coefficient (ISSRC 2008).

4.4.2 Fleet Input File

A fleet file consists information of vehicles technology with which they travel of each category. There are number of categories of vehicles including buses, trucks, passenger cars, two-wheelers, auto-rickshaws, etc., operated on the roads, with the total number of vehicles, and it is important to know the fraction of each category in total population of vehicles with their technology, engine specification, and emission standards. In IVE model, fleet file consists a list of total 1371 technologies combined with the other 45 undefined technologies.

4.4.3 Base Emission Factor and Correction Factors

The International Vehicular Emissions Model also has correction factors adjustment factor for those countries that have their specific standards for emissions. It has correction factor for each parameter as shown in Eq. 4 such as K[humidity], K[temperature]. It is shown from Eq. 4 adjusted emission factor that is the product of base emission rate $B[t]$ and various correction factors $K[t]$, where $K[i]$ $[t]$; = 1, 2, 3, …, n) (Fig. 4).

$$Q[t] = B[t] * K[1][t] * K[2][t] \ldots * K[n][t] \tag{4}$$

4.5 Development of Reduction Scenarios

We create a base year (in this scenarios, we study about the existing conditions in study area) and three other controlling scenarios for the determination of most effective scenario in terms of pollutants emission reduction by comparing these controlling scenarios with the base year (as reference scenario). Here we considered 2017

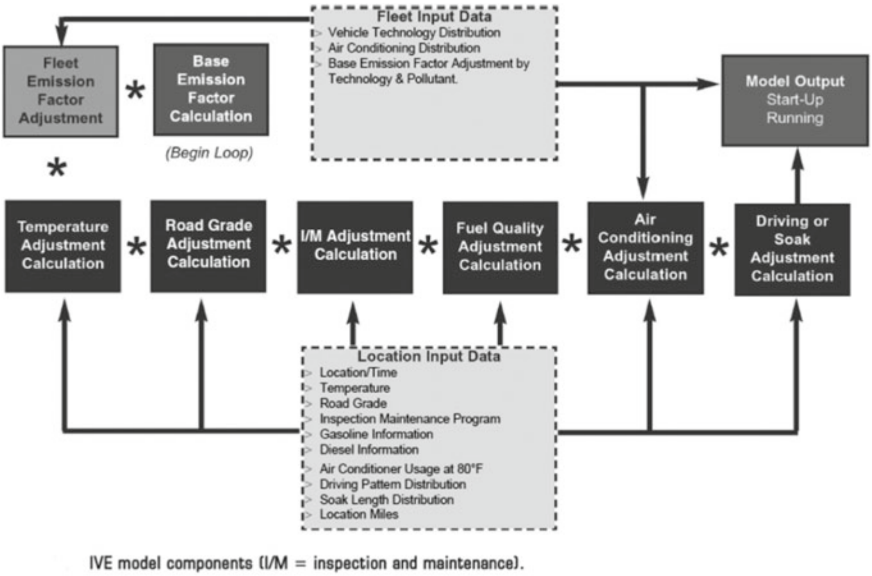

Fig. 4 IVE model core architecture (ISSRC 2008)

as base year and 2028 as the year of prediction. The name of these three controlling scenarios is odd–even policy (OEP), electronic vehicle penetration (EVP), and integrated scenario (IS). The detailed description about these scenarios is given below.

4.5.1 Business-As-Usual (BAU)

The BAU scenario means the existing conditions comprises existing number of vehicles with emission standards and policies. In base year, we calculate pollutants emission loads under existing emission standards and then we predict the possible reduction in vehicular emission under other controlling scenarios. It is considered as a reference scenario, with which we compare other controlling scenarios for the assessment of reduction in emission of pollutants.

4.5.2 Electronic Vehicle Penetration (EVP)

All EVs produce zero-direct emission because they have batteries instead of gasoline engines, but there is some fraction of in-between electric vehicles known as plug-in hybrid electric vehicles (PHEVs). They contain gasoline engines as well as electric motors, which is responsible for the evaporative emission along with the tail pipe emissions both. Since there is zero-direct emission form EVs except fraction of PHEVs, they have life cycle emission. Life cycle emission includes emission from extraction of raw material to the disposal/recycling or we can say that from life to death of products, and it includes each and every step such as extraction of raw material, processing, production, distribution, and at last recycling/disposal of the product life. It is estimated that 10% of lineal vehicle which includes total vehicle types such as three wheelers, passenger's cars, and two-wheelers will become electric vehicle in upcoming future which is a part of National Electric Mobility Mission Plan which helps in air pollution for some extent (Kashyap et al. 2015). Electrical vehicles will emit 30–40% lower carbon dioxide than conventional vehicles (https://auto.economictimes.indiatimes.com/news/passenger-vehicle/cars/electric-vehicles-inefficient-way-to-reduce-co2-emissions-study/592883). EVs produce no NO_2 directly as they do not have gasoline engines but they contribute in the production of NO_2 indirectly because they use electricity to charge their batteries.

4.5.3 Odd–Even Policy (OEP)

To overcome air pollution, Delhi's government had taken an policy known as odd–even policy: "In this method, those vehicles run on the roads which have their license plates ends with even number while on the next day those who have odd number, i.e., cars runs on the road on alternate basis one day is for even and one day is for odd." By this method, the vehicle population would become nearly half of the previous one (https://www.thehindubusinessline.com/news/science/new-study-says-odd-even-scheme-led-to-increase-in-emissions/article2341). But this policy limited to private cars only not rather than for all vehicle type. This method is also known as road space rationing. In Delhi, this policy has been introduced in three phases. The first phase runs from 1st to 15th January, 2016, and the result of this brief experiment was found that there was only a marginal drop observed in the level of particulate matter ($PM_{2.5}$) during this phase.

4.5.4 Integrated Scenario (IS)

Integrated scenario (IS) is shows the combined effect which includes odd–even policy (OEP) with electronic vehicle penetration (EVP) in order to estimate the possible reduction in vehicular emission under this scenario up to 2028 in Delhi. It sums up the properties of both OEP and EVP scenario.

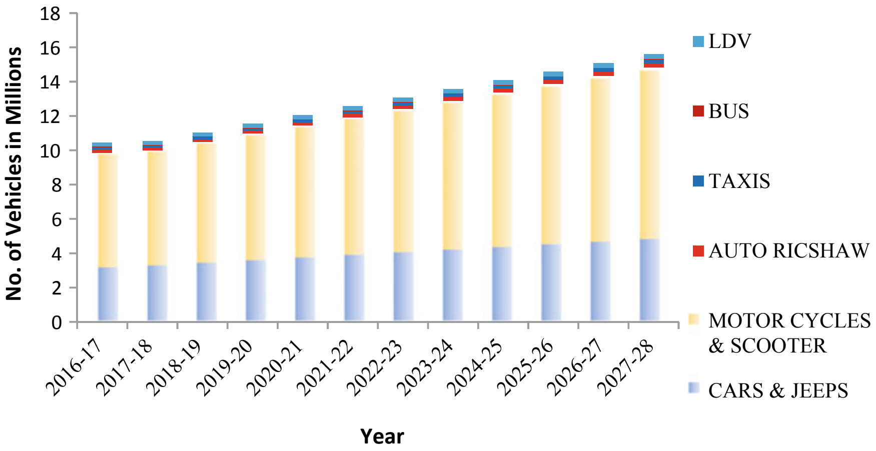

Fig. 5 Estimation of vehicle population of different categories and their distribution during 2017–2028 time periods in Delhi

5 Result and Discussion

5.1 Prediction of Vehicle Population (VP) and VKT

Trend of population growth of different vehicle types is shown in Fig. 5. This figure shows a rapid increase in vehicle population of different categories in Delhi. The total numbers of vehicles about 1 crore in 2017, and it is estimated that it would reached up to 2028. It is clear from this figure the maximum contribution in total vehicle population is of two-wheelers (motor cycles and scooters), and their population will increase from about 67 lakhs in 2017 to nearly 99 lakhs in 2028. The second highest contribution after two-wheelers is of PCs, about 31 lakhs in 2017 which will increase nearly 48 lakhs up to 2028. The rest of the vehicle type population will increase at moderate speed from 2017 to 2028. The average growth rate of vehicle population is 6% from 2017 to 2028. Total estimated VKT of all vehicle type is about 1000 millions km/day in base year, of which highest share of PCs with 40% of total VKT. While the VKT of MCs, taxis, 3Ws, buses, and LDVs are following 29%, 2%, 10%, 7%, and 12%, respectively (Goel et al. 2015).

5.2 Future Projection of Air Pollutant Emission Under Different Scenario in Delhi

The overall annual emission (metric tons) of air pollutants, namely carbon monoxide (CO), oxides of nitrogen (NO_x), volatile organic compounds (VOCs), oxides of sulfur

Table 2 Emission reduction in different scenarios

Scenario	Pollutant	Reduction in emission as compared to base year (BAU) (metric tons)			Emission reduction in (%)		
		2018	2023	2028	2018	2023	2028
OEP	CO	1.51	2.01	2.69	19.26	19.14	19.13
	VOC	0	0.00	0.00	0	26.51	24.54
	NO_x	0	0.25	0.33	0	27.7	29.23
	SO_x	0	0.00	0.01	35.16	35.63	35.98
	PM	0.01	0.02	0.03	30.44	30.29	29.78
EVP	CO	0.90	2.89	5.89	11.53	27.47	41.81
	VOC	0	0.00	0.00	0	3.45	2.15
	NO_x	0	0.07	0.16	0	9.24	13.91
	SO_x	0	0.00	0.00	4.31	11.83	18.16
	PM	0	0.00	0.00	2.32	5.38	6.85

(SO_x), and particulate matter (PM), is calculated under base year which is 2017 as BAU scenario for in Delhi is 6.585, 0.010, 0.501, 0.007, and 0.069 of CO, VOC, NO_x, SO_x, and PM, respectively. The overall annual emission (metric tons) of above-said air pollutants under different control scenarios for peak and off-peak hours separately for short-span and long-span during 2017–2028 time period in Delhi which are shown in Table 2.

It is clear from Table 2, vehicular emission of CO under BAU scenario would increases nearly 46% in 2023 (for short-span) and about 72% up to 2028 in comparison with 2017 base year. By comparison, it was observed that best-suited scenario in order of reduction in CO is electronic vehicle penetration (EVP) in short-span (2023) and long-span (2028) both cases. In EVP scenario emission, the reduction of CO would reach nearly 11.53% in 2018, 27.47% in 2023, and 41.81% in 2028 compared to BAU scenario. At the next level, odd–even policy (OEP) scenario is observed as the less effective scenario for vehicular emission of CO as compared to EVP. While the CO emission reduction in OEP scenario would reach to 19.13% in 2018, 19.14% in 2023, and 19.26% in 2028 compared to BAU scenario. At the next level, emission reduction of VOC, NO_x, SO_x, and PM would reach to nearly 26.51%, 27.7%, 35.63%, and 30.29% in 2023 (short-span) and 24.54%, 29.23%, 35.98%, and 29.78% in 2028 (long-span) in odd–even policy (OEP) scenario as compared to BAU scenario. While in EVP the emission reduction of VOC, NO_x, SO_x, and PM would reach to about 3.45%, 9.24%, 11.83%, and 5.38% in 2023 (short-span) and 2.15%, 13.91%, 18.16%, and 6.85% in 2028 (long-span), respectively, as compared to 2017 base year.

Table 3 Emission reduction in IS scenario

Scenario	Pollutant	Reduction in Emission as compared to base year (BAU) (metric tons)			Emission reduction in (%)		
		2018	2023	2028	2018	2023	2028
IS	CO	1.121	2.455	4.296	0.33	23.31	30.47
	VOC	0	0.002	0.002	0	14.98	13.34
	NO_x	0	0.165	0.248	0	19.24	13.91
	SO_x	0.003	0.005	0.008	19.74	23.73	27.07
	PM	0.010	0.015	0.025	16.38	17.83	18.31

5.3 Future Projection of Pollutant Emission Under IS Scenario

The impact of integrated scenario (IS) on reduction of vehicular emission as compared to base year (2017) is shown in Table 3. It is clear from this table, the reduction in CO emission is about 23.31% in 2023 for short-span and 30.47% in 2028 for long-span under IS scenario in comparison with base year (2017). While the reduction in emission of VOC, NO_x, SO_x, and PM would reach close to 14.98%, 19.24%, 23.73%, and 17.83%, respectively, in short-span period and 13.34%, 13.91%, 27.07%, and 18.31%, respectively, in long-span period. The estimation of emission reduction of different pollutant under integrated scenario can be seen in Table 3 (Fig. 6).

6 Limitations

The IVE model which has been used in this study is of version 2.0 May, 2008, which is not updated yet. This model does not contain latest vehicle technology indexes.

7 Conclusion

The predicted total annual emission of criteria pollutant from different vehicle types in the year 2017 which is base year are found to be 6.58, 0.010, 0.501, 0.007, and 0.069 of CO, VOC, NO_x, SO_x, and PM, respectively. After estimating the emission during 2017–2028 for base year, we found that CO gradually increases throughout the year; also, it is the dominant pollutant which is emitted through vehicles of all categories as compared to four other criteria pollutants The emission of CO in base year is 6.58 metric tons while it would increases nearly about 46.11% for short-span period which is 2023 and about 72.64% for long-span period which is 2028 in

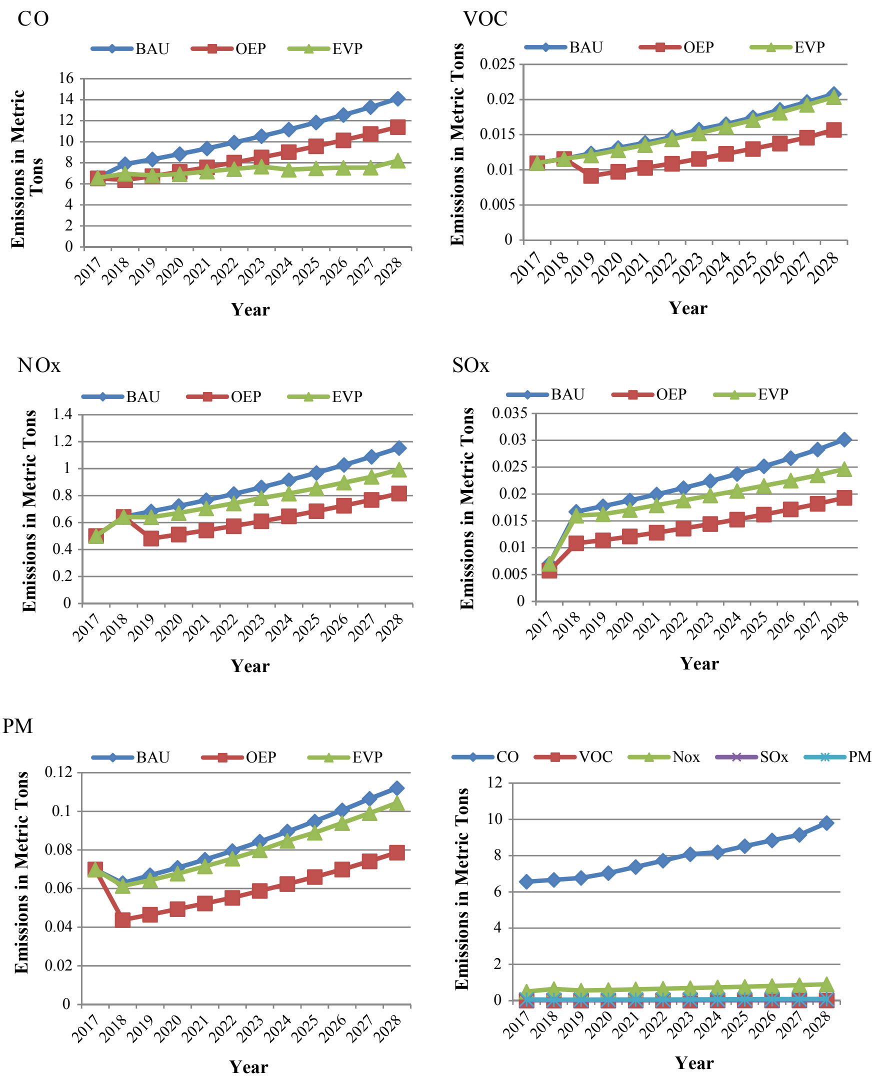

Fig. 6 **a** CO, **b** VOCs, **c** NO_x, **d** SO_x, and **e** PM; this figure shows the criteria air pollutants emission under different scenarios from 2017 to 2028 and **f** Emission reduction in percentage of CO, VOC, NO_x, SO_x, and PM under IS scenario

Delhi. At the next level, emission of other pollutants in base year is 0.010, 0.501, 0.007, and 0.069 (metric tons), and their emission would reach to about 0.015, 0.861, 0.022, and 0.084 of VOC, NO_x, SO_x, and PM, respectively, for short-span period and about 0.020, 1.152, 0.030, and 0.112 (metric tons) of VOC, NO_x, SO_x, and PM, respectively, for long-span duration in Delhi. By the result, it is found that odd–even policy (OEP) scenario is the best-suited scenario in order to emission reduction of pollutants including (VOC, NO_x, SO_x, and PM). It would reduce VOC, NO_x, SO_x, and PM emission in future for short-span as well as long-span duration as compared to BAU and other control scenarios. The emission reduction of these pollutants in OEP scenario would reach to about 26.51%, 27.7%, 35.63%, and 30.29% for short-span and 24.54%, 29.23%, 35.98%, and 29.78% for long term of VOC, NO_x, SO_x, and PM, respectively, which is higher than other scenarios.

At next level, integrated scenario (IS) is the second most effective scenario compared to BAU and other control scenario. And the emission reduction in this scenario would reach to about 23.31%, 14.98%, 19.24%, 23.73%, and 17.83% for short-span and 30.47%, 13.34%, 13.91%, 27.07%, and 18.31% for long-span period of CO, VOC, NO_x, SO_x, and PM. While electronic vehicle emission (EVP) is the least effective scenario compared to BAU and other control scenarios for the emission reduction of other criteria pollutant except CO which is the most dominant in transportation sector and emission reduction of CO under this scenario would reach to 27.47% for short-span and 41.81% for long-span and for other 3.45%, 9.24%, 11.83%, and 5.38% for short-span and 2.15%, 13.91%, 18.16%, and 6.85% for long–term period of VOC, NO_x, SO_x, and PM in Delhi.

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Attribute Assessment for Sustainable Transportation Planning for Metropolitan Cities: A Fuzzy Approach

Leena Garg and Bhimaji K. Katti

Abstract The fast growth of 'surrounding regions' often termed as 'sprawl' of metropolitan cities is a matter of concern for all the concerned planning authorities. This haphazard development directly implies multitude of social, economic and environmental repercussion (Theobald 2001). With an extension of city limits, these areas get included in the city, the fringes of the cities keep moving outwards, new nucleus are formed, new flow paths for people and goods are established. Success and sustainability of transportation planning here depend upon many important attributes like availability of adequate road network, transit nodes, modes of transportation, ease of movement, and above all an integrated land-use planning approach, supportive of successful transportation planning. A timely prediction and assessment of growth attributes can help in prioritizing the planning efforts along with expenditure on infrastructure building. Very often such attributes are intangible and require a qualitative assessment based on human perception. Main objective is to put forth a methodology that includes the quantitative assessment of qualitative or 'human experience'-based inputs as a part of this paper. The method used includes mainly three phases: first phase includes the Delphi survey for getting the qualitative inputs of the attributes considered, the relative importance of the attributes is gauged using the AHP technique, and the outcome in terms of crisp quantitative assessment is derived using the fuzzy mathematical procedure. Outcomes can help in assessment of the 'sustainability status' as compared to certain 'best practice' scenario and thus can help devise strategies and policy formulation towards achieving the same.

Keywords Urban sprawl · Transportation attributes · Analytic hierarchy process (AHP) · Likert scale · Fuzzy synthesis · Attraction potential

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1 Introduction

Ever-increasing urbanization is a defining characteristic of the twenty-first century. The defining trait for human settlements worldwide in the twenty-first century is ever-increasing urbanization. Going with the present rate of urbanization, it is estimated by most scholars across the globe that by 2030 about 5 billion people are expected to live in urban areas, i.e. about 60% of the projected global population of 8.3 billion (Baharoglu 2002). Our metropolitan cities are growing exponentially by way of population as well as the spatial spread on the face of the planet. They have expanded into the adjoining rural areas in a haphazard and unplanned manner often referred to as urban sprawl. These newly developing expanded areas attract both new migrants and the city population from the inner core congested areas, due to availability of better and cheaper accommodations in less dense settlements. The attraction potential of such new areas is based on land or the floor area cost, transportation and road connectivity, quality of physical environment besides other parameters. Availability of transportation facility with good road network and connectivity with important transit nodes like railway station/bus stand play a key role in assessing the potential to attract the population in such areas.

A visit to any midsize city in India can reflect the results of lack of adequate land-use and transportation planning efforts. Poor transit infrastructure and traffic management are evident in most cities, although the policy to develop the 'smart infrastructure' is the major thrust area by governments at all levels. Thus, the importance of proper transportation planning and implementation on timely basis cannot be emphasized enough. Looking at such a scene, it is pertinent that the transportation infrastructure is gauged carefully at the planning stage both for assessing the demand and supply equation and for foreseeing the growth pattern of the city.

Attraction potential evaluation of the transportation infrastructure has been attempted for various expanded zones of Surat City as a case study in the present paper. Surat City, being one of the fastest-growing metropolitan cities, has been taken for the case study purpose. Key attributes like connectivity status, multiple modes of transit, ease of movement of traffic and the distance from railway station/bus stand have been taken to assess the attraction potential. Qualitative assessment from the experts in linguistic terms is collected and processed for the research enquiry following the 'Delphi technique' (Gene and Wright 1999). To tackle the inherent uncertainty involved with qualitative assessment, 'fuzzy synthesis' a mathematical technique has been utilized to quantify and process the data obtained by Delphi interviews. The 'crisp' numerical values so obtained shall represent the comprehensive indices of 'transportation infrastructure' quality for the expansion areas of the city and thus shall provide further guidelines for planning and development of the city.

2 Study Area Profile

Surat City in southern Gujarat is today a well-known commercial city, as well as one of the clean cities in India. Among the major industries, textile manufacturing and trade, Jari (decorative brocade/embellishment used in sarees, etc.), diamond cutting and polishing are the main industries. The city is one of the oldest historical trade centres of India. Due to rapid industrialization and construction activities, Surat has become an important growth magnet for the migrants. This resulted in higher decadal growth compared to any city in the state in the last two decades. The decade of 1991–2001 witnessed an exceptionally high decadal population growth rate of 85.09% due to rapid inflow of population, resulting in further densification of the core city. Outgrowth areas also experienced rapid rate of growth (Surat City Dev. Plan 2006-12). With the (old) municipal area in Surat reaching a density of 217 persons/hectare, the future development took place outside the municipal limits. Keeping these trends in view, the limits of SMC were increased to 326 km^2 in the year 2007 from earlier 112 km^2. The total new expansion of Surat City area is 214.23 km^2. The city has experienced typical radial development with central core and peripheral expansions. The city has been divided into seven planning zones as shown in Fig. 1. The expansion areas have been added to the earlier zones (except for central zone) resulting in the new city form (Fig. 1). The shaded area represents the expanded area along the unshaded central core zones (Table 1).

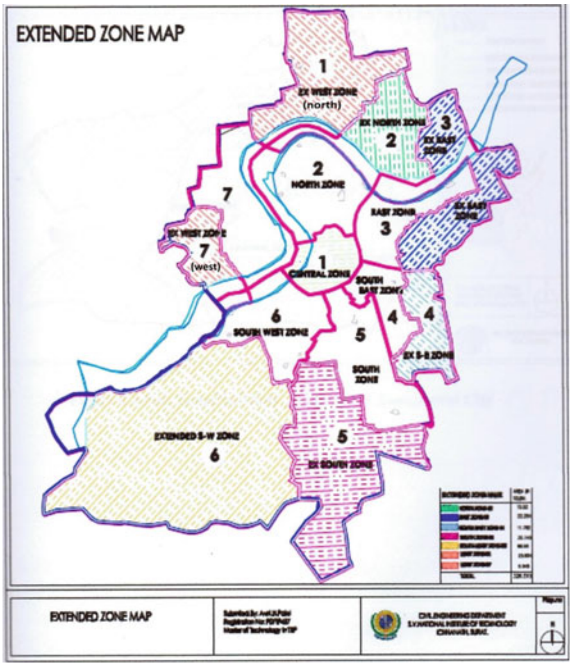

Fig. 1 Expanded zone map (Surat City)

Table 1 Expansion zone number coding

Existing Zones	Planning Zones	Expansion Zones	Code Numbers
1	Central zone	West zone expansion (north side)	1
2	North zone	North zone expansion	2
3	East zone	East zone expansion	3
4	South-East zone	South-east zone expansion	4
5	South zone	South zone expansion	5
6	South-West zone	South-west zone expansion	6
7	West zone	West zone expansion (west side)	7

3 Accessibility Attribute Rating

Transportation planning requires thorough assessment of anticipated travel demand in the newly developing expanded areas of any Municipal Corporation. For a well-informed decision-making process, the inputs from all the relevant stakeholders need to be represented at certain level. For qualitative assessment of the transportation scenario in a given sector/zone of a city certain attributes are shortlisted, like the total road lengths (all types) in a given area, routing maps leading to prominent business centres especially the transit nodes, modal choices, congestion index, etc. Here our attributes, understood to have maximum bearing upon the quality of transportation infrastructure are selected.

3.1 Rating of Transportation Attributes

For the purpose of rating, the attributes for expansion areas in Surat City, eleven different distinguished experts were consulted as per the Delphi technique. The confidentiality is maintained to get the unbiased assessment. The qualitative aspects can be quantified with certain predetermined calibration. They too are coded as expert-1, expert-2 and so on. The selected transportation attributes and rating done from 0 to 10, representing very poor to excellent as the linguistic expressions for the selected attributes (Tables 2 and 3).

Table 4 gives the typical rating for one zone.

Table 2 Transportation attributes of the expanded zones

S. No.		
1	Road connectivity with CBD	CC
2	Availability of diff. modes of transport	MT
3	Ease of Movement (lesser congestion)	EM
4	Distance from Railway station/bus stand	RS

Table 3 Quantification of the qualitative ranking

S. No.	Descriptive variables	Qualitative rating	Values
1	Very poor	E	2
2	Poor	D	4
3	Good	C	6
4	Very good	B	8
5	Excellent	A	10
6	Intermediate values		1, 3, 5, 7, 9

Table 4 Expansion zonal transportation attribute matrix (based on the expert ratings)

Transportation attributes	Experts										
	1	2	3	4	5	6	7	8	9	10	11
Connectivity with CBD	6	6	9	6	5	6	6	2	4	4	6
Choice of modes for transport	6	2	9	9	6	6	4	9	6	4	6
Ease of movement (lesser congestion)	4	8	6	8	5	7	8	5	8	4	9
Distance from railway station/bus stand	4	2	9	1	3	4	4	2	6	2	8

4 Multi Criteria Decision-Making (MCDM)

The process of decision-making in day-to-day life has been a subject of great interest and research in recent times. Most practical operational problems involve multiple criteria in decision-making (MCDM) problems. Simultaneously, optimization of the multiple criteria will provide the best decision alternative. MCDM refers to prioritising, ranking or selecting a set of alternatives under usually independent or conflicting criteria. MCDM can be extended on multi-objective or multiple attribute decision-making. In such problems, there can be single or group of decision-makers. Various approaches are adopted to deal with MCDM problems. Simple multi-attribute rating technique (SMART) and analytic hierarchy process (AHP) are commonly adopted to deal with MCDM problems, as they are simple and straightforward technique. A MCDM problem is characterized by ratings of each alternative with respect to each criteria and weights to each criterion (Hardy 1994).

4.1 Model Structure

Steps of fuzzy evaluation process are listed out below in order:

- Defining alternatives, based on basic attributes and forming a hierarchal framework (in this case four basic attributes to one attraction Potential Evaluation or APE);

Table 5 Likert type zonal attraction ratings

1	0–2	Very poor	The zone is very poor in terms of the attraction potential
2	2^{+}–4	Poor	The zone is poor in terms of attraction potential
3	4^{+}–6	Satisfactory	The zone is satisfactory in terms of attraction potential
4	6^{+}–8	Good	The zone has good potential for attraction
5	8^{+}–10	Very good	The zone has very good potential for attraction

Table 6 Likert scale and membership function

Attr	1 0–2	2 2–4	3 4–6	4 6–8	5 8–10
1	1	2	7	0	1
2	1	2	5	0	3
3	0	2	3	5	1
4	4	4	1	1	1

- Defining weights to each attribute (using analytic hierarchy process);
- Fuzzifying basic attributes;
- Defuzzifying final fuzzy sets and ranking the alternatives.

In fuzzy synthesis evaluation, the attraction attributes form the basic factors (Yau 2009; Yelin et al. 2010; Wang 2015). The response was taken on the scale of 10, as shown in the format of questionnaire, with lower end representing the least attractive and the higher end representing the most attractive. For the purpose of further simplification, the Likert scale is used to convert the ten-point scale into five-point scale. Rensis Likert gave the concept of this scale describing its usefulness in his report (Likert 1932), and thus, the scale derives its name. The scale was calibrated on the basis of response, as shown in Table 5.

The typical calculation for Expansion Zone-7 is as shown in Table 6.

The frequency values of grade evaluation matrix are used to note the membership values $\mu_i(x_j)$ from the graph (Fig. 2). It is in trapezoid format (Zadeh et al. 1996). As there is possibility of distortion of evaluation due to extreme values present in the rating (Gene Rowe, George Wright, 1999), and hence, extreme response of frequency ≤ 1 is ignored and equated to 0. Similarly, for frequency ≥ 8 is considered as the best and qualifies for 1. The membership function now would be as under:

The fuzzy membership function values $\mu_i[Z_i(x_j)]$ in the range of [1–0] is:

Table 7 is thus derived:

$$u[Z_i] = \frac{Z_i(x) - 1}{7} \begin{cases} 0 \; 0 \leq Z_i(x) \leq 1 \\ \quad 1 \leq Z_i(x) \leq 8 \\ 1 \; 8 \leq Z_i(x) \leq 11 \end{cases}$$

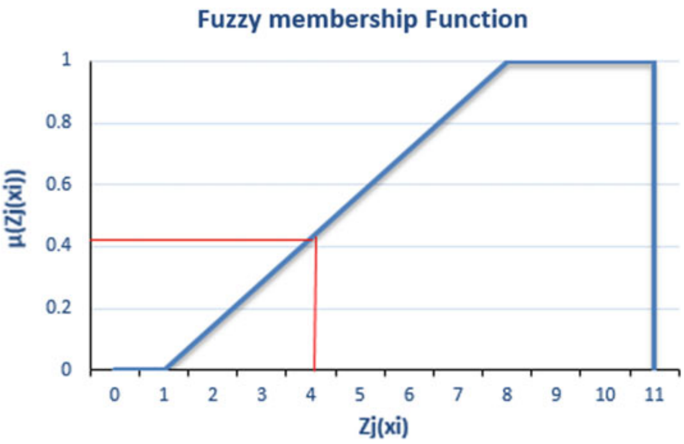

Fig. 2 Fuzzy membership function

Table 7 Likert conversion of expert inputs: graded frequency Criteria

0.000	0.143	0.857	0.000	0.000	0.212	0.000	0.030	0.182	0.000	0.000
0.000	0.143	0.571	0.000	0.286	0.368	0.000	0.053	0.210	0.000	0.105
0.000	0.143	0.286	0.571	0.000	0.140	0.000	0.020	0.040	0.080	0.000
0.429	0.429	0.000	0.000	0.000	0.280	0.120	0.120	0.000	0.000	0.000
						0.120	**0.223**	**0.432**	**0.080**	**0.105**

4.2 Defining Weights Using Analytic Hierarchy Process (AHP)

Pairwise Comparison

AHP or the analytic hierarchy process is being utilized for finding the relative importance of each participating attribute in a cluster. One important feature of AHP is the pairwise comparison between the attributes at each level. Concentrated judgements between a pair of elements and compared on a single property from the whole lot of other elements or attributes are the most effective way to analyse the relative importance. Paired comparisons in combination with well-derived hierarchical structure therefore serve the purpose of deriving a well-informed quantified measurement of entire structure (Saaty 1990). Table 8 gives the AHP computation for transportation infrastructure.

A reasonable, assumption in this calculation is that if attribute A is absolutely more important than attribute B and is rated at 5, and then B must be absolutely less important than A and is valued at 1/5. Thus, pairwise comparisons are carried out for all factors to be considered, and the matrix is completed.

Consider the attributes to be compared, as $C_1 \ldots C_n$ and denote the relative 'weight' (or priority or significance) of C_i with respect to C_j by a_{ij} and form a square

Table 8 Pairwise comparison for Transportation Infrastructure (TI) attributes

	CC	MT	EM	RS			
CC	1	0.333	2	1	0.67	0.903	**0.212**
MT	3	1	2	1	6.00	1.565	**0.368**
EM	0.5	0.5	1	0.5	0.13	0.595	**0.140**
RS	1	1	2	1	2.00	1.189	**0.280**
						4.252	**1.000**

matrix $A = (a_{ij})$ of order n with the constraints that $a_{ij} = 1/a_{ij}$, for $i \neq j$, and $a_{ii} = 1$, all $i = j$. For calculating the eigenvector, multiply together the entries in each row of the matrix and then take the nth root of that product. The nth roots are summed and that sum, normalized gives the eigenvector elements to get the weightage factor.

4.2.1 Consistency Testing

The final stage is to calculate a consistency ratio (CR) to measure how consistent the judgements have been; which is a ratio of consistency index (CI) to reference index (RI),

$$\mathrm{CR} = \mathrm{CI}/\mathrm{RI}$$

$$\mathrm{CI} = \frac{(\lambda\mathrm{max} - n)}{(n - 1)}$$

where

n Matrix dimension.
$\lambda_{\max}$ eigenvalue.
RI Random index as per Saaty's RI (Table 9).

$\boldsymbol{\lambda_{\max}}$ can be determined by finding 'A_w' values by referring to comparative matrix 'A' and eigenvalue 'w'. According to AHP theory;

$$\boldsymbol{\lambda_{\max}} = A_w/w$$

The difference, if any, between $\lambda_{\max}$ and n is an indication of the inconsistency of the judgments. If $\lambda_{\max} = n$, then the judgements have turned out to be consistent. The corresponding values can be checked with reference to Saaty's RI value (Table 9).

Table 9 Saaty's reference random index (RI)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.00	0.00	0.58	0.90	1.12	1.24	1.32	1.41	1.45	1.49	1.51	1.48	1.56	1.57	1.59

Here mean value is 4.153, and CI value comes out to be 0.051. The test application for the above illustration indicates values of consistency index and CR value observed as 0.057 which is less than 0.1. Hence, the pair comparison observation is consistent.

4.3 Synthesis Operations

Synthesis evolution matrix 'SE' for transportation infrastructure (TI) of expansion area (EZ-7) is represented as below:

TI cluster

- Connectivity with CBD
- Choice of Modes for transport
- Ease of Movement (lesser congestion)
- Distance from Railway station/bus stand

$$\text{Synthetic Evaluation matrix SE} = [woR],$$

where

w_i Weightage (AHP) Vector for TI

R Fuzzy Evaluation Matrix of TI can be depicted as under

	w_i		e_1	e_2	e_3	e_4	e_5
SE=	0.212		0.000	0.030	0.182	0.000	0.000
	0.368	o	0.000	0.053	0.210	0.000	0.105
	0.140		0.000	0.020	0.040	0.080	0.000
	0.280		0.120	0.120	0.000	0.000	0.000

=	**0.120**	**0.223**	**0.432**	**0.080**	**0.105**

$$\boldsymbol{b_j = \{0.120^*, 0.223, 0.432, 0.080, 0.105\}}$$
$$\boldsymbol{*b_1 = 0.000 + 0.000 + 0.000 + 0.120}$$

4.4 Attraction Potential Values

'b_j' is fuzzy values for five grades of e_1 (Very Poor), e_2 (Poor), e_3 (Medium), e_4 (good), e_5 (Very Good). 'b_j' fuzzy values obtained through synthetic operation on summation basis are now to be transformed to crisp values so as to find the final value of attraction potential value for 'TI' as attraction factor for the given zone. Each value of 'e_j' is multiplied by corresponding Likert scale value, i.e. from 1 to 5 and added to get the crisp value of attraction factor, attraction potential evaluation (APE). In the present case, it comes out to be as follows:

$$\begin{aligned}\mathbf{APE_7(TI)} &= \mathbf{2.302(0.120 * 1 + 0.223 * 2 + 0.432 * 3} \\ &\quad \mathbf{+ 0.080 * 4 + 0.105 * 5) = 2.708}\end{aligned}$$

Similarly, the APE for all the expansion zones can be found and the result is as shown below:

Expansion zones	1	2	3	4	5	6	**7**
APE	2.520	3.697	3.556	3.048	2.577	2.618	**2.708**

5 Conclusion

The rapid urbanization process taking place especially in our metropolitan cities shall have an inevitable implication in terms of expanding the city limits. Such new areas trigger further urban growth, which needs to be monitored and planned on timely basis. Attributes like land values, accessibility and quality of transportation infrastructure, availability of network services are some of the factors that provide pace for the growth.

The study is based on the assessment of transportation infrastructure-related attributes on Delphi technique basis, where transport terminal accessibility and quality of infrastructure facility have been considered as the main attributes. The analysis of attributes based on the expert ratings provides the guideline for ranking of the expanded areas, for future transportation development. Accordingly, North exp. zone of Surat City area stands the first rank, and East expanded area on the right bank of river Tapi finds the second rank. The zone taken as case study, i.e. the west zone (west side) is next in ranking. This kind of analysis can be used to derive meaningful deductions regarding the state of subheads of the TI as prevalent in various sectors or zones of the city, and corresponding planning action to be taken to improve the ranking or overall score. The methodology incorporates the ideology of community participation in planning which is a goal of good governance.

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Waste to Energy: Piezoelectric Energy Harvesting from Vehicular Movements

Amanjot Singh, Naveet Kaur, and Suresh Bhalla

Abstract Wireless sensor technology used for continuous structural health monitoring (SHM) of the road infrastructure has increased the usability as well as the capability of data collection. However, electrochemical batteries are the main power source for these wireless sensors but they have to be replaced or charged regularly. For efficient usability of this, wireless health monitoring systems can efficiently self-sustain via an energy source. The solution lies in piezoelectric energy harvesting from road pavement itself using the mechanical energy generated by moving traffic. In this paper, a prototype for piezoelectric energy harvesting (PEH) has been developed utilizing the d_{33} mode of piezosensors, which converts the vehicular motion energy into electrical energy. Testing of prototype PEH has been conducted by fixing it on road surface to carry out a comprehensive parametric study with varying pavement surface, weight of vehicle, and speed of vehicle on the voltage and power output. It is found that power generated from PEH surface bonded on concrete surface is approximately **10%** higher as compared to bitumen pavement, and it is also increasing with the increase in speed and weight of vehicle. Maximum open-circuit voltage of **82 V** and peak power output of approximately **2.8 mW,** whereas maximum average power output of **0.25 mW** is generated with a truck weighing 7 Ton at speed of 40 km/h on concrete surface using the 1 MΩ load resistance circuit. Maximum energy of **0.72 μJ** was stored in a **10 μF** capacitor with single pass of a vehicle.

Keywords Piezoelectric energy harvesting · Pavements · Vehicular motion · Peak power · Average power · Traffic

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1 Introduction

In today's world, automobiles are one of the most often used means of conveyance. India being a developing nation, hence, both the number of vehicles and the road network are increasing at an astonishing scale. Though this can act as a problem for the environmentalists, this can also act as boon in other ways. It results in a great potential for piezoelectric energy harvesting from the mechanical energy generated by vehicular motion. The external mechanical stress from the vehicular motion can be converted into the electrical voltage with the help of piezoelectric material, and hence, energy can be harvested in this way. Lead zirconate titanate (PZT) is one of the most robust commercially available artificial piezomaterials.

Vehicle load through the tyres will be transferred on to the road surface. The stress produced by this load will be converted into the electrical voltage using the stack actuator or d_{33}-mode of piezomaterial, thus resulting in an energy harvester. However, the vehicular load will be imparting an impact force on the road; hence, the d_{33} mode of piezo is being used in the piezoelectric energy harvesting.

Lee and Choi (2014) developed a piezoelectric energy harvester utilizing d_{33} mode of piezo which can be mounted inside a tyre for self-powering a wireless sensor system to monitor the health of tyre for increasing the vehicular safety. An output power of **1.37 μW/mm³** was achieved. Behera (2015) also conceptualized piezo-electric energy harvesting by fixing of piezo materials inside the circumference of the tyre. By fixing 32 modules of PZT-5 A material, an estimated power output of **1.2 mW** per rotation of tyre was numerically achieved. Hiba and Muthukumaraswamy (2016) adjudged that higher electromechanical coupling factor (k) makes the PZT pile and multilayer type of piezoelectric transducers most suitable for energy harvesting from road pavements. It was also concluded that there exists a linear relation between the applied external stress and the electric potential generated by the transducer. A numerical study is done by the authors claimed to produce an energy output of **150 kW** from a surface bonded harvester in one hour from one-lane road of one km highway using truck volume of 600 per hour. Innowattech Energy Harvesting Systems, a private company in Tel Aviv, Israel, has claimed an output power of **200 kWh/h** from one km single lane of a road using embedded harvesters along two-wheel footprints with about 600 heavy vehicles per hour, moving at a speed of 72 km/h on average (Fig. 1).

2 Experimental Work Done and Analysis

Two prototypes energy harvesters were fabricated in the SSDL laboratory (SSDL 2018) using 04 Nos of 3-mm-thick aluminium plates of size 20 cm × 5 cm (Fig. 2). Prototype-I was prepared using CEL (Central Electronics Ltd. 2018) circular disc type piezo PZT-5A sensors of 25 mm diameter and prototype-II with 20 mm diameter sensors. Thickness of sensors used was 1.83 mm.

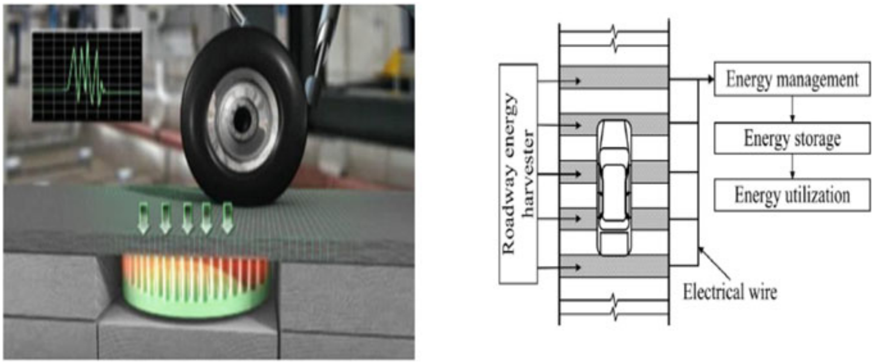

Fig. 1 Mechanism of piezoelectric energy harvesting by vehicular motion (Hiba and Muthukumaraswamy 2016)

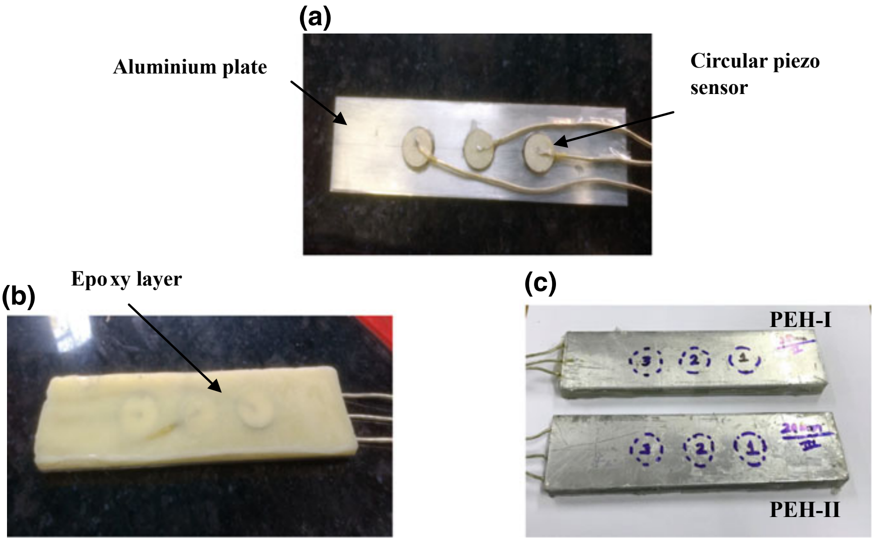

Fig. 2 Different stages of fabrication of prototype-I & II. **a** Different piezo sensors fixed on the plate. **b** Epoxy layer applied. **c** Top plate fixed

An indirect approach for measuring current and thus power was adopted in this study. A current measuring circuit as shown in Fig. 3 was fabricated and used for the same. As per the maximum power transfer theorem, the maximum amount of power will be dissipated when the load impedance matches with the impedance of the network supplying the power. The load resistances in the current measuring circuit were chosen on the basis of maximum power transfer theorem. The high electrical impedance of PZT patch results in a very small (in the range of micro Amperes) current flowing through the circuit, which is challenging to be measured directly

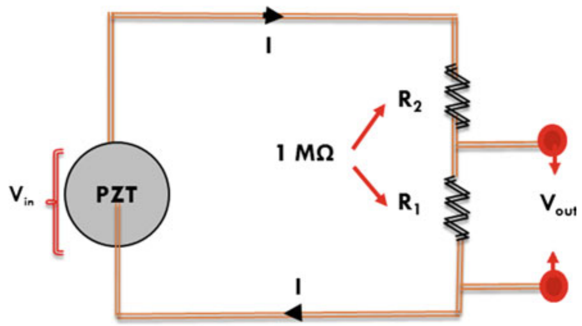

Fig. 3 Line diagram of current measuring circuit

(Kaur 2015). Therefore, the voltage (V_o) across the R_1 was measured, and further, the current and power output generated by PZT patch was determined using formula

$$I(\text{Current}) = V_o/R_1 \tag{2.1}$$

$$P(\text{Power}) = I^2(R_1 + R_2) \tag{2.2}$$

To achieve maximum power output, the resistances R_1 and R_2 were selected such that total load resistance ($R_1 + R_2$) has similar order of impedance magnitude as the PZT patch. Therefore, $R_1 = R_2 = 1$ MΩ was chosen as the impedance value of PZT patch was approximately 2 MΩ.

2.1 Comparison of Power Output from Different Size Sensors

PEH prototype-I and II were fixed on concrete pavement surface for comparison of power output by different sizes of piezosensors under d_{33} mode. SUV (INNOVA) weighing 2200 kg was used for inducing external stress on the prototype during the field experiment. Figure 4 shows the complete set-up of testing. Current measuring circuit as shown in Fig. 3 was used. Vehicle tyre was made to run over the prototype at varying speed (Fig. 4c) and using oscilloscope open-circuit voltage and voltage output across R_1 (Fig. 3) was recorded for sensors of both the specimens. Current (I) and peak power (P) were calculated using Eq. 2.2.

Table 1 shows that maximum open-circuit (OC) voltage of **80 V** was generated by PEH-I (25 mm dia sensor) when vehicle speed was 55–60 km/h, whereas maximum OC voltage of **76 V** was generated by PEH-II (20 mm dia sensor) at same vehicle speed. Peak power generated by PEH-I is **450 μW,** whereas PEH-II generated peak power of **359.12 μW**. Figure 5 shows the comparison of peak power generated by PEH-I & PEH-II at various speeds. It shows that power generation is increasing with the increase in speed of vehicle and also power output of CEL 25 mm dia sensor is

Fig. 4 Photographs showing different stages of testing of prototype. **a** Fixing of prototype on pavement surface. **b** Experiment set-up using oscilloscope, current measuring circuit and Innova. **c** Vehicle tyre passing over the prototype

Table 1 Open-circuit (OC) voltage, peak voltage and power across circuit generated by sensors

Vehicle speed (km/h)	Max OC voltage (V)		Peak voltage (V) across R_1		$R_1 + R_2$ (MΩ)	Peak power $P = I^2 (R_1 + R_2)$ (μW)	
	PEH-II	PEH-I	PEH-II	PEH-I		PEH-II	PEH-I
0–5	60	68	4.6	12	2	42.32	288.00
15–20	66	70	7.6	12.6	2	115.52	317.00
35–40	70	78	11.2	13.4	2	250.88	359.00
55–60	76	80	13.4	15	2	359.12	450

better than CEL 20 mm dia sensors as the surface area of 25 mm dia sensor is more than the surface area of 20 mm dia sensor.

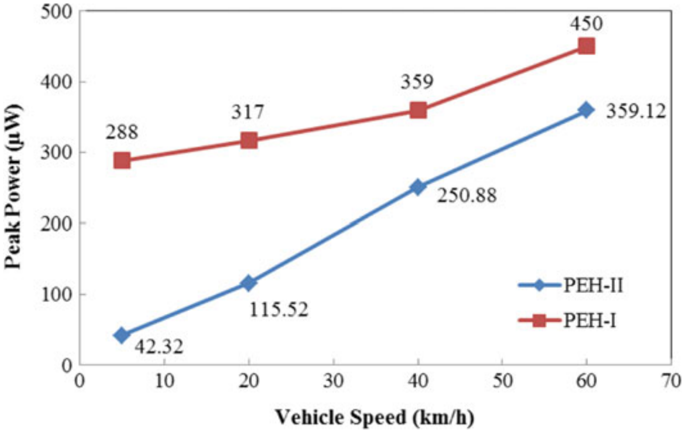

Fig. 5 Comparison of peak power generated by PEH-I & PEH-II at varying speeds

2.2 Comparison of Power Output with Different Pavement Types

PEH-I was fixed firstly on concrete surface and then on bituminous surface as shown in Fig. 6 for analysing the difference in output generation. SUV (Innova) was used for inducing external stress on the prototype. Vehicle tyre was made to run over the PEH at varying speeds for inducing external stress by the impact. Open-circuit voltage and voltage output across R_1 were recorded using oscilloscope. Further, peak power was calculated from the voltage output.

Maximum open-circuit voltage of **78 V** and **54 V** was generated by PEH-I (25 mm dia sensor) at concrete and bitumen surface, respectively (Refer Table 2). Peak power of **359 μW** and **327.68 μW** was generated by PEH-I when surface bonded on

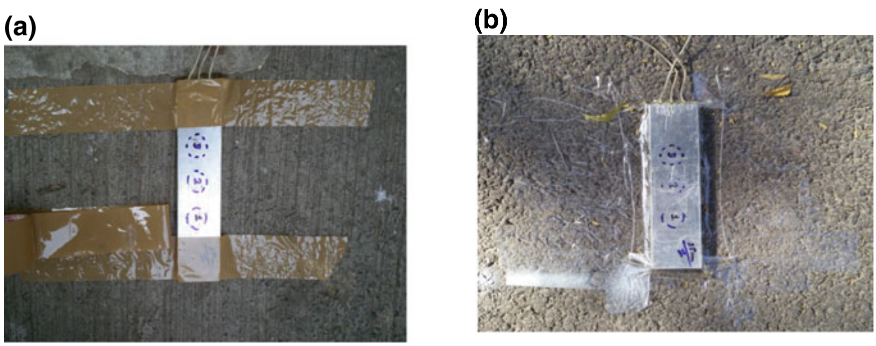

Fig. 6 Photographs showing fixing of PEH-I on different surfaces. **a** Fixing of prototype on concrete pavement surface. **b** Fixing of prototype on bituminous pavement surface

Table 2 Peak voltage and peak power generated by PEH-I (25 mm dia sensor) at bitumen and concrete pavement across R_1

Vehicle Speed (km/h)	Max OC Voltage (V)		Peak voltage (V) across R_1		$R_1 + R_2$ (MΩ)	Peak power $P = I^2 (R_1 + R_2)$ (μW)	
	Bitumen	Concrete	Bitumen	Concrete		Bitumen	Concrete
0–5	44	68	11	12	2	242.00	288.00
15–20	52	70	11.8	12.6	2	278.48	317.00
35–40	54	78	12.8	13.4	2	327.68	359.00

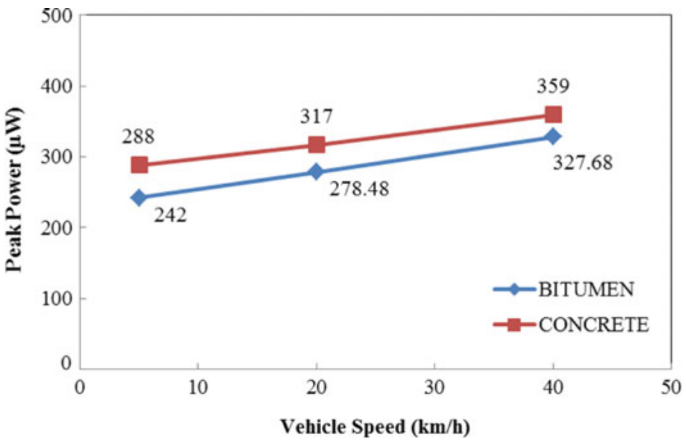

Fig. 7 Comparison of peak power generated by PEH-I at varying vehicle speeds on bitumen and concrete surfaces

concrete and bituminous surface, respectively (refer Table 2 and Fig. 7). Power output generated by PEH-I bonded on concrete surface was approximately **10% higher** than bitumen surface. This is due to the fact that concrete pavement is more rigid as compared to bitumen pavement. Power generation also increases with the increase in speed of the vehicle as the impact force on the sensor increases with the increase in speed of tyre hitting the PEH.

2.3 Comparison of Power Output Using Different Vehicle Type

Three vehicles with different weight were used for inducing external stress on the prototype. Vehicle No. 1—Fiat Punto (Car) with weight 1.1 Ton and tyre width 19 cm, Vehicle No. 2—Toyota Innova (SUV) with weight 2.2 Ton and tyre width 20.5 cm and Vehicle No. 3—Truck Eicher with weight 7 Ton and tyre width 19 cm were used

Fig. 8 Photographs showing different vehicles used for testing. **a** Fiat Punto (Car). **b** Toyota Innova (SUV). **c** Eicher (Truck)

for the study (Fig. 8). The current measuring circuit used was same as used earlier. PEH-I was fixed on surface of concrete pavement, and vehicle tyre was made to run over the PEH at varying speeds for inducing external stress by impact. Open-circuit voltage and voltage output across R_1 were recorded using oscilloscope. Further, peak power and average power were calculated from the voltage output. Average power was calculated by determining area under the curve of power and time for the contact period for excitation. Maximum open-circuit voltage of **82 V** (Refer Table 3), peak power of **2.8 mW** (Refer Table 4 and Fig. 9) and Max average power of **0.25 mW**

Table 3 Maximum open-circuit voltage generated by PEH-I with different vehicles

Vehicle speed (Km/h)	Max open-circuit voltage (V)		
	Fiat Punto (Car) (1.2 Ton)	Toyota Innova (SUV) (2.2 Ton)	Eicher (Truck) (7 Ton)
0–5	58	68	58
15–20	64	70	80
35–40	68	78	82

Table 4 Peak voltage, peak power and average power generated across R_1 with Car, SUV and Truck

Vehicle Speed (km/h)	Peak voltage (V) across R_1			Peak power $P = I^2 (R_1 + R_2)$ (μW)			Average power (μW)		
	Car	SUV	Truck	Car	SUV	Truck	Car	SUV	Truck
0-5	10.4	12	19.2	216.32	288	737.28	40.57	53.2	71.8
15-20	12	12.6	27.2	288	317	1479.68	52.9	58.44	200.56
35-40	12.8	13.4	37.6	327.68	359	2827.52	58.88	70.55	253.1

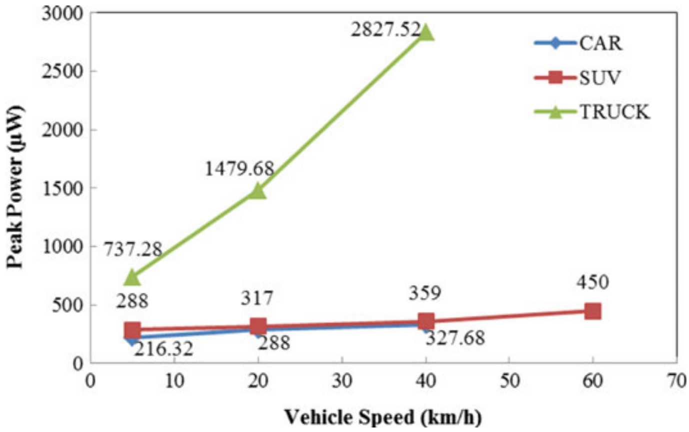

Fig. 9 Comparison of peak power generated at varying speeds with Car, SUV and Truck

(Refer Table 4 and Fig. 10) were generated by PEH-I with truck weighing 7 Ton. The power output increases with the increase in weight of vehicle used as the stress on the prototype is also increasing. Exceptionally higher power output of truck as compared to SUV may be because of lesser width of its tyre. Lesser contact area and with 1 Ton increase in weight, stress induced on the PEH also increases to a large extent which results in higher power output.

2.4 *Energy Harvesting Using Different Vehicle Types*

An energy harvesting circuit (Fig. 11) was fabricated in SSDL laboratory for storage of generated power from prototype in a capacitor. This circuit consists of a full wave bridge rectifier which converts AC input into DC output and a 10 μF capacitor for storage of DC output. Sensors in the prototype were connected to bridge rectifier for providing input voltage as generated with the passage of vehicle tyre over the prototype PEH to analyse the storage of output voltage and energy using different vehicles.

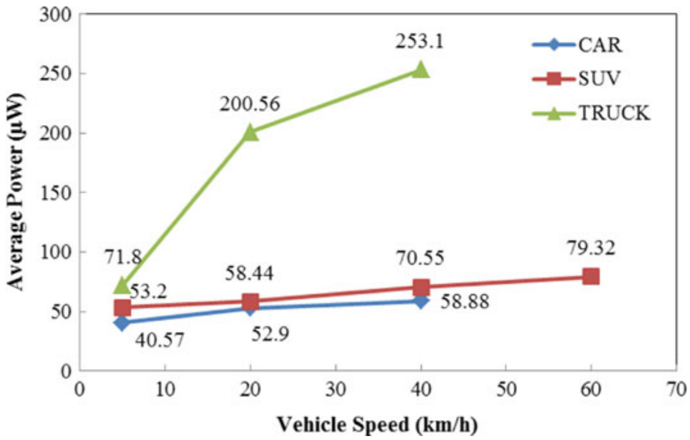

Fig. 10 Comparison of maximum average power generated at varying speeds with Car, SUV and Truck

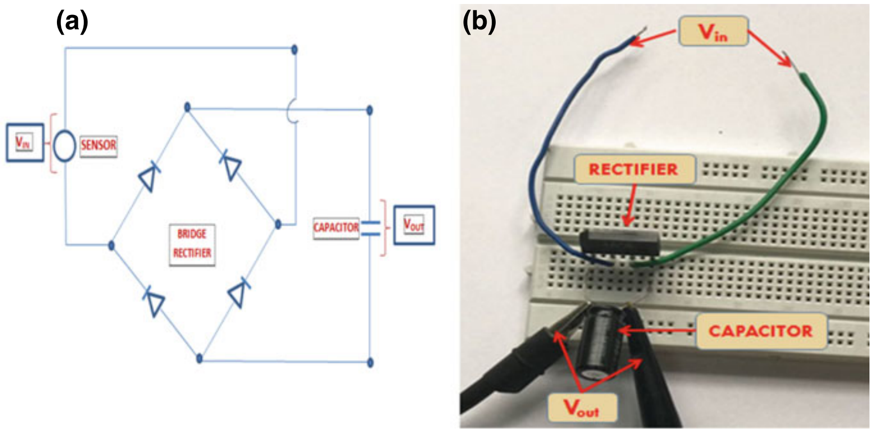

Fig. 11 Circuit for energy storage in capacitor. **a** Line diagram of circuit. **b** Actual circuit used for storing energy in capacitor

$$E = 1/2CV^2 \tag{2.3}$$

Using Eq. (2.3), the energy stored in the capacitor can be calculated. Here, C is the capacitance of capacitor and V is the voltage across the capacitor which was calculated with oscilloscope. Piezoelectric energy generated was calculated, and maximum energy of **0.34 μJ, 0.45 μJ and 0.72 μJ** was stored in 10 μF capacitor with Car, SUV and Truck, respectively (As per Table 5). Power and Energy output generated and stored increases with the increase in weight of vehicle used to induce external stress. Energy stored in capacitor is 6–24 times lesser than the energy being

Table 5 Comparison of maximum energy and power stored in 10 μF capacitor with single pass of Car, SUV and Truck

Type of vehicle	Voltage across Capacitor (V)	Energy generated (μJ)	Energy stored (μJ)	Power stored (μW)
Car	0.26	2.12	0.34	0.85
SUV	0.30	3.95	0.45	1.13
Truck	0.38	17.21	0.72	1.80

Table 6 Vehicle passes required for operation of various sensors

Type of sensor	Power required (μW)	No. of vehicle passes	Applications
TMP112 (Texas Instrument 2018)	36	20	Temperature sensor
PCT2202UK (NXP Semiconductors 2018)	54	30	Temperature sensor
MSP430FR4133 (Texas Instrument 2018)	453.6	252	Air quality index sensor
Colibry's VS1000 (Safran Colibrys 2018)	4170	2317	Vibration sensor (SHM)

generated by PEH which may be due to various losses in transmission and storage circuit.

3 Practical Applications

Energy harvested by the prototype can be utilized for powering various low power sensors such as temperature/heat detection sensors, air quality sensors and structural health monitoring sensors. Power requirement of one such sensor, TMP112 used as digital temperature sensor, is **36 μW,** whereas a maximum power of **1.8 μW** was stored experimentally in a capacitor with a single pass of truck (weight = 7 Ton). Therefore, a total of 20 such passes can store sufficient power in a capacitor for one-time operation of TMP112. Similar sensors (as shown in Fig. 4.16) which can operate with the power generated by PEH are summarized in Table 6.

4 Conclusions

This study resulted in the following conclusions:

1. Successful fabrication of a prototype piezoelectric energy harvester which converts impact energy from vehicular motion into electrical energy has been done and experimentally validated.
2. The power output from CEL 25 mm dia ceramic sensors is more as compared to the CEL 20 mm dia sensor which is due to larger surface area of sensor. Peak power output of approx. **2.8 mW,** whereas maximum average power output of **0.25 mW** is achieved by CEL 25 mm dia sensor.
3. Power generated from PEH surface bonded on concrete surface is approximately **10%** higher as compared to bitumen pavement as the rigidity of concrete surface is more than the bitumen surface.
4. Power generated increases with the increase in weight of vehicle used to induce external stress. Average power generated by Car (weighing 1.1 Ton) is **58 μW**, SUV (weighing 2.2 Ton) is **79 μW,** whereas Truck (weighing 7 Ton) is **0.25 mW**.
5. Power generated also increases with the increase in speed of vehicle as impact force increases with increasing speed.
6. Energy stored in capacitor also increases with the increase in weight of vehicle used to induce external stress. Energy stored in capacitor with single pass of Car (weighing 1.1 Ton) is **0.34 μJ**, SUV (weighing 2.2 Ton) is **0.45 μJ,** whereas Truck (weighing 7 Ton) is **0.72 mJ**.
7. Energy stored in capacitor is **6–24** times lesser than the energy being generated by PEH which may be due to various losses in transmission and storage circuit.

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Speed Trajectory of Vehicles in VISSIM to Recognize Zone of Influence for Urban-Signalized Intersection

Boski P. Chauhan, Gaurang J. Joshi, and Purnima Parida

Abstract Heterogeneous traffic in India is a complex phenomenon deals with highly transient vehicle compositions, their static and dynamic characteristics, and combination of non-motorized and motorized vehicles. It is difficult to collect and analyze the real traffic data due to the complexities of mixed traffic, creates need for simulation model to replicate the actual traffic scenario. VISSIM is microscopic traffic simulation software used to model urban-mixed traffic conditions. It is able to generate speed trajectory of vehicles in network, which is used to analyze driving activities of vehicle in terms of average speed, maximum speed, acceleration, deceleration, idling period, cruise period, etc. These driving activities mostly occur at signalized intersection because of signal control operations and creates high pollution zone. Present paper highlights traffic flow simulation for signalized intersection in VISSIM to recognize zone of influence through acceleration and deceleration activities of vehicle. It is the stretch at which vehicles come under influence of the intersection and lead to sharp deceleration followed by idle and acceleration phase. Vehicle speed trajectory is generated in VISSIM and zone of influence is identified based on spatial speed fluctuation profile. Calibration of model is carried out to match the vehicles' speed–acceleration parameters to the real traffic conditions. The speed trajectory of vehicles in real traffic has been collected through velocity box instrument embedded with GPS system. The model is validated by comparing simulation results with collected real traffic data through statistical errors. An urban-signalized intersection of Vadodara city located in Gujarat state is for the study.

Keywords Speed trajectory · Zone of influence · Driving parameters · Speed acceleration frequency matrix (SAFM) · VISSIM · Sum square difference

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1 Introduction

In urban area, vehicular population increases day by day causing complex traffic problems and more congestion (Partin et al. 2015). In India, heterogeneous traffic conditions generate vehicles chaos at intersections and junctions (Manjunatha et al. 2012). At signalized intersection, vehicle has to slow down the speed and follow the signal control system. Due to signalized control, more fluctuation occurs in speed trajectory of vehicles (Meneguzzer et al. 2017; Papson et al. 2012). It is difficult to track trajectory of all vehicles in mixed traffic conditions (Song et al. 2015). In context of difficulty in real traffic data collection and complexity in traffic conditions, development of simulation model is required (Abou Senna et al. 2013). Microscopic traffic simulation model builds real traffic conditions and drags traffic characteristics of actual situation (Manjunatha et al. 2012). In current study, VISSIM model is used to evaluate driving characteristics of vehicle at signalized intersection. In signalized intersection, vehicle is observed with driving parameters such as deceleration, idle, acceleration, and cruise mode. The zone of influence is the length of stretch, which starts with deceleration mode, followed by idle and finishes with acceleration mode. In zone of influence, driver has to compulsory follow the preceding vehicle because of signal control system and traffic congestion. It is the zone where high pollution occurs due to frequent activities of deceleration, idle, and acceleration of vehicles (Mudgal 2011).

Verkehr In Stadten–SIMulationsmodell (VISSIM) is a traffic flow modeling program used to simulate various traffic before and after transportation development (Alzuhairi et al. 2016; Doina and Chin 2007). VISSIM is a multi-modal microsimulation traffic modelling software which includes car following and lane change behavior in its simulation (Fellendorf and Vortisch 2010; PTV VISSIM 2015). In VISSIM, many intersections can be modelled simultaneously, which includes vehicular movements on road links between adjoining intersections and coordination of signal system (Siam et al. 2018). VISSIM modelling includes data collections, base data preparation, network coding, calibration and validation, static routing and dynamic assignment function, and evaluation.

The present study highlights the identification of zone of influence based on speed trajectory of vehicles. The study is useful for finding location of high pollution zone at intersection because of sharp acceleration and deceleration activities (Bokare and Maurya 2013). The simulation tool is used for generating speed trajectory of vehicles, which is further analyzed for finding zone of influence. The large length of influence zone indicates more traffic congestion and high pollution stretch. The high pollution zone can be reduced by managing traffic control system and modifying traffic composition by increasing use of public transportation in urban area (Elkafoury et al. 2015; Smit et al. 2010). The simulation tool is the software which runs on the modification of the traffic situation and shows scenario after implementations of control system (Chen and Yu 2017). In this study, only analysis of speed trajectory for finding zone of influence is shown. The actual data of speed profile is collected in morning peak hour by test car. The actual data is taken as input parameter in

VISSIM and the results of speed–time trajectory in VISSIM are compared with the actual data. SAFM is generated for the comparison of actual data and simulated data. Percentage of acceleration, idle, and deceleration time is calculated as main driving parameters from speed–time data. Speed trajectory generated in VISSIM is matched on the basis of Sum Square Difference (SSD) of the actual and modelled data in form of speed and acceleration classes.

The prime objective of study is to carry out traffic simulation in VISSIM for signalized intersection of Vadodara city to recognize zone of influence. VISSIM generates speed trajectories of all vehicles situated in network. The study compares the trajectories of simulation data with actual data. The comparison is carried out on the basis of sum squared difference of the speed and acceleration results.

2 Literature Review

Vissim is the leading microscopic simulation program for modelling multi-modal transport operations (Fellendorf and Vortisch 2010). Realistic and accurate in every details, Vissim creates best conditions to test different traffic scenarios before their realization. Vissim is being used worldwide by the public sector, consulting firms, and academicians (PTV VISSIM 2016). Vissim is a microscopic, time step oriented, and behavior-based simulation tool for modeling urban and rural traffic as well as pedestrian flows. The traffic flow is simulated under different constraints of lane distribution, vehicle compositions, signal control, and public transport vehicles (PTV VISSIM 2016). Vissim is based on a traffic flow model and the light signal control. The traffic flow model is based on a car-following model (for the modeling of driving in a stream on a single lane) and on a lane changing model. Vehicles are moving in the network using a traffic flow model. The quality of the traffic flow model is essential for the quality of the simulation. In contrast to simpler models in which a largely constant speed and a deterministic car following logic are provided, Vissim uses the psycho-physical perception model developed by Wiedemann 1974. The basic concept of this model is that the driver of a faster moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. Since he cannot exactly determine the speed of that vehicle, his speed will fall below that vehicle's speed until he starts to slightly accelerate again after reaching another perception threshold. There is a slight and steady acceleration and deceleration. The different driver behavior is taken into consideration with distribution functions of the speed and distance behavior.

Partin et al. (2015) developed a model in VISSIM to estimate emission for the parameters of average speed, total distance travelled, total stop delay, and number of roads for Indian River road, Chesapeake, Virginia. Abou-Senna et al. (2013) have described VISSIM, microscopic traffic simulation model for urban highway and further it was compiled with MOVES 2010. Results generated in VISSIM on a second basis were input to the MOVES model to estimate emissions. Song et al. (2015) have given the delay correction models for adjusting emission factors for each type of intersections and different numbers of stops. A comparative analysis between estimated and field emission factors demonstrates that the delay correction model has been carried out with reliable results. Li et al. (2009) have focused on providing information to the drivers with advanced signal status specifically for energy and emissions reduction. A generic method has been explained to analyze the effect of influence of intersection on emissions and vehicle energy important for analyzing or optimizing the effectiveness of control systems at signal or driver information systems at intersections.

3 Study Area and Methodology

3.1 Study Area

In Vadodara city, the vehicular population is increasing at an alarming rate and bulk of vehicular traffic movement is handled by 30 main roads. On an average, the vehicular population increases at a rate of around 9% per year. More than 50% of the existing vehicular population comprises two-wheelers. Old Padra (OP) road is situated in the upbeat and throbbing area within the Vadodara Municipal Corporation on very convenient and strategic location. OP Road is a major transit point connecting the posh Alkapuri Akota areas with upcoming Vasna Road and going to the base of a new fly over toward Padra. Old Para Road is prime location of Vadodara located 3kms from Vadodara Railway Station in West Vadodara. Malhar circle is situated on OP road and faces traffic problem almost every day in peak hour period. The vehicles have to slow down speed when come under the influence of signal. Malhar circle is a signalized four-legged intersection consists of major and minor streets. Each approach is operated as an individual phase. The total signal cycle length is 120 s. Instrument performance box was set up on car to obtain the speed trajectory of vehicle under actual traffic condition. The instrument gives speed trajectory of vehicles which is used as input data for simulation. Figure 1 shows location of Malhar circle with traffic volume in individual approaches.

3.2 Data Collection

Data collection has been carried out with the help of instrument, and data extraction was done through race logic software. The instrument records speed trajectory of respective vehicle on which it is mounted. Data has been collected on working days peak hour period between 10:00 a.m. and 11:00 a.m. Speed time trajectory can be extracted for the 1/10th of the second at microscopic level. Speed fluctuation parameters: acceleration, deceleration, idle, and cruise speed are calculated from the

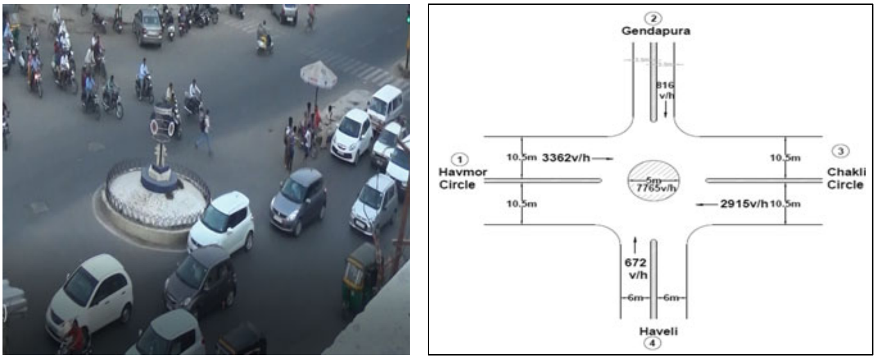

Fig. 1 Location of Malhar circle and its traffic volume

actual data. Figure 2 shows the speed trajectory of car for Malhar circle during peak hour period.

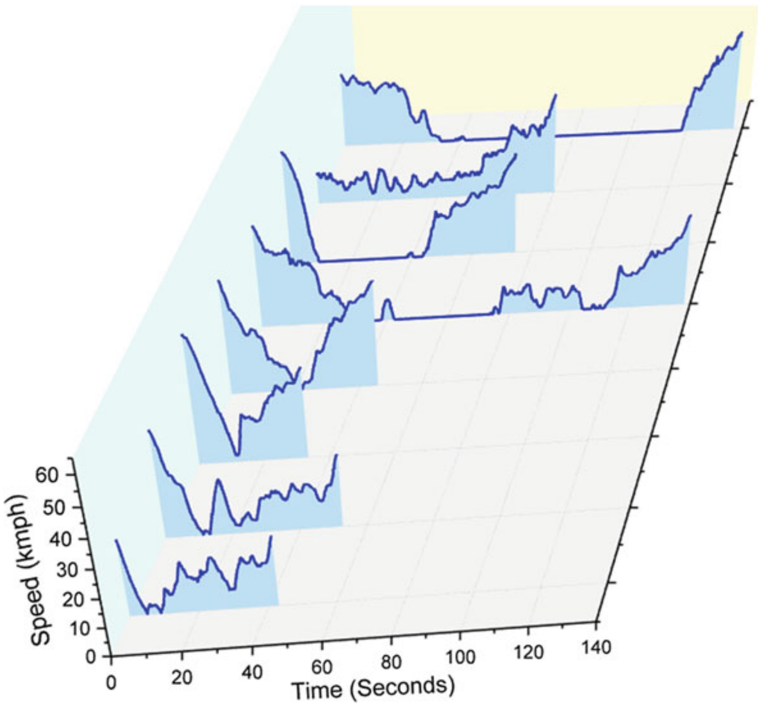

Fig. 2 Speed trajectory of car at intersection

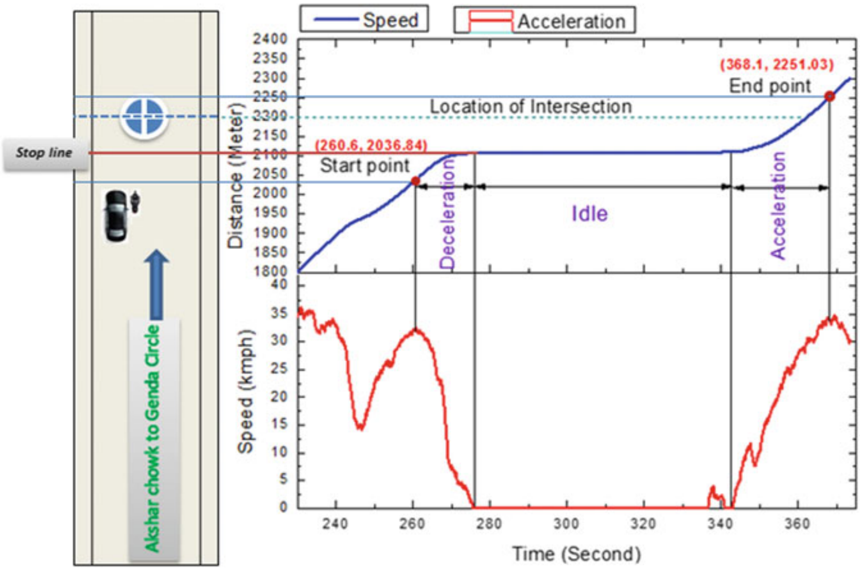

Fig. 3 Location of zone of influence from start to end point

3.3 Zone of Influence at Intersection

Zone of influence consists of deceleration mode followed by idle and acceleration. The location of zone starts when vehicle starts decelerating due to red signal phasing time or congestion at intersection and comes to a stop position or speed falls less than 5 kmph; when green phase turns on, vehicle starts accelerating till its desire speed. Zone of influence is identified by matching speed–time trajectory and distance–time trajectory of vehicles. The zone of influence is high emission zone due to acceleration and deceleration activities, which need attention for traffic and environmental point of view (Chauhan et al. 2018). Figure 3 shows location of zone of influence.

3.4 VISSIM Model Generation

VISSIM can model traffic control and management system on all levels which provides wide scope for simulation of intersections, routes, and networks. It is used for traffic-actuated signal control to describe vehicle arrival influence along the loop between vehicle arrival and signal controller (PTV VISSIM 2015). In present study, Wiedemann 74 car following model is used to simulate traffic at intersection. Wiedemann's traffic flow model is based on free driving, approaching, following and braking, and four driving states of driver (PTV VISSIM 2016; Sajjadi and Kondyli 2017). In VISSIM model, vehicle volume, compositions, desired speed, acceleration

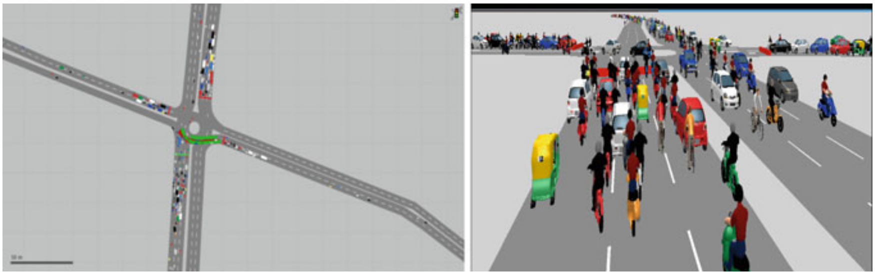

Fig. 4 VISSIM simulation model for signalized intersection

distribution, and signal control are considered as input parameters from real traffic data. Driving behavior is adjusted accordingly to get present traffic scenario. Figure 4 shows VISSIM simulation model for intersection.

4 Results and Discussion

4.1 Speed Trajectory in VISSIM

The speed trajectory of car is extracted from VISSIM simulation output and compared with actual trajectories. The profile is compared in terms of various driving parameters. The speed acceleration frequency distribution is calculated for actual and simulated data. Malhar circle is four-leg-signalized intersection situated at 2400 m distance from the reference point Akshar chowk. In actual data and simulation results, the influence zone begins at 2200 m from upstream side of intersection and finishes at 2500 m at downstream side, stretch of 300 m, whereas in simulation, the stretch of 175 m is found zone of influence. The zone is measured for simulated data to validate the results. In Fig. 5, the speed trajectory in simulation is shown and compared with actual data.

4.2 Speed Acceleration Frequency Matrix

VISSIM is able to generate speed of individual vehicle plying in network (Margreiter et al. 2014; Panis et al. 2006). It is necessary to validate the vehicle's speed generated in simulation with actual speed data. The speed analysis of simulated data is carried out through speed acceleration frequency matrix. The analysis of simulation results in detail involves development of matrix over speed and acceleration range. This matrix calculates deceleration, acceleration, idle, and cruise period in speed trajectories for respective ranges (Kamble et al. 2009). Table 1 shows normalized SAFM for

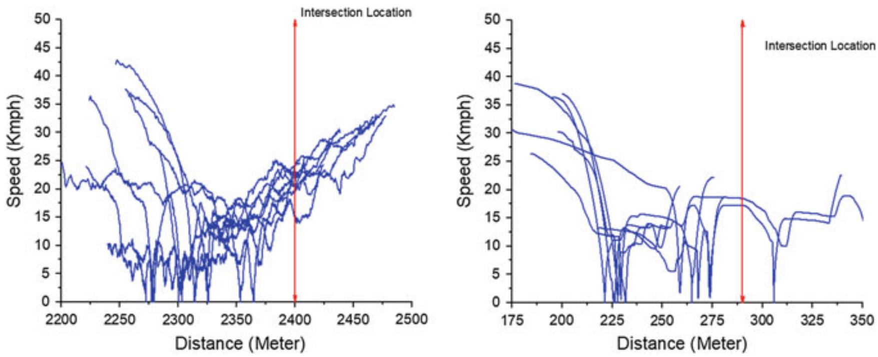

Fig. 5 Speed trajectories for actual data and simulated data

actual data in percentage time acceleration, deceleration, and cruise-idle mode. The following parameters are computed for base data and simulated data (Nesamani and Subramanian 2011).

1. Percentage of time spent in acceleration mode (P_a)—acceleration greater than 0.1 m/s^2
2. Percentage of time spent in deceleration mode (P_d)—acceleration less than −0.1 m/s^2
3. Percentage of time spent in idle mode (P_i)—Speed less than 5 kmph and acceleration −0.1 to 0.1 m/s^2

In Table 1, percentage acceleration value is 48.91%, deceleration 44.78%, idling period 0.44%, and cruise 5.87%. Table 2 shows the normalized SAFM for generated data in simulation. It has percentage acceleration value is 20.87%, deceleration 36.6%, idling period 12.52% and cruise 30.01%. The matrix can be able to analyze the speed and acceleration occurrence in respective range. It shows variation in simulation results over the wide range of acceleration with speed.

It is observed from the table that acceleration range from −5 to 4 m/s^2 are observed in actual data with speed range 0–40 kmph, where as in simulation data, it spreads from −2 to 4 m/s^2 with speed range 0–25 kmph. The idling and cruise period is more in simulated data. In context to above situation, it is mandatory to validate simulation results generated in VISSIM with the base data for model acceptance.

4.3 *Sum Square Difference (SSD) for Evaluating Speed Trajectory Generated in VISSIM*

Speed trajectory of vehicle attained from simulation results is studied and used to generate the zone of influence. VISSIM can able to generate speed trajectories of all vehicles in network. It is important to validate the generated results with the actual

Table 1 SAFM for actual data

Speed	Acceleration(m/s^2)										
	−5 to −4	−4 to −3	−3 to −2	−2 to −1	−1 to −0.1	−0.1 to 0.1	0.1 to 1	1 to 2	2 to 3	3 to 4	Total
0–5	0	0.652	0.87	1.304	3.261	0.435	5	1.957	0.435	0.435	14.34
5–10	0	0	1.087	1.957	3.043	0.652	4.565	1.522	0.87	0	13.69
10–15	0.217	0.217	0.435	1.087	3.261	1.304	2.391	2.391	0.435	0	11.73
15 –20	0	0.217	0.435	1.522	2.826	0.435	4.348	1.522	0.217	0	11.52
20–25	0	0	1.087	2.609	4.783	1.087	6.739	2.174	1.087	0	19.56
25–30	0	0	0.435	2.391	5	1.304	5	1.957	0	0	16.08
30–35	0	0.217	0.435	1.739	2.391	0.87	3.043	2.826	0	0	11.52
35–40	0	0	0.217	0.435	0.652	0.217	0	0	0	0	1.522
Total	0.217	1.304	5	13.04	25.21	6.304	31.08	14.34	3.043	0.435	100

Table 2 SAFM for simulated data

Speed	Acceleration (m/s^2)							
	−2 to −1	−1 to −0.1	−0.1 to 0.1	0.1 to 1	1 to 2	2 to 3	3 to 4	Total
0–5	0.803	2.889	12.52	0.482	0.321	0.482	0	17.496
5–10	1.445	0	0	0	0	0.321	0.482	2.247
10–15	1.124	4.173	0.482	0.803	1.124	0.642	0	8.347
15–20	2.087	9.631	12.841	4.334	1.766	0.321	0	30.979
20–25	0.482	13.965	16.693	8.347	1.445	0	0	40.931
Total	5.939	30.658	42.536	13.965	4.655	1.766	0.482	100

data. In current study, the generated trajectories are compared with the actual data of car trajectory. The data has been compared through sum square difference of the profile. The profile which has least value of difference is closely matches with the actual data. It is considered as most representative profile of speed.

$$\mathrm{SSD} = \sum_{i=1}^{Ns} \sum_{j=1}^{Na} (P_{ij} - Q_{ij})^2 \tag{1}$$

In Eq. 1, Ns is the number of speed classes, Na is the number of acceleration classes, P_{ij} is the ijth entry of the SAPD of the candidate cycle, and Q_{ij} is the ijth entry of the speed-acceleration frequency of the overall driving speed profiles.

Driving parameters: percentage time in idle, percentage time in acceleration, and percentage time in deceleration of actual profile and simulated profile are compared by calculating sum square difference value from speed acceleration frequency matrix. In Table 3, parameters of actual cycle and generated cycles are shown. It is observed from the results that sum square difference for normalized matrix of actual data and simulated data with values less than 0.3 gives close match with the observed data. The SSD values more than 0.4 have deviation in acceleration, deceleration, and idle values. The cycle consisting of less deviation is accepted for further analysis. It is observed from the table, cycle 2, 3, 7, and 10 have low SSD value and their parameters significantly match with the base data.

5 Conclusion

In present study, traffic simulation modelling is carried out in VISSIM software. The speed trajectories generated from simulation are compared with actual data. Zone of intersection influence is recognized from deceleration activity to idle and acceleration activity of vehicle. Important driving parameters such as acceleration, deceleration, and idle time are calculated for zone of influence. Speed trajectory from simulation results is compared with the base data and selected on basis of least sum square

Table 3 Parameters of actual cycle and simulated cycles

Parameters	% Acceleration	% Deceleration	% Idle	SSD
Base cycle	48.91	44.78	0.44	
Simulated cycle–1	7.34	7.99	84.5	0.778
Simulated cycle–2	*31.8*	*34.94*	*20.5*	*0.090*
Simulated cycle–3	*30.35*	*29.12*	*20.37*	*0.173*
Simulated cycle–4	16.95	14.35	63.66	0.482
Simulated cycle–5	7.25	6.94	85.65	0.796
Simulated cycle–6	22.32	14.88	46.37	0.417
Simulated cycle–7	*20.87*	*36.6*	*12.52*	*0.181*
Simulated cycle–8	23.65	20.86	42.58	0.308
Simulated cycle–9	23.11	19.1	53.27	0.327
Simulated cycle–10	*25.29*	*25.73*	*19.56*	*0.244*
Simulated cycle–11	16.22	20.04	51.34	0.426

difference of speed–acceleration classes. It is observed that parameters of generated trajectories in VISSIM with lower sum square difference are closely matched with real data. These profiles are representative of traffic data. The results are summarized from the vehicle operation second by second bases from VISSIM model to obtain precise speed trajectory. The speed trajectories in base data are obtained with 48.91% acceleration, 44.78% deceleration, and only 0.44% idling period, whereas in simulation idling period is more than observed data. In microscopic traffic simulation modelling, driver's behavior depends on the vehicles running ahead as well as in adjacent lane. Specific lateral distance is required to run the model and to achieve real traffic characteristics. The zone of influence is recognized by representative speed trajectories in VISSIM. It gives the stretch of high pollution due to vehicle exhaust emissions near intersection. VISSIM can able to modify signal phasing time and vehicle compositions and shows its effect on length of zone. The more length, more pollution in intersection vicinity can be observed. To reduce pollution, the acceleration and deceleration activities should be controlled through traffic management and signalized system.

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Estimating the Impact of Flyover on Vehicle Delay, Fuel Consumption, and Emissions—A Case Study

Lenjisa Bedada, Mukti Advani, Satish Chandra, and Jayesh Juremalani

Abstract Road Over Bridges (ROB or flyover as it is popularly called in India) are considered as a solution to reduce the delay at an oversaturated at-grade signalized intersection. However, this provision of flyover remains a short-term solution, as it is seen that flyover also reaches to the same congestion level after a few years due to increased personalized vehicular traffic. To analyze the effect of a flyover over the years, vehicular delay, fuel consumption, and emission for the range of traffic volume are estimated. One intersection, Bhikaji Cama Place intersection in New Delhi, is considered as a case study. This is a four-legged signal-controlled intersection. Three scenarios are considered. Scenario A is the existing condition of flyover and signal phasing plan, Scenario B is with flyover and with proposed signal phasing plan, and Scenario C is without the flyover and proposed signal phasing plan. Scenario B includes comparison of alternate signal phasing plans to obtain the best signal design for existing traffic to minimize the delay. Delay estimation, fuel consumption, and emission estimation have been carried out by existing methods for all the three scenarios.

Keywords Flyover · Traffic · Delay · Sustainability

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1 Introduction

Intersections are integral part of a road network in a city and handle higher and complex traffic with more number of movements compared to mid-blocks. Traffic control at intersections is required for safe movement and it can be either in the form of a roundabout or a traffic signal. The separation of through movement on one of the roads by constructing flyover is considered as an option for reducing delay by transport planners. Researchers have discussed benefits and cost of constructing a flyover as well as demolishing it. Arguments toward justifying a flyover construction is primarily based on delay reduction for existing and future traffic scenarios. On the other hand, removal of flyovers is based on experiences and theoretical understanding regarding sustainability. A quantifiable approach is required to check the period of flyovers' life span for which it results in reduced delay and emissions. It has been commonly observed that every newly constructed flyover resolves traffic congestion issue at specific location for initial few year and then after, due to growth in number of vehicles, it itself faces congestion over it. This congestion is resulting in delay primarily depends on traffic volume at intersection and width of flyover and at-grade roads. However, a scientific approach is required to calculate the amount of traffic a flyover can handle without unacceptable delay, fuel consumption, and emission generation.

2 Experiences with Flyovers

The first flyover in Bangladesh was commissioned in year 2004 at Mohakhali intersection (Taleb and Majumder 2012). This is a four-lane, 1.12 km long, and 17.9 m wide carriage way to reduce the traffic congestion. Mamun et al. (2016) observed the performance of the Mohakhali flyover by comparing the network traffic with and without flyover. First, the analysis of existing condition (with flyover) was simulated in VISSIM to obtain the density, flow, speed, and delay and then, these performance measures were obtained for the network without flyover using the same traffic load. It was concluded that the flyover should be extended and few additional links should be constructed to handle traffic congestion more efficiently at the study area.

Marisamynathan and Lakshmi (2016) studied the performance level of signalized intersections, based on traffic and geometric characteristics. Traffic field data were collected for Retteri-signalized intersection, Chennai, India, for 12 h duration on weekdays. Vehicle delay was estimated by the classical Webster's formula and level of service (LOS) was identified and a flyover was proposed as an immediate remedial measure at the intersection. However, authors have mentioned that projected traffic will reach to its maximum capacity in next 4 years.

Panchal et al. (2017) investigated feasibility of flyover at intersection on ring road in Vadodara city. It was revealed that at a highly congested intersection, for better and efficient transportation, flyover may be provided to satisfy the needs of current vehicular traffic. Yahampath et al. (2017) have mentioned that the Dehiwala intersection in Sri Lanka was replaced by a flyover in order to eliminate the traffic congestion. However, congestion and accident trends have not reduced even after construction of flyover. Accident data analysis, speed, and travel time surveys were used to compare the existing conditions and before construction of the flyover. Outcome of the study was that expected benefits of reduced delay have not been achieved from the flyover project due to induced traffic.

Salatoom and Taneerananon (2015) evaluated a flyover built along the intercity highway to the Hatyai airport in Thailand. This flyover was constructed to enhance the capacity of the intersection and to reduce the vehicle delay and queue length. SIDRA software was used to compare the solutions before flyover (in the year 2009) and after construction of the flyover in terms of delay and accidents. Authors estimated an immediate reduction of 33.8% delay for the traffic moving below the flyover. The vehicle delay before construction of the flyover was 32,116 man-minutes per year in 2009 which is expected to be 27,990 man-minutes in 2021.

Goyal et al. (2008) assessed environmental benefits of flyover construction over signalized junctions in Nagpur city, India. A flyover was constructed throughout four signalized intersections. Authors analyzed the change in route choice and observed that approximately 35% of the total traffic was diverted to flyover. This was resulted in a reduction of 35% in total vehicular emission.

Anwari et al. (2016) studied operational effectiveness of eight flyovers in Dhaka city. These flyovers were built to reduce traffic congestion and accidents. At-grade vehicle speed was found to be 3.39 times slower than the vehicle speed on flyover. Analysis of field data revealed that 67% of traffic flow was at-grade and 49% of this traffic was of non-motorized vehicles.

Peiris (2011) studied flyover as traffic management measure at Dehiwala junction in Sri Lanka. This flyover was constructed in 2009 to eliminate the traffic congestion, delays, accidents, and pollution. However, it was not found to be effective due to sudden drop in speed ahead of flyover, which resulted in more number of accidents.

Many researchers have worked in the area of quantification of congestion at at-grade intersections. However, studies related to such quantification for grade-separated intersection are very limited. Maitra et al. (2004) modelled traffic impact of flyover at an urban intersection under mixed traffic environment in Kolkata, India, where a flyover was being constructed, and another adjacent intersection in close proximity was considered for their studies. After they conducted necessary surveys, data analysis, simulated the model and validated the model, and they have reported that, a comprehensive planning approach is missing in most of the cases. The locations for flyovers usually have been decided based on observed level of congestion for motorised vehicles. Though, this should be based on holistic approach considering the imapct on all other non-motorised modes as well. Flyovers do shift the problem by location or sometimes by time instead of solving it. A more detailed analytical planning approach ais required is required to check the impact of a flyover.

The feasibility of construction of a flyover, Kumharon Ka Bhatta in Udaipur was evaluated in 2014 (ICLEI 2014). The scenario of flyover was compared with at-grade intersection improvements for reducing traffic delays. It was estimated that after consideration of flyover, almost 47% of total traffic at the intersection can pass uninterrupted. However, delay was not included as a performance measure in before and after scenario.

A study on Economic Assessment of a flyover located in Rajkot (India) was carried out by Chhatbar and Shinkar (2016). Estimated total benefit from the flyover was about Rs. 50.4 million per year through savings in fuel consumption and reduced delay. The total cost of construction of flyover mentioned in study is Rs. 255 million. However, there is no mention about the base and horizon year of traffic volume and delay studies.

Most of the literature available with respect to flyovers is in terms of feasibility report or delay estimation for a short period of time. Feasibility reports most commonly include only traffic volume count (vehicle carrying capacity) for base and horizon year to design road geometry. Few studies have included vehicular delay in base year but ignored the delay in horizon year. Studies considering the flyover as a solution have mentioned it as a immediate relief providing solution. All benefits of flyover in terms of reduced travel time and emission are applicable to a certain level of traffic which is not mentioned in any of the existing studies. It has been seen that there is no study covering the impact of a flyover for a long period of time, i.e., life span of the flyover. Though, many researchers have agreed that flyover is a short-term solution or a location specific solution, a study is required to quantify the duration for which flyover works as a solution and what happens beyond that period.

According to Siegel (2007), In America, the first generation of freeways is approaching the end of their lifespan and removal is the best alternative—and the only alternative that helps deal with looming environmental problems such as global warming. Cervero (2007) mentions that despite worsening traffic congestion, a number of American cities have torn down or are in the midst of demolishing elevated structures in favor of at-grade boulevards and arterials with far less traffic carrying capacities. In Great Britain, where there is a very active anti-freeway movement, transportation planners are no longer allowed to count reduced travel time as a benefit of building a new freeway (DoT 1996). The Department of Transport has adopted a guidance document which stipulates that the cost–benefit studies on new freeways must assume that elasticity of demand may be as high as 1.0 with respect to speed—which means that the average trip length increases as much as speed increases, so building freeways and increasing speeds just lengthen trips and does not save any time.

Navlakha (2016) studied the flyover phenomenon to reduce traffic congestion. Pimpri-Chinchwad New Town Development Authority (PCNTDA) had proposed two flyovers at Kalewadi Phata in Pune with an estimated cost of Rs. 189.2 million. The total length of the two flyovers to be constructed parallel to each other was 730 m. After field studies, it was found that flyovers will not be able to solve the congestion issue in spite of spending huge money. It was mentioned that building flyovers is a vehicle centric approach oriented on demand than the sustainable transport.

Based on various reports published by Preservation Institute, Siegel (2016) concludes that when freeways run through downtowns, there are huge economic benefits to tearing them down. Milwaukee spent $25 million to demolish the 1-mile-long Park East freeway, while it would have cost $100 million to rebuild that 30-year-old freeway. Removing the freeway opened 26 acres of land for new development, including the freeway right of way and surface parking lots around it, which have already attracted over $300 investment. San Francisco increased nearby property values by 300 percent by tearing down the Embarcadero Freeway and opening up the waterfront was opened up, stimulating the development of entire new neighborhoods. San Francisco increased nearby property values by 300 percent by tearing down the Embarcadero Freeway and opening up the waterfront was opened up, stimulating the development of entire new neighborhoods.

Seoul has removed the Cheonggye freeway and restored the river. Seoul has built bus facilities to replace the freeway capacity with the aim of reducing usage of personalized motorized vehicle usage.

DOT (2018) Boston, Massachusetts had a traffic problem due to traffic congestion in the city. An elevated six-lane highway called the Central Artery which was constructed in 1959 and carried 75,000 vehicles per day. Later in 1990s, it was carrying almost 200,000 vehicles per day. This was resulted in a poor level of service and 10–16 h long traffic jam situations. The accident rate on the deteriorating elevated highway was four times the national average for urban interstates. Table 1 presents the summary of flyovers demolished in the different parts of the world in last three decades.

3 Study Focus and Objectives

From the literature, it has been understood that the impact of a flyover on delay, fuel consumption, and emission primarily depends on the amount of traffic it is carrying. Therefore, it is required to study impact of traffic on flyover's performance. To analyze flyover's life span with respect to its impact on vehicular delay, fuel consumption and emission levels for the range of traffic volume, alternate scenarios have been compared for selected intersection, i.e., Bhikaji Cama Place intersection, New Delhi, India.

The objectives of present study is to compare vehicular delay at Bhikaji Cama Place intersection for three scenarios covering alternate signal phasing and presence/absence of flyover:

(A): Existing conditions (with flyover and with existing signal phasing plan)
(B): with flyover and with proposed signal phasing plan
(C): without flyover with proposed signal phasing plan.

Table 1 Details of flyovers demolished world over

Flyover already demolished	Year of construction	Year of demolishment	Budget spent on construction.	Budget spent on demolishment and restoration
San Francisco, CA: Embarcadero Freeway	1959	1991	–	$15 million (restoration)
San Francisco, CA: Central Freeway	1950	1992	–	–
Milwaukee, WI: Park East Freeway	1970	1999	$100 million	$25 million
Toronto, Ontario: Gardiner Expressway	1966	2001	$200,000 million	$39 million
Niagara Falls, NY: Robert Moses Parkway	1964	2017	$42 million (construction)	–
Boston, MA: Interstate 93 (moved underground)	1959	1995	$1.3 billion (reconstruction/restoration)	$500 million (cost estimating due to congestion and idling)
Paris, France: Pompidou Expressway	1967	2002	$50 million (restoration)	1.5 million Euros
Seoul, South Korea: Cheonggye Freeway	1968	2003	US$281 million (restoration)	–

Source Different websites

4 Broad Methodology

Figure 1 presents the study methodology. Present study is broadly divided into three scenarios. Keeping objectives in focus, three scenarios have been developed. For each scenario, delay estimation has been carried out for the range of traffic volumes. Also, fuel consumption and emission have been estimated for all three scenarios.

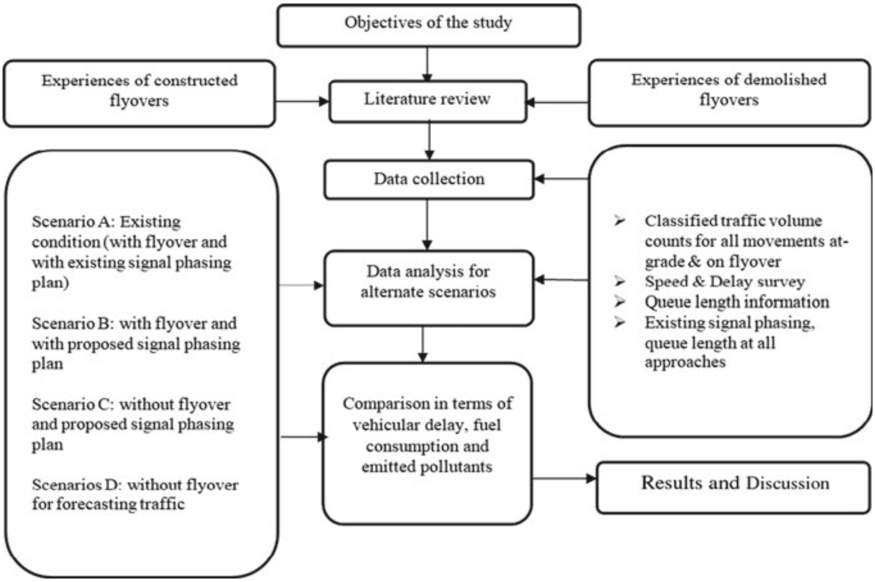

Fig. 1 Study methodology

5 Data Collection and Traffic Characteristics

Intersection considered for this study is a four armed intersection in South Delhi, India, as shown in Fig. 2. It is formed by crossing of Mahatma Gandhi Road (Ring Road) and Africa Avenue Marg. The Africa Avenue Marg links the South Delhi and

Fig. 2 Bhikaji Cama Place intersection

Gurugram with Chanakya Puri, Sarojini Nagar and Cannaught Place. Ring Road at this intersection is having a flyover, which facilitates signal free straight movement of vehicles coming from All India Institute of Medical Sciences (AIIMS) and Dhaula Kuan.

Classified traffic volume counts were collected for all the movements through videography/manually as presented in Table 2.

Out of total traffic at intersection, 1.03 lakh vehicles use at-grade intersection and remaining 1.46 lakh vehicles use flyover daily. Table 3 presents modal split of at-grade and flyover traffic. After deducting free left-turning traffic from at-grade traffic, 61,122 PCUs are handled by the signal.

For all the three scenarios given in Table 2, signal design and vehicle delay have been estimated based on methods developed by Mathew (2007) and Indo-HCM (2017), respectively. Within scenario (B), three different signal phasing plans as shown in Fig. 3 have been compared to obtain the best one for existing traffic and intersection.

Delay calculation was made for all scenarios for the range of traffic volume at intersection. Traffic volume ranges coded in this study by “levels” are presented in

Table 2 Classified traffic volume counts at Bhikaji Cama intersection (24 h data)

Location	Direction	Car	MTW	Auto	Bus	Trucks	NMV	Total
Approach “A”	Straight	1152	4484	1036	811	452	125	18,060
Munirka	Right turn	8150	3738	498	262	236	167	13,051
	Left turn	5736	2498	485	37	451	123	9330
Approach “C”	Straight	1158	779	174	14	124	262	2511
AIIMS	Right turn	2786	1275	255	160	154	37	4667
	Left turn	5139	2790	597	348	230	140	9244
Approach “B”	Straight	11,747	2275	402	324	192	34	14,974
Emabssy	Right turn	4797	1840	341	110	141	37	7266
	Left turn	5152	2892	832	560	202	44	9682
Approach “D”	Straight	1016	501	122	69	188	109	2005
Dhaula Kuan	Right turn	4937	2776	620	294	690	60	9377
	Left turn	1486	888	199	145	133	73	2924
From ‘C’ to ‘D’	Flyover straight	41,976	17,437	2572	2059	3763	11	67,818
From ‘D’ to ‘C’ on	Straight flyover	44,764	24,099	2418	2913	4845	13	79,052
Total		149,995	68,272	10,550	8106	11,801	1235	249,959

Table 3 Modal split at-grade and on flyover

Mode	At-grade (%)	On flyover (%)
MTW	12	5
Auto rickshaw	3	2
Car	73	70
BUS	6	12
LCV	3	10
HCV	2	1
MAV	1	0.1

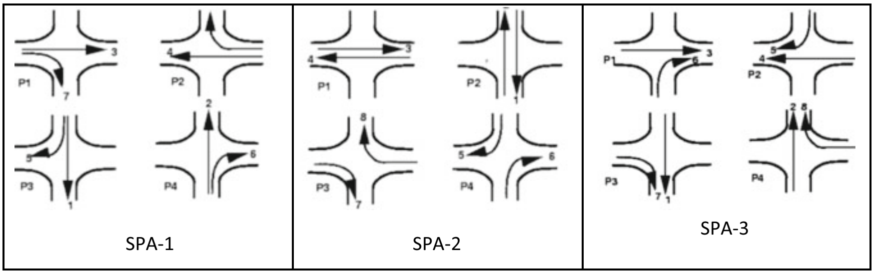

Fig. 3 Signal phasing alternatives (SPA) considered within scenario (B)

Table 4. This value of traffic volume is only for two straight movements (straight movements on flyover and at-grade, 6-lane divided).

Figure 4 shows the average delay for the levels of traffic volume moving at-grade and on flyover.

As shown in Fig. 5, the minimum delay for vehicles moving at-grade is ~80 s and it increases with the increased traffic volume. At flyover, the minimum delay is zero, i.e., at free flow speed, there is no delay due to surrounding traffic. For the traffic flow of 3000 PCU/hour (between level 4 and 5) delay at-grade and on flyover becomes the same, i.e., 220 s/veh and beyond this traffic level, moving on flyover results in higher delay compared to moving at-grade. Therefore, 3500 PCU/hour is the critical traffic beyond which flyover does not work better than at-grade traffic. At

Table 4 Levels for traffic volume considered in this study

Level	Traffic volume (PCU/h)
Level 1	100
Level 2	500
Level 3	1000
Level 4	2000
Level 5	4000
Level 6	5000

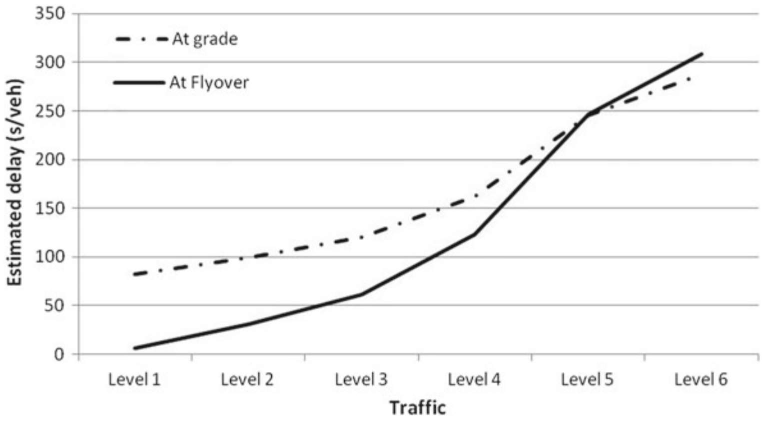

Fig. 4 Estimated delay at-grade and on flyover for different levels of traffic

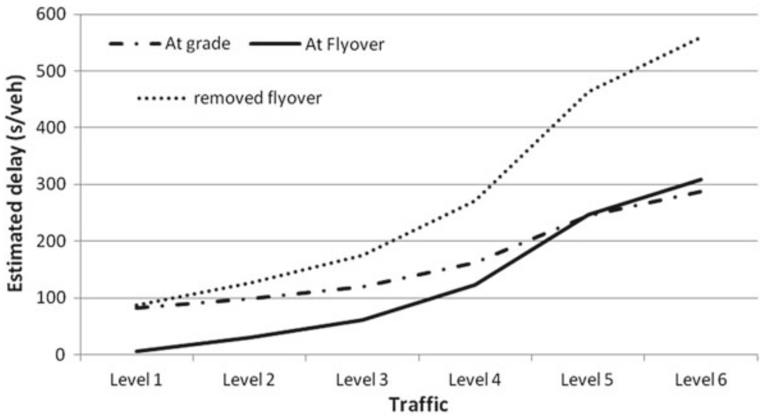

Fig. 5 Delay for different levels of traffic at-grade, on flyover, and at-grade in case flyover is demolished

this stage, if flyover is removed, then the sum of at-grade and flyover traffic will be moving at-grade which will increase the average delay as shown in Fig. 4.

As shown in Fig. 5, the delay will always be higher for similar modal split conditions in case the flyover is removed.

Apart from delay, other important parameters to compare alternatives are fuel consumption and emissions emitted by vehicles. Fuel consumption for all vehicles facing delay has been calculated based on fuel efficiency factors presented in Table 5.

Figure 6 presents the fuel consumption for alternate scenarios and for the different levels of traffic volume. This indicates highest fuel consumption in case flyover is demolished.

Table 5 Adopted fuel efficiency factors

Vehicle type	Fuel consumption at idling (mg/s)
Motorcycle	0.0389
Three wheeler	0.067
Car	0.167
Bus	0.25
Light commercial vehicle	0.192
Heavy commercial vehicles	0.255
Multi-axle vehicles	0.344

Source Gangopadhyay et al. (2013)

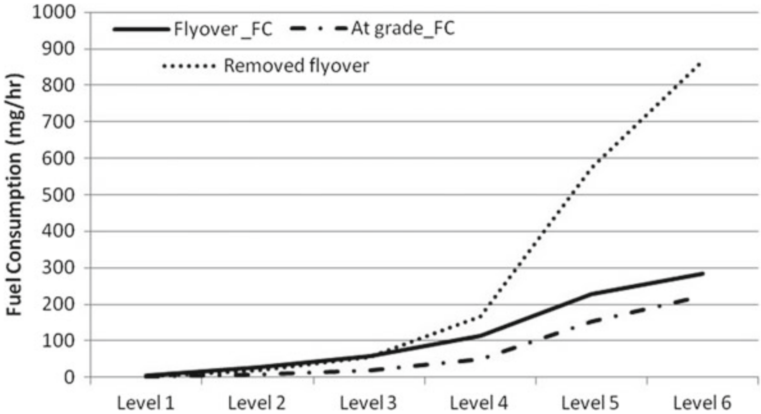

Fig. 6 Fuel consumption for the range of traffic volume and for alternate scenarios

However, if all vehicles moving at-grade and facing delay due to straffic signal, do switch off their vehicle engine, fuel consumption reduces drastically as shown in Fig. 7.

Similarly, emissions for alternate scenarios have been calculated based on emission factors presented in Table 6.

For existing traffic and delay (Scenario A) estimated CO and CO_2 emitted from vehicles is 9.84 kg and 89.66 kg during morning peak hour, respectively. For scenario B, this can be reduced to 2.89 kg and 27.40 kg. Emission for scenario C is not carried out due to unavailability of speed-based emission factors which are essential for estimating the pollutants emitted from vehicles when they move at different speeds.

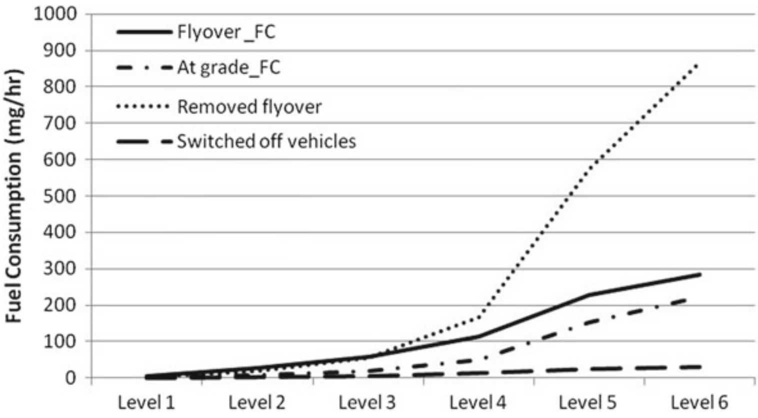

Fig. 7 Fuel consumption reduces for 'switching off engines' behavior

Table 6 Net calorific value (NCV) and emission factor

Fuel type	NCV (TJ/t)	Emission factor (kg/TJ)	
		CO_2	CO
Petrol	44.8	69,300	8000
Diesel	43.33	74,100	1000
CNG	48.00	56,100	400
LPG	47.00	63,100	400

Source Revised downwards by CRRI as per recommendation of IPCC 2006 guidelines (Singh et al. 2008)

6 Conclusions

When traffic volume on the major street of a signal-controlled intersection becomes high, a flyover or complete interchange is usually constructed to relieve the congestion level at particular location. However, this solution remains short term only due to growing number of private vehicles. This growth has the major impact on effective usage of a flyover. After a critical volume, even flyover starts facing congestion-related problems. At selected intersection, i.e., Bhikaji cama place, optimal signal phasing plan is resulted in reduced delay. Present study based on one intersection reveals that 3500 PCU/hour is the critical traffic for a 6-lane divided road beyond which flyover does not work better than at-grade intersection.

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An Integer Programming Formulation for Optimal Mode-Specific Route Assignment

Aathira K. Das and Bhargava Rama Chilukuri

Abstract Traffic in developing countries has a heterogeneous vehicular mix that uses all the network links and lacks lane discipline. Modeling and controlling such a mixed traffic system are challenging since most of the well-established models were developed for homogeneous traffic. It is hypothesized that segregating the mixed traffic by assigning a unique mode to each link will enhance system capacity. Towards achieving it, this paper proposes optimal mode-route assignment formulations with the objective of minimizing the total system travel time. However, perfect segregation is not always possible since the solution depends on the network topography. A viable solution is to make some links multi-modal. Another formulation is also presented in this paper to address this issue. Both the formulations are demonstrated using sample networks. Linear and nonlinear integer mathematical programming methods are used to explore the qualitative characteristics of optimal mode-route assignment using the single-path routing method. The results indicate that, in the worst case where perfect segregation is not possible, proposed formulation II can identify network with the least number of the multi-modal links. This research will help to develop effective strategies to model, control, and enhance the safety of mixed traffic networks.

Keywords Mode-route assignment · Optimization · Multi-modal networks

1 Introduction

The rapid urbanization of cities, coupled with population growth, has resulted in increased trip rates, traffic volumes, vehicular miles, vehicular delay, and traffic congestion in major metropolitan cities across the globe. Traffic congestion occurs whenever traffic demand exceeds the road capacity. However, capacity expansion

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through physical infrastructure addition is expensive and is not always possible. Therefore, soft strategies are required to address the issue of traffic congestion. Traffic in developing countries is highly heterogeneous, with the mix comprising of vehicles with varied sizes and engine kinetics. Moreover, all the network links serve all modes of transport, unless there is any mode restriction enforced on any particular link. A mixture of vehicles causes uncontrollable chaos on the roads. The flow pattern throughout an urban network is a result of the decisions of users of the system to reduce the disutility related to their travel. Commuters have their own choices pertaining to the trips they make, comprising the choice of taking a trip, the freedom to choose a mode of transport, the choice of deciding on the OD pair, and the choice of route spanning between the chosen OD pair. The choices they make also result, in part, in how they transfer across modes of their choice and where the transfer points are located. All these decisions, when aggregated, result in a certain flow pattern through the network. Therefore, determining the flow patterns with certainty will be difficult in different situations. Modeling such a system is challenging due to the variations in the maneuverability and bounded velocities and accelerations. Additionally, the lack of lane discipline increases the complexity owing to a large number of inter-modal interactions brought about by it. This may also result in a reduced capacity of the roadway due to a large number of unutilized white spaces in the stream and critical safety issues. Studies also show that higher rates of accidents are observed in mixed traffic than in homogeneous traffic (Forkenbrock and March 2005; Middleton and Lord 2005).

It is hypothesized that segregating the traffic mix will reduce the adverse effects of the mixed traffic. Moreover, it will reduce intra-modal interactions and unutilized white spaces, thereby increasing the network capacity. Homogeneous traffic attracted much attention as segregating modes may increase efficiency and enhance stability and safety. It can overcome all the adverse effects of heterogeneity. Traffic flow models and optimal control strategies are well developed for homogeneous traffic. Segregation of traffic has many physical constraints. The topography of an existing city may not allow separate routes for each vehicle class in a network. However, isolation of a particular vehicle type from mixed traffic has been implemented as part of many successful studies. It has potential benefits similar to the segregation of mixed traffic. Segregating light and heavy vehicles on existing roads could facilitate a more efficient flow of traffic. Similar considerations ascertain whether segregating cars and trucks would reduce accidents. Merits of separating cars and trucks have been a topic of debate since the early days of motoring (De Palma et al. 2008). De Palma et al. analyzed the prospective advantages of segregating cars and trucks in mitigating congestion, accidents, etc. Recent proposals in the USA have focused on establishing truck-only lanes. Berglas et al. (1984) reinforced the notion that providing separate lanes for cars and buses would reduce delays and increase throughput. Moreover, surveys indicate that light-vehicle drivers hate trucks and would be even willing to pay to avoid them (Arnott et al. 1998). Many studies suggest the huge potential of desired outcomes like reduced delays and increased capacity from providing dedicated bus lanes in the city centers of developing countries. Thus, the segregation of traffic can be implemented in different ways. To achieve this, one needs to determine the optimal unique mode-link assignment. To the authors' knowledge, there is no literature that studied this problem. However, researchers have studied similar problems in other areas. One of the similar problems is the multi-commodity flow problem, in which individual commodities share common arc capacities. The intention is to move the goods through the network at minimum cost without exceeding arc capacities (Ahuja et al. 1993). The multi-commodity-capacitated network design problem is an NP-hard discrete optimization problem (Magnanti and Wong 1984). Methods including partitioning, resource-directive decomposition, and price-directive decomposition have been popularly used to solve the problems. Recently proposed heuristic methodologies for multi-commodity network design problems use a trajectory-based algorithm (Ghamlouche et al. 2003) to select the arcs of the network, following which optimizer is used to solve the linear programming problem. Gendron et al. (2016) proposed an iterative linear programming-based heuristic for solving multi-commodity fixed-charge network design problem, which was compared with cycle-based evolutionary algorithm. Another similar problem is the design of public transport networks where optimal routes (set of links) are determined for a particular mode (here, transit).

While each of these papers studied a similar problem, their formulation cannot be directly applied to this problem. This is because the formulation has to be constrained in such a way that the optimal solution leads to a network with all links to be mode-specific or a combination of mode-specific and multi-modal links unlike in case of multi-commodity flow problem in which individual commodities share common arc capacities.

The formulations proposed in this paper aim to provide an elementary framework for optimal mode-route assignment for small-scaled networks. The formulations are constrained in such a way that the optimal solution leads to a network with all links to be mode-specific or a combination of mode-specific and multi-modal links. A comparison is made between two, one which permits cent percent segregation (formulation I) and one which may have both homogeneous and mixed traffic links (formulation II).

2 Problem Formulation

The context of the study in this paper is a road network represented as a graph featured with multiple origins and destinations. Each origin-destination (OD) pair is connected by more than one route. K modes are required to be served between each OD pair, for example, two-wheelers, three-wheelers, four-wheelers, etc. The links have a mode-dependent cost. And, there is a fixed demand for each mode at each destination.

Let c_{ij}^k be the unit flow cost on link (i, j) if the link is assigned with vehicle type k. Let b_{ij}^k be the supply or demand of vehicle type k at node i, and let M be the maximum flow capacity on network. Then, the problem may be modelled as an integer linear program (ILP) using continuous flow variables x_{ij}^k that represent the amount of flow on each link (i, j) for each vehicle type k, and 0–1 design variables y_{ij}^{k} that indicates if the link (i, j) is used by the vehicle type k or not. y_{ij}^{k} is 1 if vehicle type k is present on link (i, j) and 0 otherwise. Thus, the problem is to choose which arcs should be present in the routes and to assign vehicle type for each arc so as to minimize total system travel time while observing flow constraints. The Big-M constraint in the formulation will ensure that the link cost is incurred only if arc carries positive flow of a particular mode type.

2.1 Assumptions

The current study is a simplified version of the real-world problem with the following assumptions:

1. The origin-destination (OD) travel demand is given and remains static during the analysis.
2. Only two mode types are considered and are referred to as $m1$ and $m2$.
3. Network structure, link characteristics, and cost of each link (i, j) are known.
4. All links of the network have infinite capacity.

2.2 Formulation I

The problem is formulated as a linear programming problem to evaluate the mode-specific flow on each link. The objective is to find the optimal flow on the network minimizing the total system travel time while observing flow constraints in consequence of which optimal solution gives mode-specific routes between each OD pair.

The objective function is as given in Eq. 1.

$$\text{Min } Z = \sum_{(i,j)} \sum_{k} c_{ij}^{k} x_{ij}^{k} \tag{1}$$

Subject to

$$\sum_{j} x_{ij}^{k} - \sum_{i} x_{ij}^{k} = \begin{cases} b_i^k, & i \in s \\ -b_i^k, & i \in t \\ 0, & \text{otherwise} \end{cases} \quad \forall (i, j) \tag{2}$$

$$x_{ij}^{k} \leq M \times y_{ij}^{k} \quad \forall (i, j) \tag{3}$$

$$\sum_{k} y_{ij}^{k} \leq 1 \quad \forall (i, j) \tag{4}$$

$$0 \le y_{ij}^k \le 1 \quad \forall (i, j) \tag{5}$$

$$x_{ij}^k \ge 0 \quad \forall (i, j) \tag{6}$$

Here, Z is the objective function. Equation (2) is the flow conservation constraint for each node $i \in N$ and each vehicle type. Equation (3) gives the Big-M constraint, which links the decision variable with the binary variable. Equation (4) ensures that only one vehicle type is assigned to a link (i, j) and Eq. (5) defines the binary variable y_{ij}^k. Equation (6) is the non-negativity constraint for link flows.

2.3 Formulation II

Formulation I gives a cent percent modal segregation. In cases where there is no feasible solution possible from the above-mentioned formulation, formulation II is proposed. Thus, here, the problem is formulated as a nonlinear programming problem. The objective here is to find the optimal flow on the network minimizing the total system travel time with minimum multiple-mode links.

The objective function Y is as given in Eq. 1.

$$\text{Min } Y = \sum_{(i,j)} \sum_{k} c_{ij}^k \, x_{ij}^k \, y_{ij}^k \tag{7}$$

Subject to

$$\sum_j x_{ij}^k - \sum_i x_{ij}^k = \begin{cases} b_i^k, & i \in s \\ -b_i^k, & i \in t \\ 0, & \text{otherwise} \end{cases} \quad \forall (i, j) \tag{8}$$

$$x_{ij}^k \le M \times y_{ij}^k \quad \forall (i, j) \tag{9}$$

$$\sum_k y_{ij}^k \le 2 \quad \forall (i, j) \tag{10}$$

$$0 \le y_{ij}^k \le 1 \quad \forall (i, j) \tag{11}$$

$$x_{ij}^k \ge 0 \quad \forall (i, j) \tag{12}$$

The only difference here from the formulation I is in constraint shown in (10). This constraint ensures that a link can have a positive flow of either one mode or both vehicle types. A higher cost is considered for the link, which will carry mixed

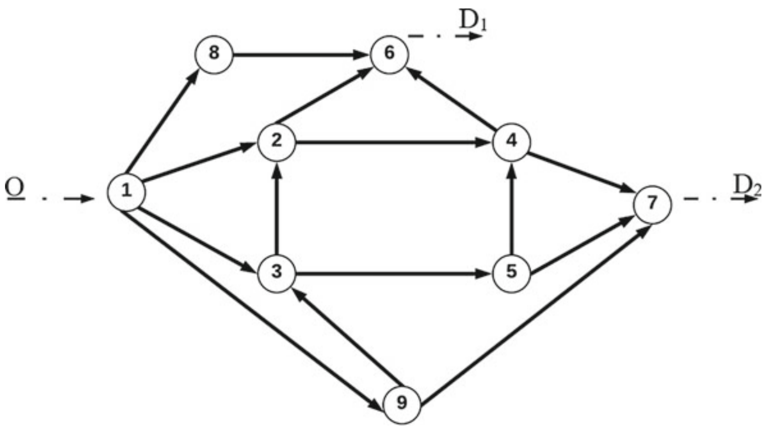

Fig. 1 Sample network for formulation I

traffic. Thus, minimizing the cost ensures that the least number of links are assigned with multiple modes.

3 Application

In this section, the proposed approaches are applied to a example urban network with nine nodes. The network has travel demand from node 1 (origin) to nodes 6 and 7 (destinations), as shown in Fig. 1. Two different networks with same demand matrix are used to demonstrate the two formulations. For simplicity in the examples, a small network with one-way links is considered. The demand matrix of the two classes of vehicle separately in the peak hour in veh/h of the day is given below as (13) and (14) for modes $m1$ and mode $m2$, respectively:

$$D_{m1} = \begin{array}{ccc} & D_1 & D_2 \\ O & 100 & 100 \end{array} \tag{13}$$

$$D_{m2} = \begin{array}{ccc} & D_1 & D_2 \\ O & 80 & 220 \end{array} \tag{14}$$

Table 1 Solution of Sample network for formulation I

Link	c_{ij}^1	c_{ij}^2	x_{ij}^1 (veh/h)	x_{ij}^2 (veh/h)
(1,8)	12	22	100	0
(1,2)	8	14	0	80
(1,3)	6	15	100	0
(1,9)	9	5	0	220
(8,6)	15	32	100	0
(2,8)	10	22	0	0
(2,6)	5	12	0	80
(2,4)	7	19	100	0
(3,2)	11	26	100	0
(3,5)	10	19	0	220
(4,6)	13	29	0	0
(4,7)	12	26	100	0
(5,4)	9	15	0	0
(5,7)	4	11	0	220
(9,3)	15	8	0	220
(9,7)	11	32	0	0

3.1 Sample Network for Formulation I

The layout of the sample network is shown in Fig. 1. In this network, a feasible solution is found with mode-specific routes for the two OD pairs using the proposed formulation. Simplex method is used for solving the linear problem.

Table 1 presents the results for the first formulation. The flow of each class of vehicle through the links of network is given. Thus, the formulation has resulted in mode-specific links. Many links remain unassigned with flow which will be taken care of in a real-world large network. Cost values are assumed as given in the table.

3.2 Sample Network for Formulation II

The layout of sample network is shown in Fig. 2. A feasible solution cannot be obtained with formulation I for the given sample network as links outgoing from all the nodes cannot be practically mode-specific. Thus, the mixed traffic is considered as a third mode here and a higher cost is assigned to this mode.

Generalized reduced gradient algorithm (GRG) is used to solve the nonlinear programming problem. Table 2 presents the results for the second formulation. Three links are found to be mode-specific and others are found to have mixed traffic.

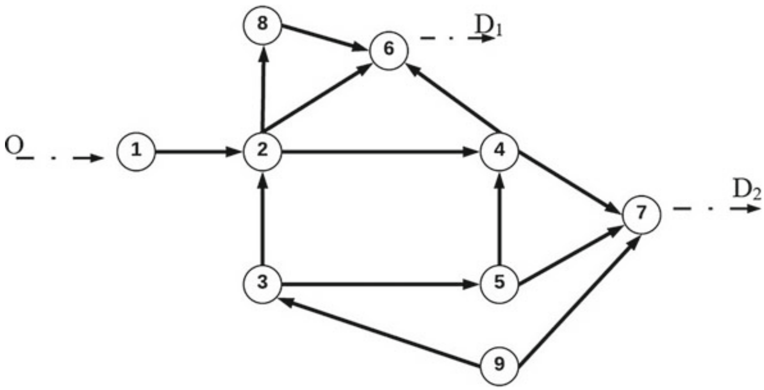

Fig. 2 Sample network for formulation II

Table 2 Solution of sample network for formulation II

Link	c_{ij}^1	c_{ij}^2	c_{ij}^{mix}	x_{ij}^1 (veh/h)	x_{ij}^2 (veh/h)
(1,2)	12	22	56	200	300
(2,8)	8	14	48	50	0
(3,2)	6	15	46	0	0
(3,9)	9	5	38	0	0
(8,6)	15	32	68	50	0
(2,6)	10	22	54	50	40
(2,4)	5	12	40	100	260
(3,5)	7	19	52	0	0
(9,7)	11	26	61	0	0
(4,6)	10	19	55	0	40
(5,4)	13	29	66	0	0
(4,7)	12	26	59	100	220
(5,7)	9	15	54	0	0

4 Discussion

This paper presented two formulations to segregate mixed traffic. It is found that, in cases where there is no feasible solution, formulation II will identify minimum links that need to be made multi-modal to achieve maximum segregation. While we assumed simple networks to demonstrate these models, formulation I can be scaled to a much larger and complicated network. However, formulation II being a nonlinear programming formulation may not be as easily scalable, and efforts are underway to develop alternate formulations.

Though the basic formulations presented in this paper are simple, the complexity increases with scalability and nonlinear cost and capacity functions. An appropriate solution approach is to be identified other than the solver solution approaches. This research will help to develop efficient strategies to model, control, and enhance safety of mixed traffic networks. Once the traffic is segregated, new strategies can be developed for homogeneous traffic links which will be more efficient as the traffic flow models and the optimal control strategies are well developed for homogeneous traffic. Segregating modes may increase efficiency and enhance stability and safety. Moreover, it will reduce intra-modal interactions and unutilized white spaces, thereby increasing the network capacity.

Some of the other areas currently under development include incorporating additional variables, capacity constraints, other solution approaches, dynamic cost functions and traffic assignment, transfer points and demand transfer across modes, etc.

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Identifying a Suitable Pedestrian Simulation Software—A Case Study on Emergency Evacuation of Classroom

Hemant Jain, Lakshmi Devi Vanumu, and K. Ramachandra Rao

Abstract Pedestrian simulation software has been in use for evaluation analysing "what if" scenarios, simulating emergency evacuations besides design of facilities. The main aim of this study is to examine and identify the appropriate pedestrian simulation software for representing the real-world situations. This process has been carried out by comparing the performance of different simulation software such as VISWALK, PEDESTRIAN DYNAMICS, and PATHFINDER. VISWALK works on the principle of social force model whereas PEDESTRIAN DYNAMICS works on social force model with Visual angle method while Path Finder works on multi-agent-based model with continuous modelling approach. Classroom evacuation during 2013 Ya'an earthquake in China (Li et al. in Saf Sci 79:243–253, 2015) is considered as the research problem to evaluate the simulation performances. In addition, calibrated parameter values of VISWALK (Gaddam et al. in Transportation Research Board 97th Annual Meeting, 2018) have also been used to check their adoptability to emergency situation in (Li et al. in Saf Sci 79:243–253, 2015). To achieve the desired level of precision, optimum simulation runs were performed using standard normal coefficients. The results show that VISWALK with calibrated parameters is able to represent real condition accurately. However, PATHFINDER and PEDESTRIAN DYNAMICS are user-friendly compared to VISWALK. The study presents the performance of simulation softwares, their merits, and demerits. Finally, it is concluded that irrespective of modelling approach, the calibration and validation of the model parameters for a given condition are essential.

Keywords Evacuation of pedestrians · Pedestrian simulation software · Calibration

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1 Background

Estimation of evacuation time in emergency conditions is a challenging task for researchers. The evacuation time of pedestrians in any situation depends on several factors related to pedestrian characteristics and type of infrastructure. To replicate the emergency situation in real-life environment with pedestrians or animals as subjects is risky and ethically objectionable. Hence, researchers are using some of the pedestrian simulation softwares to solve the problems related to emergency evacuations, design of pedestrian facilities, etc. With the availability of many simulation softwares, it is difficult to select a suitable one which can efficiently represent the real-world situations. In this regard, the present study aims to examine different pedestrian simulation softwares and identify an appropriate one by considering various aspects.

A cellular automata model is developed by Liu et al. (2009) to investigate the evacuation of classroom with obstacles. The results show that, along with spatial distance from the exit, other factors such as route chosen by the pedestrian and also density around the exit plays an important role in evacuation. Moreover, it is found that the model reproduced the experiment well with some discrepancies involving more stochastic behaviour of people in reality compared to that of model. In another study, Yang et al. (2011) tried to find the difference between real-world experiments and simulated experiments. This study used video recordings of the May 12, 2008, Wenchuan, magnitude 8.0, and earthquake in southwest China. The analysis shows a nonlinear variation between the arrival time and the order of person arriving, which is different from simulated exercises in which this relation is linear. Heliövaara et al. (2012) studied the selection of exit by evacuees' under various behavioural conditions where two exits located asymmetrically in a corridor. The results of the study show that the pedestrians may not be able to make optimal decisions when assessing the fastest exit to evacuate. Moreover, shorter egress times of the crowd are observed when the pedestrians behave egoistically instead of cooperatively. Chen et al. (2013) conducted several experiments to examine the route choice behaviour of pedestrians during evacuation of a classroom having two exits. They proposed a microscopic pedestrian model based on cellular automata. The simulation results show that the evacuation time is linearly increasing with the number of pedestrians. Li et al. (2015) used social force model to simulate emergency evacuation of a classroom during 2013 Ya'an earthquake occurred in China. Authors tried to understand the behaviour of pedestrians in real-life emergency situation. Further, they calibrated and optimized the parameters of social force model by using a differential evolution algorithm. From the available literature, Bernardini et al. (2016) proposed a database for earthquake evacuation models which involves speeds, accelerations, distance from the obstacles, etc. It is observed that the pedestrians prefer to move with an average speed of 2.3–3 m/s. From the fundamental diagrams of pedestrians' dynamics in emergency conditions during earthquakes, it is observed that, for the same density values, higher speeds and flows are observed compared to other studies (especially for evacuation drills and fire evacuation). Cuesta and Gwynne (2016) in their study provided the data related to five evacuation experiments from the same school where students of age group 4–16 years old were involved in the drills. Several performance datasets related to pre-evacuation times, travel speeds, route use, evacuation arrival curves, etc., were obtained. Li et al. (2017) study proposed a stair-unit model to describe the topologies of a stairwell and the results were compared with the real evacuation drills. The outcomes proved the consistency of the proposed model in representing the real behaviour. Gu et al. (2016) used the data obtained from videos of real emergency evacuation situations and analysed the evacuation behaviour of school students' during earthquakes. Comparison of students' behaviour in normal and emergency conditions is studied using regression models. The results revealed that student's behaviour in normal conditions is linear whereas in emergency, it is convex. However, lower reaction times are observed compared to other studies and "faster is slower effect" has not been observed. Han and Liu (2017) introduced information transmission mechanism into social force model to simulate pedestrian's behaviour in emergency situations considering the condition that most of the pedestrians are not familiar with the evacuation environment. They found that their model is able to produce the actual behaviour of pedestrians in emergency situations and also the pedestrians are able to choose the correct path through information transmission mechanism. Several experiments in various classrooms were conducted by Gaddam et al. (2018) with higher secondary school, under-graduate, and post-graduate students, and proposed a methodology to calibrate and validate the pedestrian simulation parameters in VISWALK for emergency evacuation conditions.

The paper is organized as follows: First section introduces the current research problem describing the existing research work. Section 2 describes the methodology adopted for the study. Third section presents results of the study. Finally, Sect. 4 concludes the paper.

2 Materials and Methods

Three pedestrian simulation software's VISWALK, PEDESTRIAN DYNAMICS, and PATHFINDER were selected for comparing their simulation performances for a particular type of research problem. VISWALK works on the principle of social force model whereas PEDESTRIAN DYNAMICS works on social force model with visual angle method while Path Finder works on multi-agent-based model with continuous modelling approach. Various features of these three softwares were presented in Table 1.

Classroom evacuation in china during 2013 Ya'an earthquake is considered as the research problem to evaluate the simulation performances and pedestrian evacuation data has been gathered from (Li et al. 2015). The classroom model is built in the three selected softwares to simulate the pedestrians' evacuation behaviour. The snapshots of evacuation scenarios in real situation as well as model development in different softwares were presented in Fig. 1. The inputs like number of pedestrians, route choice, and pedestrian speeds, etc., were assigned to the different softwares accordingly. To achieve the desired level of precision, optimum number of simulation runs

Table 1 Description of different softwares

S. No.	Software	VISWALK	PEDESTRIAN DYNAMICS	PATHFINDER
1	Principle	Social force model	Social force model with visual angle method	Multi-agent-based model with continuous modelling approach
2	Model building	Difficult	Moderate	Easy
4	Input type (number of pedestrians)	Pedestrian flow per hour	Percentage	Number of pedestrians or pedestrian flow per hour
5	Evacuation time	Not obtained by default. Manually observe the clock to check when the last pedestrian evacuates	Not obtained by default. To obtain evacuation time, we need to use emergency routing properly	Obtained by default after the simulation run
6	Parameters	Tau (τ), Lambda (λ), Grid size (D), Noise, React to N, VD	Density delay weight, Viewing distance, Re-route periodical, Re-route frequency, History Penalty, Personal Distance	Acceleration time, Reduction factor, Persist Time, Collision response time, Slow factor, Comfort Distance, Current room distance penalty
7	Remarks if any	Assignment of elements or modifications to the model is difficult	Modifications to the model are easy	User friendly, easily adaptable for any kind of simulation from microlevel to macrolevel

was calculated using standard normal coefficients (Varsha et al. 2016). The required number of simulation runs is found out to be 20 by considering confidence level as 95% and the confidence interval as 3 times of the sampling standard deviation.

The methodology suggested by Gaddam et al. (2018) has been adopted for calibration of pedestrian simulation parameters which involves identification of the most influential parameters in emergency evacuation, optimization of model parameters using real-world experiments. For further details, the reader is advised to refer Gaddam et al. (2018).

The methodology for calibration and validation of evacuation model is as follows:

Step 1: Import AutoCAD (.dwg) file to the simulation software to build the model
Step 2: Provide pedestrian count, composition, and speed distributions as input

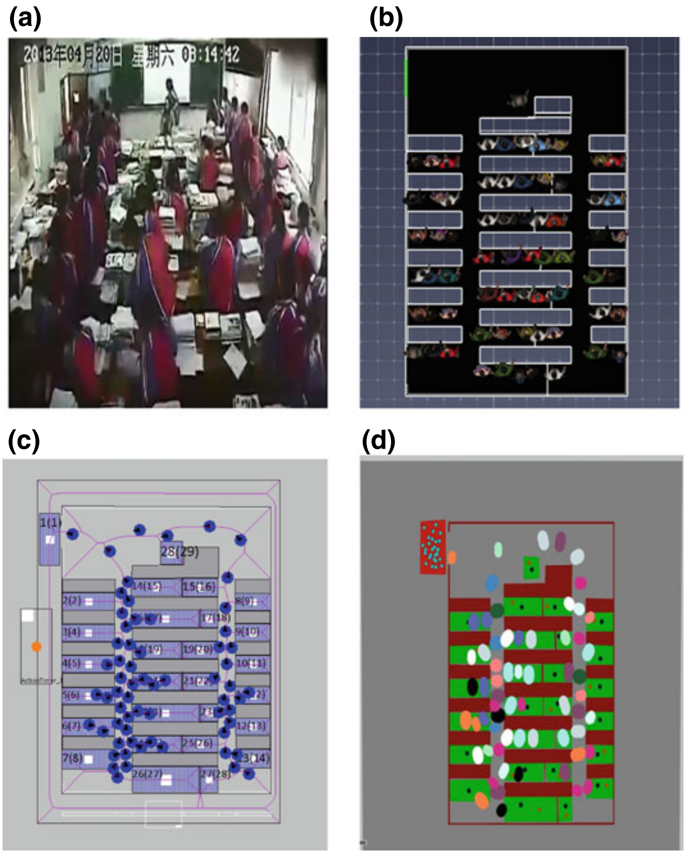

Fig. 1 Snapshot of classroom evacuation scenario in **a** real-world situation, **b** path finder, **c** PEDESTRIAN DYNAMICS, **d** VISWALK

Step 3: Select the measure of performance (MOP) and goodness of fit (GOF). The present study considered GEH1 statistics as MOP and cumulative absolute residual error (CARE) as GOF.
Step 4: Evaluate the simulation model with default parameters. If satisfied, proceed for model validation. If found unsatisfied, proceed for sensitivity analysis by performing two-way ANOVA test.
Step 5: If sensitivity analysis is found satisfied, proceed for calibration. If found unsatisfied, change the range of parameters and again perform sensitivity analysis until the satisfied results were obtained.
Step 6: Calibrate the model using latin hyper cube sampling and CARE
Step 7: If the GOF is found less than the critical value, proceed for model validation using GEH1 statistics and visual verification. Otherwise, perform the calibration process until the desired results were obtained.
Step 8: Print the results and end the procedure

Table 2 Description of default and calibrated parameters of VISWALK and PATHFINDER

S. No.	Software	Parameters	Default values	Calibrated values
1	VISWALK	τ	0.4	0.4
		λ	0.176	0.3
		D	5	0.6
		React to N	8	4
		VD	3	3
		Noise	0.2	0.2
2	PATHFINDER	Acceleration time	1.1	0.4
		Reduction factor	0.7	0.6
		Collision response time	1.5	3.8
		Slow factor	0.1	0.3
		Comfort distance	0.08	0.3

3 Results

Several simulations were performed and obtained the evacuation times of pedestrians. In PEDESTRIAN DYNAMICS software, with the default parameters, it is observed that the difference in the results of variance is too high. Hence, the calibration of parameters was not been carried out. Description of default and calibrated parameters of VISWALK and PATHFINDER softwares were presented in Table 2. The results obtained from different softwares were compared with the real data and found that the results of VISWALK with calibrated parameters is closely representing the real situation. Further, some acceptable results were observed for PATHFINDER with calibrated parameters. Figure 2 shows the comparison of evacuation times of pedestrians obtained with the simulation software using real data. The performance of three simulation softwares, merits, and demerits were presented in Table 3.

4 Conclusions

In this study, pedestrian simulation software, VISWALK, PEDESTRIAN DYNAMICS, and PATHFINDER were used to test their suitability in replicating a real-life situation. Classroom evacuation during 2013 Ya'an earthquake in China (Li et al. 2015) is considered in this exercise. After performing simulations with these softwares, it is observed that, working with PATHFINDER and PEDESTRIAN DYNAMICS is easy compared to VISWALK in terms of model building, pedestrian input, and route assignment, etc. In PEDESTRIAN DYNAMICS, with default parameters, the outcomes obtained are not significant and also the variance obtained was too high. Hence, the calibration of parameters was not carried out. In spite of having better results with PATHFINDER along with calibrated parameters, it is observed that

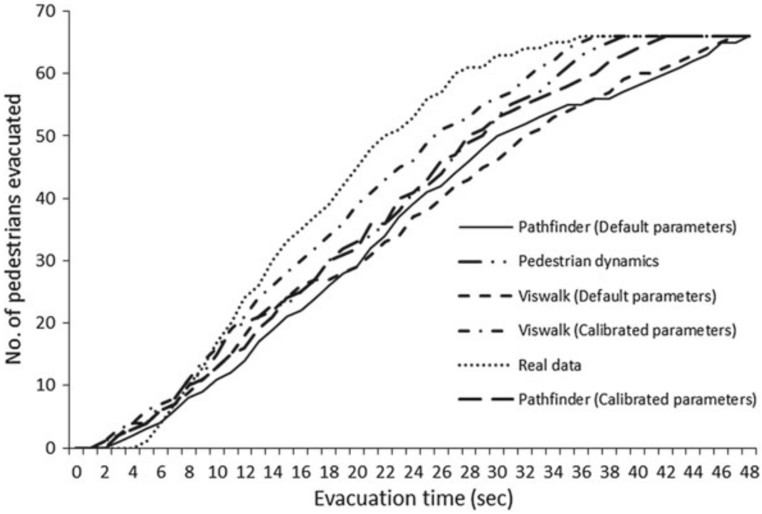

Fig. 2 Comparison of classroom evacuation results from various pedestrian simulation softwares with observed data

every time, the result is exactly same for the same model, because the random seed value cannot be changed that is generated by software which may be considered as the disadvantage for this software. Even though working with VISWALK is difficult compared to PATHFINDER and PEDESTRIAN DYNAMICS, the results obtained from VISWALK is more accurate and reliable. From the study, it is observed that irrespective of modelling approach, the calibration and validation of the model parameters for a given condition is essential. The parameters for VISWALK provided in this study have been tested for several classroom evacuation situations in Delhi (Gaddam et al. 2018) and it proved its validity. In future, the suitability of the calibrated parameters provided in this study can be evaluated for various types of pedestrian facilities which involve different age groups.

Table 3 Description of performance of simulation softwares

S.No.	Simulation software	Evacuation time		Ease of working[c]	Level of detail	Merits and demerits
		Mean ± S.D (s)	Error (s)[b]			
1	VISWALK (default parameters)	44 ± 0.93	+7	2	Micro	Model building is quite difficult. Requires lot of effort and time compared to other two softwares. Software will not calculate evacuation time by default
2	VISWALK (calibrated parameters)	37.55 ± 0.93	+0.55			
3	PEDESTRIAN DYNAMICS	44.50 ± 3.48	+7.50	3	Micro	Variance of result is too high. Cannot assign the exact number of pedestrians if there are multiple numbers of entries or exits
4	PATHFINDER	48[a]	+11	5	Micro and macro	Model building is easy but the random seed values should be given manually
5	PATHFINDER (calibrated parameters)	41.6[a]	+4.6			

S.D. Standard deviation
[a]No seed values (in every run, same value will be obtained)
[b]Total evacuation time in real condition is 37 s
[c]Scale 0–5 (0 for very difficult, 5 for very easy)

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A User Perception-Based Prioritization of Determinants of Walkability of Pedestrian Infrastructure Based on Multi-attribute Decision Making (MADM) Approach: An Indian Experience

Nikitha Vendoti, Bandhan Bandhu Majumdar, V. Vinayaka Ram, and Sridhar Raju

Abstract To plan the pedestrian infrastructure facilities, it is imperative to understand the pedestrian perception towards key critical attributes influencing sidewalk and crosswalk. However, a review of existing research literature suggests that evaluation of walkability of pedestrian infrastructure remain unexplored in a typical Indian setting. This paper addresses this research gap with respect to pedestrian infrastructure evaluation in Hyderabad, the capital of Telangana, a premier IT hub and a focal point for educational institutions, where a significant proportion of population uses walking for their daily commute needs. This paper demonstrates a user perception-based approach, proposed to identify and analyse the important factors of the urban environment that support or detract pedestrians from walking. In this study, initially, a brief literature review is taken up to identify a key set of parameters influencing the walkability of pedestrian infrastructure in typically Indian context. Then, user perceptions on these attributes are collected in a suitable five-point Likert scale. The collected data are then analysed based on grey relational analysis (GRA), a suitable multi-attribute decision making (MADM) technique to identify and prioritize the most important variables related to pedestrian walkability. Results indicate that safety and security are perceived as the most important parameters influencing pedestrian walkability.

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Keywords Pedestrian · Walkability · Multi-attribute decision making (MADM) · Grey relational analysis

1 Background and Objective

India being one of largest democracies in the world is fast moving towards its vision of being one of the leading economic powers. This has led to an accelerated urbanization. Rapid urbanization of Indian cities has resulted in an increase in the population influx. This increase in the urban population has emanated an increased need for mobility adding to an increased vehicular ownership and alarming growth of traffic on roads. Elevated levels of traffic on roads have contributed to severe traffic congestion on roads and rise in levels of pollution. Automobiles has made commute easier and comfortable for the people but also has made people vulnerable to high toxic levels of emissions from the combustion of fuels in automobiles. However, to deal with the heavy traffic congestion, often the focus has been on expanding the roads and constructing overpasses rather than trying to curb the traffic on roads and improving the sustainable modes of transportation. In regard to the major concerns of increasing traffic and levels of pollution, it is imperative that we promote walking as a prepared mode of transport at least for shorter distances and public transport for longer distances to address these issues to a certain extent. With a country like India, which seems to be bursting at seams in regards to its population, shrinking spaces and mixed land use, it is viable that we encourage walking which could considerably reduce the number of trips being performed by the automobiles.

Walking is one of the basic and sustainable modes of transport. Everyone uses walking in their daily commute to varying extents. However, the comfort and ease of travel offered by motorized mode of transport has made people to rely upon these modes even for a shorter distance travel adding to traffic congestion and pollution. Moreover, pedestrians are the most vulnerable road users who are at greater risk of being prone to accidents. Despite considerable number of trips being performed by foot, there is no adequate pedestrian infrastructure in the country. Often the design of roadways is based on managing the vehicular traffic with very little to no emphasis on the pedestrian facilities. The roads are expanded to deal with the increasing levels of traffic at the cost of pedestrians. The lack of proper pedestrian infrastructure has forced the pedestrian to use the road space intended for vehicles which not only disrupts the traffic but also compromises their safety putting them at higher risk of being involved in accident. This issue has to be addressed by proper design of pedestrian facilities thereby encouraging more people to walk.

In this background, this study assesses the walkability of pedestrian infrastructure in India, by examining the various factors influencing pedestrian's perception to walk. The objective of this study is (1) to conduct a detailed review of pedestrian walkability to understand the gaps in the existing literature and propose appropriate method. (2) To identify the factors influencing the walkability. (3) To prioritize the factors using MADM technique. Findings and inferences from this study would provide necessary inputs to urban planners. In the following section, a brief review of existing literature is presented for identification of suitable research gaps and demonstration of a suitable methodology.

2 Literature Review

A range of methodologies including quantitative as well as qualitative methods have been existing so far. For improved understanding of the research problem, a brief review of existing literature is conducted. In the following section, a summary of previous researches is briefly presented.

A study by Florian Gr¨aßle and Tobias Kretz (210) focused at the understanding the route choice of pedestrian. An observation was made on how the pedestrians would choose the route when they were presented with two similar alternative routes. The study inferred that the pedestrians choose geometry over the shortest path. The pedestrians refrained from using the shortest path giving importance to the geometry. Most of the pedestrian preferred to walk along the long path with paved walk way than walking over the shortest grassy terrain. However, this behaviour of pedestrians is ambiguous and it has been inferred that the route choice of pedestrians varies with the varying situations like flow on the routes, consideration for time etc.

In a study by Kelly et al. (2011) three methods were used to evaluate the walkability of the pedestrian infrastructure and a comparison was made. Stated preference (SP) survey, on-street survey and mobile method were the three methods used for the evaluation of the pedestrian environment. A household survey has been conducted using 47 attributes as checklist to determine how important they are in influencing the walkability of pedestrians. From the survey, the most significant set of nine attributes were identified. These attributes include the speed of the traffic, volume of the traffic, illumination of the streets, width, evenness and cleanliness of the pavement, cyclists on the pavement, detours and road crossings. Around 100 respondents were selected and they are presented with a hypothetical scenario. The respondents were asked to select their preferred route based on the prevailing pedestrian conditions with regard to the nine-identified factors. Using a tool developed in Excel, the given route is evaluated and utility scores were calculated. The study inferred that traffic volume was most detrimental to the pedestrian with cleanliness of the footpath being second most attribute to be deterring the pedestrians. In the on-street survey method, a paper questionnaire was designed to evaluate the user perception towards the 21 attributes identified from literature survey. The results indicated that ease of crossing the road and provision of litter bins and traffic volume were the worst-scored factors.

Another relevant study by Ariffin and Zahari (2013) aimed to assess the factors that would promote walking through the evaluation of people's perception towards identified factors. A survey was conducted to evaluate the user perception towards the attributes influencing their walking. Pedestrian concern with regard to existing pedestrian conditions on the roads was found out by walkability audit. A majority of respondents were found to be stating "places being far away" was the main factors deterring them from walking. Crime issue was addressed to be the next most important attribute preventing the pedestrians from walking more often. Volume and speed of cars, travel with small children were found be having more or less equal impact in affecting the pedestrian's walkability. Presence of sidewalk and the condition of the sidewalk were found to be the having very less influence on pedestrians. Very few pedestrians identified the street light and presence of curb ramps to be that factors preventing them from walking often.

A very interesting study by Cubukcu et al. (2014) developed walkability maps for an area. This study aimed to evaluate the walkability using geographic information systems (GIS). The study which was conducted in the districts of Turkey. The data of 6500 road segments corresponding tonine districts were evaluated. The land used data were digitalized using GIS and analysed using spatial design network analysis. The parameters influencing the walkability were identified and classified into seven categories, namely land use, accessibility, safety from traffic, crime safety, comfort for walking and cycling and environment aesthetics. Walk score a measure of walkability is evaluated by network design and accessibility to amenities. Network designed was analysed based on two scores, namely centrality and betweenness scores. Evaluation of access to amenities is done by network analysis. The walkability maps were developed from the walkability score evaluated.

In their study Zainol et al (2014) evaluated the pedestrian satisfaction towards the facilities available in the study area. The study was based in the Melaka historical city centre. A survey questionnaire was developed to evaluate the user perception towards the attributes on a 9-point Likert scale. The attributes in the survey corresponded to the pathway, aesthetic and amenities, signage and street furniture, personal safety and separation from the traffic flow. The analysis and evaluation of road segments in the study area were done using analytical hierarchy process.

A review study by Kadali and Vedagiri (2016) identified the various factors in influencing the pedestrians level of service pertaining to sidewalk, cross walk at intersection and midblock cross walk. Few studies have developed qualitative method considering the factors like safety, security, continuity, comfort and convenience. Various parameters like surface quality, pedestrian volume and obstructions have been used for the assessment of level of service (LOS) offered by the pedestrian facilities in some of the studies. In few studies, the main focus has been on the safety considered by the adjacency to traffic flow and level of segregation from the traffic. Most of the studies existing have been based on user perception. In regard to crosswalk, signal control, delay at intersections, turning manoeuvers, vehicle stop line behaviour and pedestrian vehicle interactions were identified to be the most significant attributes. From the studies, it has been inferred that the midblock cross is more dangerous than crosswalk despite the absence of turning traffic. Studies have also proved that midblock crosswalk is likely account for more number of accidents or the vehicle pedestrian conflicts.

From the literature review, it can be inferred that there has been very limited existing research work on pedestrian infrastructure in the country. The user perception-based evaluation and prioritization of attributes would help in developing

Table 1 List of identified attributes

Attributes relevant to sidewalk and crosswalk	Attributes relevant to crosswalk
Presence of sidewalk facility	Traffic speed
Safety from traffic	Traffic volume
Foot path geometry	Presence of crossing facilities
Footpath cleanliness	Complex turning movements of traffic
Aesthetics and amenities	Signal control
Street lighting	Pedestrian vehicle interactions at intersection
Safety and security	Intersection geometry
Other pedestrian characteristics	Pedestrian holding space near junction
Weather condition	Pedestrian delay at intersections
Pollution	Vehicle stop line behaviour at junction

a pedestrian friendly environment. Table 1 lists the most significant attributes identified from literature review. The attributes are further divided into two groups, namely attributes relevant to sidewalk and crosswalk both and specific to crosswalk only.

3 Methodology

3.1 Area of Study

This study attempts to understand the pedestrian's perception towards the factors influencing their level of walkability with regard to the prevailing pedestrian facilities in the city of Hyderabad, the capital of Telangana. Hyderabad, being the capital city, has an interesting mix of population. Not only does it attract many tourists but also draws a lot of trade. It being a premier IT hub and a focal point for educational institutions and coaching centres, it will be safe to say that significant proportion of population uses walking in their daily commute. Despite the considerable amount journey made by foot, there are no proper pedestrian facilities in the city. Most of the existing footpaths are encroached by vendors or obstructed by poles or the vehicles being parked on the sidewalks, which forces the people to use the road space which is laden with high volumes of traffic. An on-street survey of pedestrians is conducted in the pedestrian crowded areas to understand user perception towards various parameters affecting the walkability of pedestrians.

3.2 Data Collection

Initially, a set of most significant factors with regard to typical Indian context were identified from the list of factors obtained from a detailed literature review. The data collection is based on a typical travel behaviour survey questionnaire. A survey questionnaire was designed to evaluate the user's perception towards each of the identified attributes. The responses were collected on suitable five-point Likert scale. Users were provided with five levels of criteria for each of the attribute, namely very less important (−2), less important (−1), neutral (0) important (+1) and very important (+2). Around 120 responses were collected through on-street and off-street survey, out of which eight responses were removed from the data due to incomplete information and the remaining 112 responses were used for the development of data base and further analysis.

4 Analysis

4.1 Preliminary Analysis

This section presents the results of a preliminary analysis of the responses recorded from the respondents. Out of the total respondents, males constitute 53% and females constitute 47% with majority of them being in the age group between 20 and 35 years. From the analysis, it can be inferred that 65% of the people walk every day. The respondents were majorly found to be walking to school or work followed by walking for recreation. The details of descriptive analysis were presented in Table 2.

The following charts represent (Figs. 1 and 2) the percentage of respondents corresponding to various levels of importance given to the attributes. A considerable amount of strong or moderate agreement (strongly agree or agree) on an attribute indicates that the particular attribute is perceived to be strongly or moderately influencing (motivating/deterring) pedestrian's walkability. For example, 48% and 23% of the pedestrians have provided very strong and strong agreement with the attribute safety and security. This observation clearly indicates that improper safety and security from the crimes are the most deterring factors as perceived by pedestrians. Secondly, safety from traffic has been perceived by majority of the respondents as the factor deterring the pedestrians from walking. In specific to cross walk, the pedestrians perceived signal control as very important. On the otherhand, it can be noticed that aesthetics and amenities were considered to be less important based on user's perception. In regard to cross walk, pedestrians perceived the delay caused in waiting to cross a road to be very less important.

Table 2 Preliminary analysis of data

Socio-economic classification	Groups	% of respondents
Age	<20	19
	20–35	57
	35-–55	20
	>55	4
Gender	Male	53
	Female	47
Frequency of walking	Once in a week	6
	2–3 times in a week	12
	Sometimes in a week	17
	Everyday	65
Purpose of walking	Walking to school/work	54
	Walking to transport	19
	Walking to run errands	9
	Walking for recreation	18

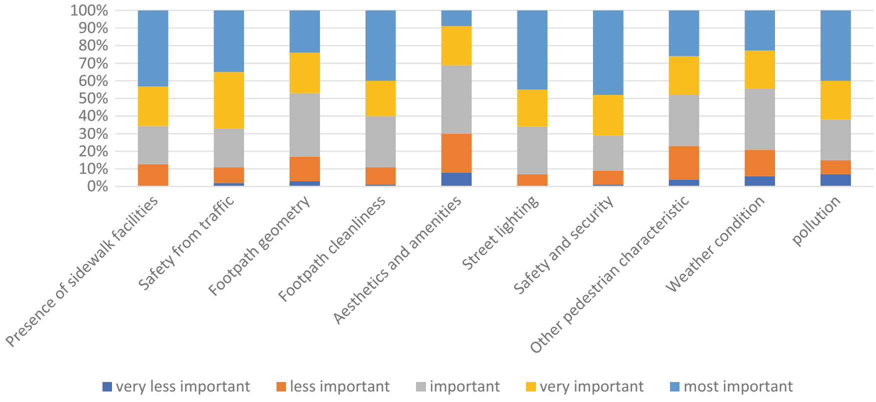

Fig. 1 Percentage of level of importance given to various sidewalk attribute

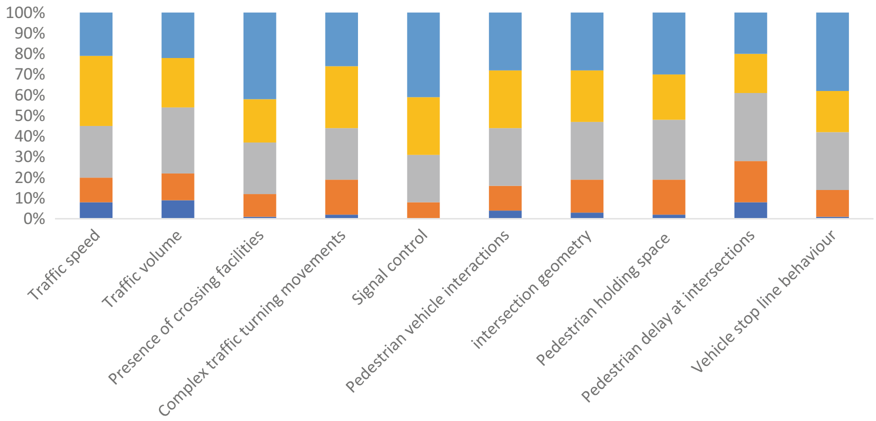

Fig. 2 Percentage of level of satisfaction given to various crosswalk attributes

4.2 Prioritization of Factors Influencing Pedestrian's Walkability Based on MADM Techniques

Deng proposed grey relational analysis to investigate a distribution-free discrete data series based on Likert-type scale. GRA combines a range of scores associated with different factors into a single factor and reduces the dimension of the original problem. A number of previous studies used GRA for solving MADM problems. In his paper, Wu (2007) argued that the justifiability of traditional techniques of statistical analysis is based on assumptions made with regard to the distribution of population and variances of samples. The lesser size of sample generally affects the accuracy and solidity of the results produced by such techniques. As Wu (2007) suggested that in some of the real-life situations, making decisions under uncertainty and with insufficient or limited data available for analysis are actually essential. In this regard, J. Deng proposed the grey system theory in 1974. In various fields, for the analysis of data, grey systems theory has been widely applied. The grey relational analysis is an effective approach for data analysed on a Likert scale which analyses the discrete data series using grey system's theory. This technique is especially useful for small or limited data availability. For Likert-type data analysis, GRA is one of the widely adopted approaches. However, as a further extension of this research, a Fuzzy-GRA application would be ideal.

4.3 Theoretical Background of Grey Relational Analysis

Deng proposed the grey systems theory in 1974 to enhance the decision making under incomplete and uncertain information available. The term grey in grey relational analysis stands for poor, representing insufficient and uncertain information available. Grey relational analysis (GRA), a subset of the grey systems theory is used for the evaluation of discrete data. Using GRA, the relative importance of an attribute over other attributes can be evaluated. The process of grey relational analysis is discussed in detailed below.

Step-1: Generate reference data series x_0
In the above expression, m is the number of respondents. In general, the x_0 reference data series consists of m values representing the most favoured responses.
Step-2: Generate comparison data series x_i

$$x_i = (d_{i1}, d_{i2}, \ldots d_{in})$$

where $i = 1,\ldots, k$. k is the number of scale items. So, there will be k comparison data series and each comparison data series contains m values.
Step-3: Compute the difference data series Δ_i

$$\Delta_i = (|d_{01} - d_{i1}|, |d_{02} - d_{i2}|, \ldots, |d_{0n} - d_{in}|)$$

Step-4: Find the global maximum value $\Delta_{\max}$ and minimum value $\Delta_{\min}$ in the difference data series.

$$\Delta\max = \forall i^{\max}(\max\Delta_i)$$

$$\Delta\min = \forall i^{\min}(\min\Delta_i)$$

Step-5: Transform each data point in each difference data series to grey relational coefficient. γi(j) represents the grey relational coefficient of the jth data point in the ith difference data series, and then the coefficient can be estimated from the following form

$$\gamma_i(j) = \frac{\Delta\min + \varsigma\Delta\max}{\Delta_i(j) + \varsigma\Delta\max}$$

where $\gamma_i(j)$ is the jth value in Δi difference data series. ς is a value between 0 and 1. The coefficient ς is used to compensate the effect of Δmax should Δmax be an extreme value in the data series. In general the value of ς can be set to 0.5.
Step-6: Compute grey relational grade for each difference data series. Let Γ_i represent the grey relational grade for the ith scale item and data points in the series are assumed of the same weights.

$$\Gamma_i = \frac{1}{m}\sum_{n=1}^{m}\gamma_i(n)$$

The magnitude of Γ_i reflects the overall degree of standardized deviance of the ith original data series from the reference data series. In general, a scale item with a high value of Γ indicates that the respondents, as a whole, have a high degree of favoured consensus on the particular item.

Step-7: Sort Γ values into either descending or ascending order to facilitate the managerial interpretation of the results.

4.4 GRA-Based Prioritization of Factors

Table 3 presents the prioritization of factors based on the average grey score evaluated.

Table 3 Ranking of attributes based on GRA

Attribute relevant to sidewalk and crosswalk	Average grey score	Rank
Presence of sidewalk facilities	0.8719	4
Safety from traffic	0.8874	2
Footpath geometry	0.8296	7
Footpath cleanliness	0.8709	5
Aesthetics and amenities	0.8014	10
Street lights	0.8858	3
Safety and security	0.8940	1
Other pedestrian characteristics	0.8267	8
Weather	0.8211	9
Pollution	0.8642	6
Attributes specific to crosswalk	**Average grey score**	**Rank**
Traffic speed	0.8286	7
Traffic volume	0.8187	8
Complex traffic turning movements at intersections	0.8403	5
Signal control	0.8830	1
Pedestrian vehicle interactions at crosswalk	0.8427	3
Intersection geometry	0.8384	6
Availability of pedestrian holding area near junction	0.8417	4
Pedestrian delay at intersection	0.8018	9
Vehicle stop line behaviour at junction	0.8637	2

5 Discussion

A greater value of grey score for an attribute indicates that a particular attribute has been given the top-most priority by the pedestrians. Safety and security was the top prioritized factor with grey score of 0.8940 indicating the pedestrians utmost concern for safety and security from the crimes happening on the roads. The second most consideration of pedestrian as inferred from the result was the safety from the traffic. Street light, presence of sidewalk facilities and footpath cleanliness were the third-, fourth- and fifth-ranked attributes followed by pollution, footpath geometry. Signal control was the first-ranked factor specific to a crosswalk with the vehicle stop line behaviour being the second-ranked attribute. Pedestrian vehicle interaction, holding space at intersection and complex turning movements were the next top-ranked factors. The pedestrians were least concerned about the delay incurred in waiting to cross the road.

6 Conclusion

Based on a detailed investigation on user perception towards a set of attributes influencing both sidewalk and crosswalk infrastructure, the following concluding remarks can be made:

- Firstly, safety and security were perceived as the most important attribute. In the present-day scenario of increasing crime rates and possibility of mishaps taking place, the pedestrians gave at most attention to their safety and security on roads.
- Secondly, safety from the on-street traffic is found to be the second most important consideration to the pedestrians. This finding clearly indicates that, adequate buffer distance has to be provided so that it could give adequate space for protecting the pedestrians in situations where the vehicle has gone out of control. However, in typical Indian context, inadequate space and increasing levels of traffic on roads have made the interaction more and simultaneously increased the risk of pedestrians being involved in accidents.
- Thirdly, provision of adequate illumination through street lights was found as an influential variable as perceived by users. Provision of street lights can reduce the fear of crime and encourages the pedestrians to walk even after dark by increasing the safety and security.
- Fourthly, among the crosswalk specific factors, signal control was perceived as the top-most important attribute by the pedestrians. The unsignalized intersections are likely to result in unsafe pedestrian crossing.
- Fifthly, vehicle stop line behaviour was also perceived as an important attribute as it is very essential the that vehicles comes to a stop at the stop line and does not stand on the roadway ahead of the stop line which would result in less available space for the crossing pedestrians.

- Sixthly, this study is one of the unique efforts towards prioritization of attributes influencing user perception at sidewalks and crosswalks in typical Indian context and shows an application of multi-attribute decision making approaches such as GRA.

Overall, these findings can be used as preliminary guidelines for planners and stake-holders in India. Although, the results are case-specific, the demonstrated methodology can be applied in other cities with similar socio-economic settings.

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Methodology to Identify a Key Set of Elements Influencing Bicycle-Metro Integration: A Case Study of Hyderabad, India

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Abstract Rapid urbanization and related motorization along with random development at the suburban level without adequate infrastructure have created several transport-related externalities such as (a) traffic injuries and fatalities, (b) congestion, (c) transportation-related air and noise pollution and d) mobility issues faced by poor in a typical urban Indian context. In this regard, the increased use of public transit (PT) system could be an effective demand-management instrument to mitigate the above-mentioned externalities. Although, bus is the most popular PT-mode across the nation, metro rail has emerged as an effective alternative during the recent past. Nevertheless, a successful metro rail system incorporates an efficient feeder system. The inadequate supply of feeder system for the metros, arising due to the absence of the necessary planning is consequently forcing the metro riders to stick to auto-rickshaws or private motorized vehicles like two wheelers, which in turn are polluting the environment. On the other hand, bicycle transportation, if planned and implemented with necessary measures, could be an effective, pollution free and cheaper feeder to metro in a typical Indian context. In this regard, this paper aims to propose a methodological framework to identify the key set of factors influencing bicycle a feeder or access transportation mode to metro by understanding the perception of bicyclists and non-bicyclists towards key attributes related to bicycle-metro integration.

Keywords Bicycle · Metro · Feeder mode · Commuter's perception · Multi-attribute decision making (MADM) · RIDIT

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1 Introduction

As the world is moving towards twentieth century, there have been notable advancements in almost every field of science and technology. The world is becoming a smaller place, with the rapid development in transportation systems. Rapid urbanization has commanded to densely populated cities and towns, which has led to complications like overpopulation, pollution and traffic congestions. Increase in disposable income of commuters has led to increased number of private vehicles on the street, traffic congestions, pollutions, accidents and road safety issues. In this regard, the increased use of public transit (PT) system could be an effective instrument to mitigate the above-mentioned externalities. Although, bus is the most popular PT-mode across the nation, metro rail has emerged as an effective alternative during the recent past. Nevertheless, a successful metro rail-service depends significantly on the service quality of the feeder mode system. The inadequate supply of feeder system for the metros, arising due to the absence of the necessary planning is consequently forcing the metro riders to stick to auto-rickshaws or private-motorized vehicles like two wheelers, which in turn are polluting the environment. Moreover, the currently available feeder systems including auto-rickshaws, taxis and share-cabs fares of which dominate the total fare of the whole journey. Hence, there is a need to integrate the metro-system with a reliable, eco-friendly and cost-effective feeder system to address the first/last-mile problems and attract more commuters to metro-rails. In this context, bicycle a green and active mode of transportation, could be an effective, pollution free and cheaper feeder to metro in a typical Indian context.

In this background, this study assesses the feasibility of using bicycle as a first/last-mile solution for the metro-system in India, by examining the various factors influencing people's acuity to the use of bicycle as a feeder mode. Bicycles will serve as a perfect mode for a feeder system, as they can be parked at the transit station, or even can be taken in the transit units. Derived results from this study would provide necessary inputs to planners for implementation. In the following section, a brief review of existing literature is presented for better understanding and identification of suitable research gaps.

2 Literature Review

This section presents a brief review of existing research literature available related to bicycle and public transportation integration. A brief description of each of the research works are presented in the following section in a chronological manner (earliest one first). The literature review is further sub-divided into two components. The first component of the literature review briefly discusses some of the existing research studies investigating bicycling-related attributes in general, and the second part of the literature review specifically presents a review of past studies investigating towards the determinants of bicycle-public transit integration.

In one of the very interesting bicycling-related studies, Taylor and Mahmassani (1996) conducted a stated preference survey using hypothetical scenarios under which the users had to choose to make a work trip by automobile only, park and ride or bike and ride. The authors found out that the negligible cargo carrying capacity of the bicycle is one of the major deterrents for cycling. Presence of light traffic, lockers at the bicycle stations and shower facilities at the station was found to encourage cycling. Although these factors are important, further research clearly indicated that access distance was more crucial to promote bicycling as a feeder mode. In his study, Rietveld (2000) analysed the access modal choice from home end to the railways station and from railways station to trip end, in Netherlands and concluded that the less modal share (23%) of bicycle as a feeder mode for the railways was due to (a) No availability of bicycle at the end of trip. (b) Security concern regarding bicycle at station. In another study, Krygsman et al. (2004) analysed the access and egress distances to the public transport. Results have clearly shown that an increase in access and egress time is associated with the less likability of the public transport. Rietveld and Daniel (2004) in their research found that the physical effort required in cycling is a major concern discouraging bicyclists. Gatersleben and Appleton (2007) found out that respondents who cycle occasionally, had a very positive attitude towards cycling and they can be persuaded to cycle more often by providing social support as stated by Prochaska's model of behavioural change. Heinen et al. (2011) evaluated the difference in attitude between cyclists and non-cyclists, and between part-time and full-time cyclists. The authors concluded that majority of the people make their mode choice decision based on the direct benefits, such as time, comfort and flexibility. Nkurunziza et al. (2012), in their research tried to find the motivators and barriers leading to the people's perception about choosing bicycle as a mode of transportation, in Dar-es-Salaam, Tanzania, which has a low cycling modal share of 5%. The authors concluded that just mitigating the physical barriers is not enough; awareness among people must be raised to change the mentality of the general population. Zhao (2014) through his research revealed that traffic safety concerns and increasing air pollution due to increasing vehicle counts are the major reasons for the low cycling rates. Rahul and Verma (2014) conducted a very pertinent study in the city of Bangalore, India, to estimate the maximum distance beyond which a person will not choose to cycle or walk to his destination. The authors found the maximum distance to be 1972 m, beyond which a person will choose a motorized mode of travel. This distance obtained from the research will be helpful in planning the feeder systems in India.

In the second component of the literature review, a set of previous studies aimed at investigating the role of different attributes on bicycle-metro integration are briefly presented. Among such studies, Bachand-Marleau et al. (2011) aimed to (a) characterize the potential transit users interested to use bicycle as a feeder and (b) identify the factors influencing bicycle-metro integration in the city of Montreal, Canada. As Montreal already has a public bicycle rental system, Bixi, the city residents are already familiar with using bicycle as a mode of transport. The study was carried out through an Internet survey. The author identified four key methods to integrate bicycles with public transport such as (a) bicycles being brought through public transit,

(b) Increasing the number of bicycle parking slots near transit stations, (c) Increasing and improving the existing connections between transit stations and bicycle paths and lanes and (d) Introducing public bicycle sharing schemes at key trip generators. The results concluded; 63% of the sample was ready to combine bicycle with public transport. Respondents unwilling to integrate public transport with bicycle, preferred to travel short to medium distances just by bicycle or, they were worried about bicycle theft at the transit station. Of all the various methods to combine bicycle and public transit, 60% of the users preferred carrying bicycles while travelling through mass transit modes. An improved quality parking facilities were found to be significantly more attractive to regular commuters. The most important factors affecting the users were identified as transit mode, overall land-use of the access and egress zone along with overall urban form and reasons for making the trip.

In another pertinent study, Heinen and Bohte (2014) analysed the user perception about integrating bicycles with public transport, with the help of an Internet survey in Netherlands. Public Transport and Bicycle can be integrated in several ways. (a) Riding your own bicycle to the transit station. (b) Taking the rental bicycles from the origin to the transit station and while returning, taking the rental bicycle from the transit station and riding it home. (c) Transit facilities allow to carry your bicycle inside the transit unit. The author has considered the first alternative in the paper as it is the most commonly used bicycle-transit integration in Netherlands. Results concluded that even if the distance was kept constant, using a mode of transport was largely dependent on the attitude characteristics of the user.

In another study, Verma et al. (2016) evaluated the behavioural aspects of people responsible for low cycling rates in Bangalore city, India. They concluded that perception about cycling changes from childhood to adulthood due to shame and not considering cycling a respectable means of transport. Growing into adulthood, they were keen to shift to motorized modes and believed it to be a sign of prosperity, which is not correct and should be mitigated by conducting workshops and seminars. In a recent study, Zhao and Li (2017) conducted a detailed investigation to identify the determinants for the integration of bicycle as a transfer mode for the metro-system in the city of Beijing, China. They observed that when bicycle is used as a transit mode, destination accessibility and built environment play a major role. Transit passenger's maximum tolerable distance for cycling was found in the range of 1.2–3.7 km depending upon various factors. Unsafe and unmaintained parking facilities at the transit facilities were observed as barriers to bicycle-transit integration. The study revealed that the presence of parks along the way of the bicycle lanes would encourage bicycling rates. The authors also classified the users into three classes, namely (a) users with relatively lower income, with fewer alternatives and longer distance from transit stations; (b) users with relatively higher income with high standard of living and (c) users, who are attracted to public bicycle sharing scheme located near transits.

Based on the detailed literature review, the following gaps requiring further research attention could be identified:

Firstly, in India, the travel distances are short-to-medium length, which is favourable for bicycling. Despite that, not much work has been done on bicycle infrastructure development. Secondly, in most of the studies, all the factors regarding the user perception are analysed simultaneously, but prioritization of the factors must be done because all factors will not have the same impact. Thirdly, less attention has been paid towards the integration of bicycle and metro or any other public transport for that matter. Due to the recent increase in traffic problems, it takes high time to check the feasibility of economical modes of transportation.

3 Data Collection

User perception on key attributes influencing bicycle-metro integration is collected in typical Likert-type scale from metro commuters. The survey was conducted near metro stations, so generally, metro commuters were surveyed. A questionnaire was designed to get to know the user's socioeconomic characteristics, trip making characteristics as well as the perception regarding the bicycle-metro integration attributes. The responses were collected on a standard five-point Likert scale. Users were provided with five different choices for each statement, namely strongly disagree (−2), disagree (−1), neutral (0), agree (+1) and strongly agree (+2). A total of 130 users were surveyed, out of which 27 were removed due to incomplete data and the remaining 103 data sets were considered for database development and subsequent analysis. Descriptive statistics of the complete data set are presented in Table 1.

4 Preliminary Analysis

In this section, a preliminary descriptive analysis of the collected behavioural data is presented (Fig. 1). A significant proportion of strong or moderate agreement ("strongly agree" and "agree") on a factor indicates that the factor is perceived as a strong or moderate deterrent/motivator to bicycle-metro integration. Similarly, a factor with a substantial share of users indicating disagreement ("strongly disagree" or "disagree") suggests that the factor is perceived as weakly influencing bicycle-metro integration. For example, 18% and 36% of respondents have provided very strong and strong agreement with the attribute weather condition, respectively. This finding clearly indicates that predominantly hot and humid climate is the most deterring factor towards bicycle-metro integration in India. Similarly, traffic conditions have been perceived as one of the critical factor towards successful operation of this particular integration. On the other hand, topographical factors such as terrain condition and bicycle facility-related factors such as bicycle rental cost were found to be relatively less important attribute based on user perception. Similar inferences can be made for other attributes. However, for better interpretation and ranking, appropriate multi-attribute decision making (MADM) techniques such as RIDIT need to be adopted.

Table 1 Descriptive analysis of the data set

Classification	Socioeconomic groups	Sample proportion (%)
Gender	Male	77
	Female	23
Age	<20 years	6
	20–35 years	78
	35–55 years	12
	>55 years	4
Education	Up to 10th	2
	12th	17
	Graduate	61
	Post-graduate	20
Occupation	Business	9
	Retired	4
	Self employed	11
	Service/job	30
	Student	46
Monthly family income (INR)	Up to 10,000	7
	10,000–20,000	10
	20,000–30,000	10
	30,000–40,000	17
	40,000–60,000	25
	60,000–80,000	15
	More than 80,000	16

4.1 Prioritization of Factors Influencing Bicycling Based on MADM Techniques

RIDIT, proposed by (Bross) is a MADM technique for comparing two or more sets of "ordered qualitative data". Among the two data sets, one is designated as a reference or base data set, and the other is compared with respect to the reference data set. The word-RIDIT is an abbreviation of "Relative to an Identified Distribution and the suffix—it represents a transformation". RIDIT has been extensively used for priority rankings.

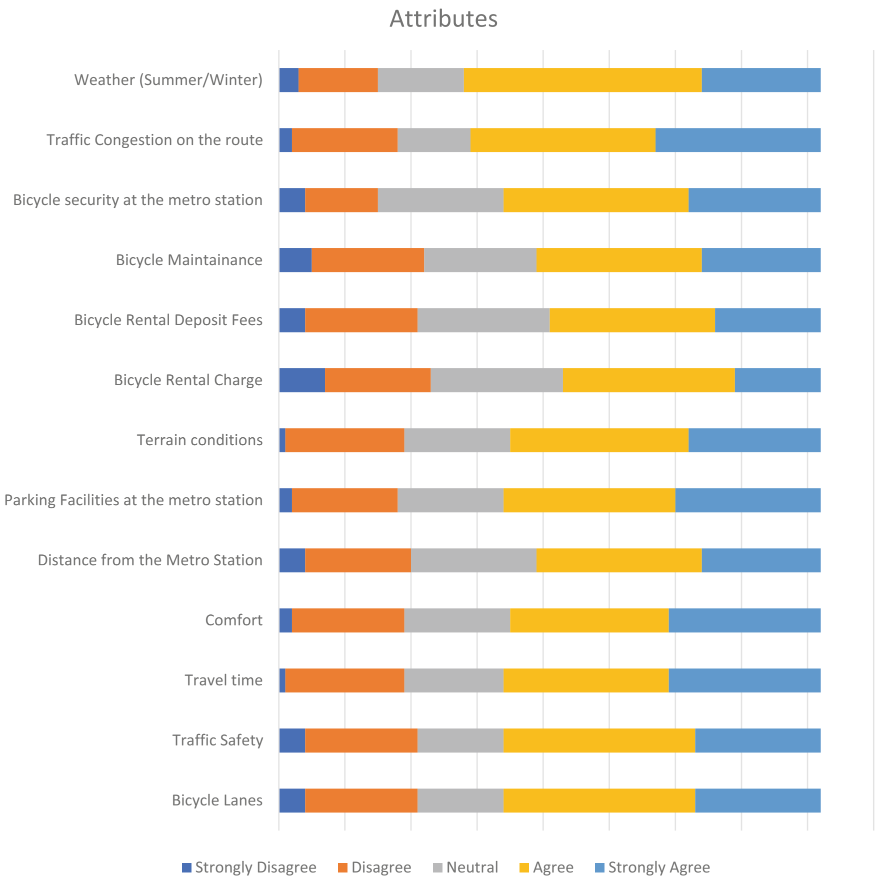

Fig. 1 Summary of attributes

4.2 Theoretical Background of RIDIT

RIDIT is an established multi-attribute decision making (MADM) approach proposed by Bross (1958). The name "RIDITS" was chosen for its analogy with "probits" and "logits". Ridits are generally based on the observed distribution of a response variable for a specified set of individuals. This approach is very closely related to distribution-free methods based on ranks such as Wilcoxon Test (Bross 1958). RIDIT possesses two very important properties. Firstly, it assigns a rank value to each class proportional to the relative frequency of observations in that class. Secondly, the rank value is standardized within 0–1. The latter property eliminates the problem of variation in the relative positions with respect to number of ranks. RIDIT technique appears to suppress the differences in distributional shape (Selvin 1977).

Procedure

Step-1: Identification of a reference or base data set is necessary for analysis. In this particular scenario, the collected responses from the respondents could be considered as the reference data set, only if the actual population remains un-identified.

Step-2: The response category frequency f_i for which $i = 1, \ldots, n$ needs to be estimated in this step.

Step-3: The third step comprises of determination of the frequency that is accumulated at mid-point and can be expressed as F_j for each category of responses as the following

$$F_1 = \frac{1}{2} f_1$$

$$F_j = \frac{1}{2} f_j + \sum_{k=1}^{j-1} f_k \text{ where } k = 2, 3, 4, \ldots, n$$

Step-4: In this particular step, the RIDIT value Rj is calculated for each response category corresponding to the reference data set:

$R_j = \frac{F_j}{N}$. In this expression, $j = 1, 2, 3, 4, \ldots\ldots, n$

In the aforementioned formulation, N represents the number of total responses that are obtained from the travel survey. The expected RIDIT value for the reference data set remains 0.5.

Step-5: In fifth step of the analysis, RIDITs as well as mean RIDITs needs to be estimated for the comparison data set. Comparison data set can be developed by including the frequencies of corresponding responses obtained from the survey through Likert scale items. If there are a total of m Likert-type scale items, then there would simultaneously a total of m sets of comparison data sets. Subsequently, corresponding RIDIT value r_{ij} needs to be estimated for every category based on the following expression:

$$r_{ij} = \frac{R_j x \pi_{ij}}{\pi_i} \text{ where } i = 1, 2, 3, 4, \ldots, m$$

In the above expression, π_{ij} represents the frequency corresponding to the jth category specific to the ith scale item. On the other hand, π_i indicates the summation of all frequencies corresponding to scale item i across all corresponding categories. This expression can be presented in the following form:

$$\pi_i = \sum_{k=1}^{n} \pi_{ik}$$

The mean RIDIT ρ_i corresponding to each Likert scale item needs to be estimated with the following expression

$$\rho_i = \sum_{k=1}^{n} r_{ik}$$

Step 6: In this stage, the confidence interval specific to the mean RIDIT needs to be computed. If the size of the sample size of the respective reference set of data becomes significantly large compared to any of the comparison data set, then the corresponding 95% confidence interval of the mean RIDIT can be expressed as follows:

$$\rho_i \pm \frac{1}{\sqrt{3\pi_i}}$$

Step 7: The final stage of RIDIT analysis comprises of calculation of Kruskal–wallis test statistic to test for significant differences across response categories

$$H_0 = \forall i,\ \rho_i = 0.5$$

$$H_a = \exists i,\ \rho_i \neq 0.5$$

$$W = 12\sum_{i=1}^{m} \pi_i (\rho_i - 0.5)^2$$

A relatively low estimate of ρ_i is generally ranked higher over a relatively higher value of ρ_i as a low value of ρ_i clearly reveals low probability of being in a negative propensity. Table 2 presents the RIDIT value estimates for different attributes and corresponding derived ranks.

4.3 RIDIT-Based Prioritization of Factors

A comparatively higher estimate of Kruskal–Wallis (W) of 22.24 against the critical chi-squared value with 12 degrees-of-freedom at 5% level of significance [$\chi^2_{10-1} =$ 16.92] clearly shows that user responses on the importance of deterrents are statistically significantly different. Among all the attributes, Weather ($\rho_i = 0.6083$) is perceived as the top-most factor. This indicates that hot and humid climatic conditions are major deterrents for the bicyclists in Hyderabad. Bicycle security at metro station ($\rho_i = 0.5982$), traffic congestion on route ($\rho_i = 0.5822$) and traffic safety ($\rho_i = 0.5747$) are ranked as the second, third and fourth most important factors, respectively. This finding clearly indicates that bicycle security plays an important

Table 2 Comparison of various attributes: RIDIT values

RIDIT Values specific to the reference-dataset						RIDIT values corresponding to the comparison data set								
Attributes	−2	−1	0	1	2	−2	−1	0	1	2	Pi	LB	UB	Rank
Bicycle lanes	4	17	13	29	19	0.0009	0.0281	0.0517	0.2055	0.2017	0.4880	0.4242	0.5518	9
Traffic safety	1	18	15	25	23	0.0002	0.0297	0.0596	0.1772	0.2442	0.5109	0.4472	0.5747	4
Travel time	2	17	16	24	23	0.0004	0.0281	0.0636	0.1701	0.2442	0.5065	0.4427	0.5702	6
Comfort	4	16	19	25	18	0.0010	0.0264	0.0755	0.1772	0.1911	0.47124	0.4075	0.5350	10
Distance from the metro station	2	16	16	26	22	0.0050	0.0264	0.0636	0.1843	0.2336	0.5084	0.4447	0.5721	5
Parking facilities at the metro station	1	18	16	27	20	0.0002	0.0298	0.0636	0.1913	0.2123	0.4973	0.4335	0.5610	8
Terrain conditions	7	16	20	26	13	0.0017	0.0264	0.0795	0.1843	0.1380	0.4230	0.3662	0.4937	13
Bicycle rental charge	4	17	20	25	16	0.0010	0.0281	0.0795	0.1772	0.1699	0.4556	0.3919	0.5194	12
Bicycle rental deposit fees	5	17	17	25	18	0.0012	0.0281	0.0676	0.1772	0.1911	0.4652	0.4014	0.5289	11
Bicycle maintenance	4	11	19	28	20	0.0010	0.0182	0.0755	0.1984	0.2123	0.5055	0.4417	0.5692	7
Bicycle security at the metro station	2	16	11	28	25	0.0004	0.0264	0.0437	0.1984	0.2654	0.5345	0.4708	0.5983	2
Traffic congestion on the route	3	12	13	36	18	0.0007	0.0198	0.0517	0.2551	0.1911	0.5185	0.4547	0.5822	3
Weather (Summer/Winter)	4	12	8	17	41	0.0009	0.0198	0.0318	0.1205	0.4353	0.6084	0.5447	0.6721	1
f_j	43	203	203	341	276									
I/2 * f_j	21.5	101.5	101.5	170.5	138									
F_j	21.5	144.5	347.5	619.5	928									

(continued)

Table 2 (continued)

RIDIT Values specific to the reference-dataset						RIDIT values corresponding to the comparison data set								
Attributes	−2	−1	0	1	2	−2	−1	0	1	2	Pi	LB	UB	Rank
R_j	0.0202	0.1355	0.3260	0.5811	0.8705									

role towards choice of bicycle as a feeder to metro. Traffic condition of the road is another major factor, where congestion and risk from motorized vehicle act as a major barrier towards safe bicycling. Distance from metro station ($\rho_i = 0.5721$) and travel time ($\rho_i = 0.5064$) are ranked fifth and sixth respectively. Such findings clearly indicate the consistency in user responses, as they have perceived both travel time and access distance almost similarly. Factors such as bicycle maintenance ($\rho_i = 0.5692$), parking facilities at metro station ($\rho_i = 0.5610$) and bicycle lanes ($\rho_i = 0.5517$) are ranked seventh, eighth and ninth, respectively. Comfort ($\rho_i = 0.4712$), bicycle rental deposit fee ($\rho_i = 0.4651$), bicycle rental charge ($\rho_i = 0.4556$) and terrain conditions $\rho_i = 0.4299$ are perceived as least important attributes. These results indicate that users do not perceive bicycle rental charges or deposit as an important factor, and they are willing to use the facility with adequate fee. As the terrain conditions in Hyderabad are almost flat, it has been given the lowest ranking. Such findings could be judiciously used for bicycle-metro infrastructure planning in Indian context.

5 Conclusion

In this study, a detailed investigation on user perception towards various attributes influencing user's choice of bicycle as a feeder mode to metro has been investigated in detail. Based on the research findings and results, the following concluding remarks can be made.

Firstly, results clearly indicate that weather condition has been rated as the most important factor influencing commuter's choice decision to use bicycle or not. This finding could be ascribed to the fact that as India is a tropical country, the climate during summer will always be adverse and discourage bicycling. Hence, in such climatic condition, users should be given other incentives or better infrastructures to attract them towards bicycling. Secondly, it can also be inferred from the results that bicycle safety on the station was among the top-rated attributes influencing bicycle-metro integration. Such findings strengthen the need for developing secure bicycle parking facility at metro stations to attract more commuters to use metro-rails. Thirdly, threats from mixed traffic were also found to be a critical factor towards bicycle-metro integration, which should be carefully planned by appropriate traffic management measures. Fourthly, factors such as bicycle rental charge and bicycle maintenance were rated with relatively less priority compared to other factors, stating against the stereotype that bicycle is used because of its cost effectiveness. Hence, it can be inferred that users are willing to pay more to use bicycle as a feeder mode if appropriate bicycle parking facilities can be developed. Access distance to the metro station is also an important factor, as bicycle is only suitable as a feeder mode for short-to-medium distances. Results clearly indicate that access distance plays an important role towards influencing one's decision to choose bicycle as a feeder mode to metro in typical Indian condition.

Finally, the authors would like to mention that the methodology demonstrated and the survey instrument used in this study are generic and can be directly used for planning purposes in other cities. Furthermore, the factors prioritized in this study could serve as an initial basis for bicycle infrastructure planning for integration into metro rail in cities with similar urban characteristics.

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Road Safety

Rating and Prioritization of Crashes Black Spots and Road Safety Measures. Case Study: National Highway-44, India

Shawon Aziz, Pradeep Kumar Sarkar, and Jigesh Bhavsar

Abstract The study is focussed on star rating and prioritization of accident black spots based on severity and likelihood of crashes. The study also includes prioritization of road safety measures, economic analysis, and investment plan for the same. The case study is based on a stretch on NH 44 in Delhi urban area of 18.38 km road length. The study is carried out on sixteen identified black spots along the stretch identified by Delhi police. The data has been analyzed using International Road Assessment Program (*i*RAP) software, "ViDA" and star rating scores have been calculated separately for vehicle occupants pedestrians, motor cyclists, and bicyclists separately for all black spots. The findings suggest that an investment of 32.73 crores for road safety countermeasures on these 16 black spots on NH-44 can save 9235 fatal and serious crashes in 20 years with net present value of economic benefits 181.2 crores coupled with benefit–cost ratio of 5.58.

Keywords Crashes · Fatalities · Safety · Black spot · Star rating

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1 Introduction

Road accident is a public health issue which is spread worldwide. Citizens of nation deal with it on a day-to-day basis. As per Global Status report on Road Safety 2018 by World Health Organization, road traffic crashes is the 8th leading cause of death across the world and it is the main cause of death among those aged 5–29 years (World Health Organization 2018). Road crashes lead to death of 1.35 million people in the year 2016 (World Health Organization 2018). The Global Road Injury Mortality rate is 18 deaths per one lakh population and for India, it is 22.6. 74% of road traffic deaths occur in middle-income countries, which account for 70% of the world's population, but only 53% of the world's registered vehicles (World Health Organization 2018). As per National Crime Records Bureau, in India, 177,423 people died in road crashes and 413,547 people got seriously injured in 148,707 road accidents in the year 2015 (Ministry of Home Affairs, India 2015). Out of the 53 metropolitan cities, the fatalities per 100 crashes (accident severity) in the year 2014 were recorded highest in Ludhiana (66.9) followed by other metropolitan cities as shown in Table 1 (Ministry of Home Affairs, India 2015). The problem is dynamic and complex in nature and requires a collaborative effort of engineering, management, enforcement, and education and information technology measures. This paper deals with the engineering measure of black spot prioritization to address the issue of road safety.

2 Road Crash Scenario in Delhi

As per Delhi police records, since 2008 in past ten years, 75,750 crashes have occurred in Delhi in which 72,398 people have been victim of injuries while 18,835 people have lost their lives (Delhi Police 2015). Accidents reported in year 2017 were of the order of 6673 in which 1584 fatalities occurred and 6604 people were injured. Thus, there were a total of 8188 road accident victims as shown in Table 2 (Delhi Police 2017). This implies that victims who lost their lives accounted for 19.34% of the total. The decadal fatality rate of 2008–2017 is observed to be 20.65%

3 Introduction to International Road Assessment Program (*i*RAP)

The International Road Assessment Program (*i*RAP) is a non-profit organization that works in partnership with government and non-government organizations for safer roads (iRAP 2016). They have developed online software named "ViDA" which helps to assess star rating to roads in accordance with safety parameters. A road's star rating is based on an inspection of infrastructure elements that influence the likelihood and severity of crashes occurring on a road. Different road protection

Table 1 Top 10 metro cities road accident statistics

S. No.	Fatal crashes		Person killed		Person injured		Total crashes		Crash severity	
1	Delhi	1629	Delhi	1671	Chennai	9355	Mumbai	22,570	Ludhiana	66.9
2	Chennai	1083	Chennai	1118	Delhi	8283	Chennai	9610	Amritsar	57
3	Bengaluru	703	Bengaluru	729	Indore	4848	Delhi	8623	Kanpur	48
4	Kanpur	530	Kanpur	600	Bengaluru	4098	Indore	5784	Patna	41.3
5	Lucknow	515	Lucknow	537	Mumbai	3938	Bengaluru	5004	Lucknow	39.6
6	Mumbai	512	Mumbai	534	Kolkata	3604	Kolkata	4561	Surat	27.4
7	Patna	493	Agra	503	Mallapuram	3305	Bhopal	3459	Pune	25.4
8	Allahabad	454	Patna	493	Jabalpur	3074	Jabalpur	3124	Jaipur	22.5
9	Kolkata	431	Allahabad	481	Bhopal	2601	Hyderabad	2908	Delhi	19.4
10	Raipur	425	Kolkata	450	Hyderabad	2561	Mallapuram	2719	Ahmedabad	16

Table 2 Accident trend in Delhi (2008–2017)

Year	Non-injurious accident	Non-fatal accidents	Fatal accident	Total accidents	Persons injured	Persons killed
2008	362	6058	2015	8435	7343	2093
2009	131	5113	2272	7516	6936	2325
2010	63	5093	2104	7260	7108	2153
2011	71	5162	2047	7280	6975	2110
2012	115	5000	1822	6937	6639	1866
2013	169	5619	1778	7566	7098	1820
2014	209	6785	1629	8623	8283	1671
2015	160	6343	1582	8085	8258	1622
2016	129	5698	1548	7375	7154	1591
2017	91	5017	1565	6673	6604	1584
Total	1500	55,888	18,362	75,750	72,398	18,835

Source https://delhitrafficpolice.nic.in/about-us/statistics/

scores are produced for car occupants, bicyclists, pedestrians, and motorized two-wheeler riders. Following the inspections of the road infrastructure elements, star rating score (SRS) is calculated for each 100 m section of road using the ViDA software (iRAP 2016). The SRS forms the basis for generating the star ratings (and, in turn, Safer Roads Investment Plans). A high score equates with a high level of risk, and a low score equates with a low level of risk (Table 3).

After star rating the model is calibrated, an estimate of the fatalities and serious injuries at any point on the network can be made. This information can then be utilized to examine the potential of reducing fatalities and serious injuries through application of proven engineering measures at all locations throughout the stretch. The countermeasures are also subject to a hierarchy, with the countermeasures having higher benefit–cost ratio taking precedence. This ensures that there is no duplication of treatments that impact the same road feature. This is followed by a comprehensive road safety investment plan in which unit rates of every countermeasure treatment are taken by applying wholesale price index of current year schedule of rates provided by Public Works Department (PWD) India.

Table 3 Star rating corresponding to mode-wise star rating score (iRAP Methodology Fact Sheet 2018)

Star ratings	Star rating scores				
	Vehicle occupants and motorcyclists	Bicyclists	Pedestrians		
			Total	Along	Crossing
5 Stars	0 to <2.5	0 to <5	0 to <5	0 to <0.2	0 to <4.8
4 Stars	2.5 to <5	5 to <10	5 to <15	0.2 to <1	4.8 to <14
3 Stars	5 to <12.5	30 to <30	15 to <40	1 to <7.5	14 to <32.5
2 Stars	12.5 to 22.5	30 to <60	40 to <90	7.5 to <15	32.5 to <75
1 Star	22.5+	60+	90+	15+	75+

4 Measures to Improve Road Accidents

The basic principle of road safety is to improve "driver expectancy" by removing ambiguity in road environment. The geometric design of roads must be consistent without any abrupt change. Further special care must be taken for the protection of vulnerable road users like pedestrians and bicyclists. The major road safety measures that have been applied to the accident black spots have been briefly described in Table 4. This has been worked out on the *i*RAP guidelines.

5 Case Study: Delhi NCT (16 Black Spots Along NH-44)

As per Delhi police, an accident black spot is defined as a location on road where three or more fatal accidents or more than nine non-fatal accidents take place within a year. Out of 137 black spots identified by Delhi Police, 16 black spots lying within the Delhi Urban area along an 18.38 km stretch on National Highway-44 have been studied and more than 90 road attributes have been recorded such as delineation, median type, pavement condition, curve sharpness, and street lighting. The black spots are shown in Fig. 1.

As shown in Table 5, 197 accidents have taken place on these black spots in the year 2015 in which 84 people have died while 113 faced serious injuries. The road inventory survey, traffic volume count bicycle and pedestrian peak hour flow surveys were carried out at all 16 accident black spots. After collection of data from primary and secondary sources, it was coded as per *i*RAP Star Ratings and Investment Plans: Coding Manual. The coded data was then processed using ViDA software of *i*RAP to get star rating and safer road investment plan for all the black spots.

6 Black Spot Analysis

6.1 Near Azadpur Bus Terminal

Inference

Analysis results from Table 6 show that the 55% of road stretch near bus terminal experiences 1 star rating for four-wheeler riders and motorcyclists while the 95% of the stretch exhibits 2 star rating for bicyclists and the 60% of the stretch also exhibits 2 star rating for pedestrians.

Table 4 Major engineering treatments (iRAP Toolkit 2017a, b)

S. No.	Safety measure	Cost	Life years	Effectiveness (%)	Remark
1	Additional lane, lane widening	High	10–20	25–40	Provides a safe opportunity for one direction of traffic to overtake and can improve traffic flow
2	Road safety barriers	Medium	10–20	40–60	Flexible, rigid or semi-rigid barriers made of steel, rails etc.
3	Delineation	Low	1–5	10–25	Line markings, retro-reflective markers, guideposts, warning signs, chevron signs
4	Signalized intersections	Medium	10–20	25–40	Vehicle actuated signals with dynamic signal time cycle and pedestrian phase
5	Pedestrian facilities	Medium	10–20	40–60	Pedestrian fencing, refuge islands, footpaths
6	Shoulder sealing	Medium	5–10	25–40	Reduces head on crashes, better grip, and serve as safe cycling space
7	Traffic calming measures	Medium	10–20	25–40	speed humps, table topping, kerb built-outs, roundabouts
8	Speed management	Medium	5–10	25–40	Roundabouts, gateway treatments, pavement narrowing, curve treatments
9	Grade separated pedestrian crossings	High	10–20	Above 60	Eliminate pedestrian–vehicular conflicts
10	Service lanes and bicycles lanes	High	10–20	40–60	Safe movement of vulnerable road users

6.2 *Near Azadpur Junction*

Inference

Analysis results from Table 7 show that the 50% of road stretch near Azadpur Chowk has 1 star rating for four-wheeler riders and motorcyclists. While the entire stretch has 1 star rating for pedestrians and 90% of stretch has 2 star rating for bicyclists.

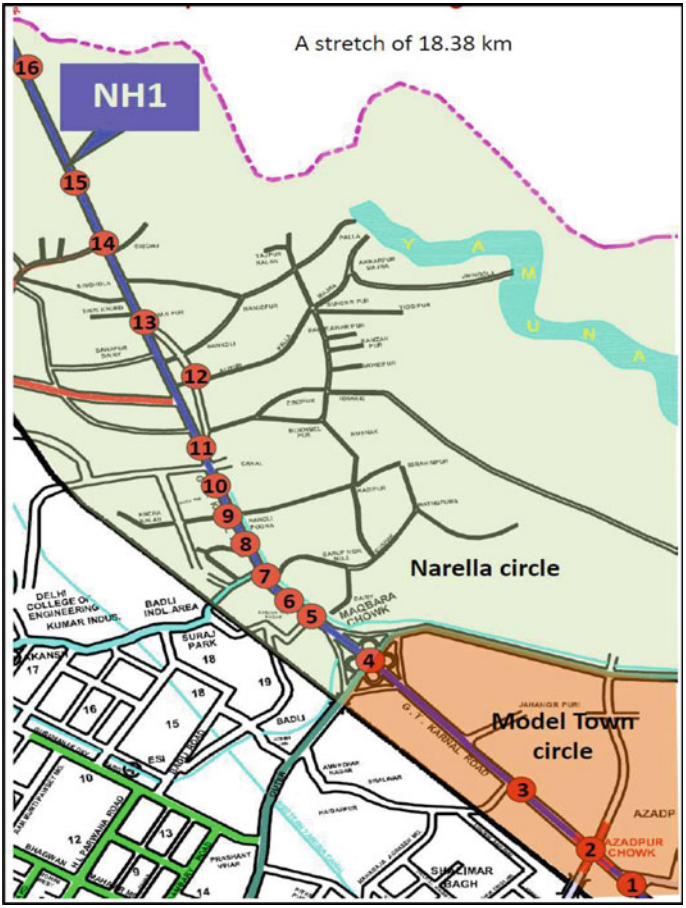

Fig. 1 Black spots along NH-44 from Azadpur to Singhu border

6.3 Near Azadpur Sabzi Mandi

Inference

Analysis results from Table 8 show that 64% of road stretch near Azadpur vegetable market has 1 star rating for four-wheeler riders and motorcyclists, while the entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

Table 5 Black spots NH-44 from Azadpur to Singhu border (Delhi Police 2015)

S. No.	Accident-prone zones	Non-fatal accident	Fatal accident	Total accidents
1	Azadpur bus terminal	10	1	11
2	Azadpur Chowk	10	8	18
3	Azadpur Sabzi Mandi	10	9	19
4	Mukarba Chowk	4	8	12
5	SGT nagar	7	8	15
6	CNG pump SGT nagar	3	8	11
7	Libaspur bus stand	7	3	10
8	Swaroop nagar	6	4	10
9	Nangli Budhpur	3	7	10
10	Budhpur Ganda Nala	9	4	13
11	Sai Baba Mandir	6	5	11
12	Shani Mandir	6	3	9
13	Bakoli bus stand	11	3	14
14	Khampur village	6	3	9
15	Tikri Khurd village	8	5	13
16	Singhu border	7	5	12
	Total	113	84	197

Table 6 Mode-wise star rating on NH-44 near Azadpur bus terminal

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0.5	25	0.5	25	0.8	40	0	0
2 Star	0.40	20	0.4	20	1.2	60	1.9	95
1 Star	1	55	1.1	55	0	0	0.1	5
Total	**2**	**100%**	**2**	**100%**	**2**	**100%**	**2**	**100%**

Table 7 Mode-wise road star rating on NH-44 near Azadpur junction

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0.5	50	0.5	50	0	0	0	0
2 Star	0.00	0	0	0	0	0	0.9	90
1 Star	1	50	0.5	50	1	100	0.1	10

Table 8 Mode-wise star rating on NH-44 near Azadpur Sabzi Mandi

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0.4	29	0.2	14	0	0	0	0
2 Star	0.10	7	0.3	21	0	0	1.4	100
1 Star	1	64	0.9	64	1.4	100	0	0
Total	**1.4**	**100%**	**1.4**	**100%**	**1.4**	**100%**	**1.4**	**100%**

Table 9 Mode-wise star rating on NH-44 near Mukarba junction

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	1.88	90	0	0	0	0	0	0
3 Star	0	0	1.88	73	0	0	0	0
2 Star	0.20	10	0.2	8	0	0	2.08	100
1 Star	0	0	0.5	19	2.08	100	0	0
Total	**2.08**	**100%**	**2.58**	**100%**	**2.08**	**100%**	**2.08**	**100%**

6.4 *Near Mukarba Junction*

Inference

Analysis results from Table 9 show that 90% of stretch at Mukarba Chowk has 4 star rating for four-wheeler riders and 3 star rating for motorcyclists, while the entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

6.5 *Near Sanjay Gandhi Transport Nagar (SGT Nagar)*

Inference

Analysis results from Table 10 show that 67% of the stretch at Swaroop Nagar has 3 star rating for four-wheeler riders and motorcyclists, while the entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

Table 10 Mode-wise star rating on NH-44 near SGT nagar

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
4 Star	0	0	0	0	0	0	0	0
3 Star	2.2	67	2.2	67	0	0	0	0
2 Star	1.10	33	1.1	33	0	0	3.3	100
1 Star	0	0	0	0	3.3	100	0	0
Total	**3.3**	**100%**	**3.3**	**100%**	**3.3**	**100%**	**3.3**	**100%**

Table 11 Mode-wise star rating on NH-44 near SGT nagar CNG pump

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	1.3	100	0.7	54	0	0	0	0
2 Star	0.00	0	0.6	46	0	0	1.3	100
1 Star	0	0	0	0	1.3	100	0	0
Total	**1.3**	**100%**	**1.3**	**100%**	**1.3**	**100%**	**1.3**	**100%**

Table 12 Mode-wise star rating on NH-44 near Shani Mandir

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0	0	0	0	0	0	0	0
2 Star	1.88	90	1.88	90	0	0	2.08	100
1 Star	0.20	10	0.20	10	2.08	100	0	0
Total	**2.08**	**100%**	**2.08**	**100%**	**2.08**	**100%**	**2.08**	**100%**

6.6 Near SGT Nagar CNG Pump

Inference

Analysis results from Table 11 show that entire stretch SGT Nagar has 3 star rating for four-wheeler riders. 54% of the road stretch has 3 star rating for motorcyclists, while the entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

6.7 Near Shani Mandir

Inference

Analysis results from Table 12 show that entire stretch near Shani Mandir has 2 star rating for vehicle occupants and motorcyclists, while the entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

6.8 Libaspur Bus Stand

Inference

Analysis results from Table 13 show that entire stretch near Libaspur has 2 star rating for bicyclists and 1 star rating for pedestrians. 74% of the road stretch has 3 star rating for motorcyclists and four-wheeler riders.

Table 13 Mode-wise star rating on NH-44 near Libaspur bus stand

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	2	74	2	74	0	0	0	0
2 Star	0.10	4	0.1	4	0	0	2.7	100
1 Star	0.6	22	0.6	22	2.7	100	0	0
Total	**2.7**	**100%**	**2.7**	**100%**	**2.7**	**100%**	**2.7**	**100%**

Table 14 Mode-wise star rating on NH-44 near Swaroop nagar

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	2.2	67	2.2	67	0	0	0	0
2 Star	1.10	33	1.1	33	0	0	3.3	100
1 Star	0.0	0	0	0	3.3	100	0	0
Total	**3.3**	**100%**	**3.3**	**100%**	**3.3**	**100%**	**3.3**	**100%**

6.9 Swaroop Nagar

Inference

Analysis results from Table 14 show that entire stretch near Swaroop Nagar has 2 star rating for bicyclists and 1 star rating for pedestrians. 67% of the road stretch has 3 star rating for motorcyclists and four-wheeler riders while 33% of the stretch has 2 star rating for motorcyclists and four-wheeler riders.

6.10 Nangli Budhpur Village

Inference

Analysis results from Table 15 show that entire stretch near Nangli Budhpur Village has 2 star rating for four-wheelers and motorcycles, while for pedestrians and bicyclists, the entire stretch has 1 star rating.

Table 15 Mode-wise start rating on NH-44 near Nangli Budhpur village

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
4 Star	0	0	0	0	0	0	0	0
3 Star	0	0	0	0	0	0	0	0
2 Star	2.40	100	2.4	100	0	0	0	0
1 Star	0.0	0	0	0	2.4	100	2.4	100
Total	**2.4**	**100%**	**2.4**	**100%**	**2.4**	**100%**	**2.4**	**100%**

Table 16 Mode-wise star rating on NH-44 near Sai Baba Mandir

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0	0	0	0	0	0	0	0
2 Star	2.70	100	2.7	100	0	0	2.7	100
1 Star	0.0	0	0	0	2.7	100	0	0
Total	**2.4**	**100%**	**2.7**	**100%**	**2.7**	**100%**	**2.7**	**100%**

Table 17 Mode-wise star rating on NH-44 near Budhpur Ganda Naala

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	1	91	1	91	0	0	0	0
2 Star	0.10	9	0.1	9	0	0	1.1	100
1 Star	0.0	0	0	0	1.1	100	0	0
Total	**1.1**	**100%**	**1.1**	**100%**	**1.1**	**100%**	**1.1**	**100%**

6.11 Sai Baba Mandir

Inference

Analysis results from Table 16 show that entire stretch near Sai Baba Mandir has 2 star rating for bicyclists, four-wheelers and motorcycles and 1 star rating for pedestrians.

6.12 Budhpur Ganda Naala

Inference

Analysis results from Table 17 show that 91% of the stretch near Budhpur Ganda Naala has 3 star rating for vehicle users and motorcyclists while entire stretch has 2 star rating for bicyclists and 1 star rating for pedestrians.

6.13 Bakoli Bus Stand

Inference

Analysis results from Table 18 show that entire stretch has 3 star rating for vehicle occupants, while 80% of the stretch has 3 star rating for motorcycles. Entire stretch has 2 star rating for bicyclists and 1 star rating for pedestrians.

Table 18 Mode-wise star rating on NH-44 near Bakoli bus stand

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	4.1	100	3.3	80	0	0	0	0
2 Star	0.00	0	0.8	20	0	0	4.1	100
1 Star	0.0	0	0	0	4.1	100	0	0
Total	**4.1**	**100%**	**4.1**	**100%**	**4.1**	**100%**	**4.1**	**100%**

Table 19 Mode-wise star rating on NH-44 near Khampur village

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	2.97	88	2.97	88	0	0	0	0
2 Star	0.10	3	0.1	3	0	0	3.07	91
1 Star	0.3	9	0.3	9	3.37	100	0.3	9
Total	**3.37**	**100%**	**3.37**	**100%**	**3.37**	**100%**	**3.37**	**100%**

6.14 Khampur Village

Inference

Analysis results from Table 19 show that 88% of the road stretch has 3 star rating for vehicle occupants and motorcyclists while entire stretch has 1 star rating for pedestrians and 91% of the stretch has 2 star rating for bicyclists.

6.15 Tikri Khurd Village

Inference

Analysis results from Table 20 show that 91% of the road stretch near Tikri Khurd Village has 3 star rating for vehicle occupants and motorcyclists while entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

Table 20 Mode-wise star rating on NH-44 near Tikri Khurd village

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	2.1	91	2.1	91	0	0	0	0
2 Star	0.20	9	0.2	9	0	0	2.3	100
1 Star	0.0	0	0	0	2.3	100	0	0
Total	**2.3**	**100%**	**2.3**	**100%**	**2.3**	**100%**	**2.3**	**100%**

Table 21 Mode-wise star rating on NH-44 near Singhu border

Star rating	Vehicle occupant		Motor cycle		Pedestrian		Bicycle	
	Length (km)	%	Length (km)	(%)	Length (km)	%	Length (km)	%
5 Star	0	0	0	0	0	0	0	0
4 Star	0	0	0	0	0	0	0	0
3 Star	0	0	2.1	64	0	0	0	0
2 Star	0.60	50	0.6	18	0	0	1.2	100
1 Star	0.6	50	0.6	18	1.2	100	0	0
Total	1.2	100%	3.3	100%	1.2	100%	1.2	100%

6.16 Singhu Border

Inference

Analysis results from Table 21 show that half of the stretch has 1 star rating and other half has 2 star rating for vehicle occupants and motorcyclists. While entire stretch has 1 star rating for pedestrians and 2 star rating for bicyclists.

Prioritization of Road Safety Measures and Road Investment Plan

The road investment plan shows a list of affordable and economically sound road safety treatments. Each countermeasure proposed is supported by strong evidence that, if implemented, it will prevent deaths and serious injuries in a cost-effective way. Countermeasure cost per fatal and serious injury (FSI) along with benefit–cost ratio (BCR) has been indicated in Table 22. The most cost-effective measure offers the highest BCR and is assigned with the highest priority followed by other countermeasures. All costs shown in Table 22 are in Indian Rupee (INR).

Inference

An investment of about 0.3273 billion Indian rupees on this 18 km stretch on National Highway 44 can save 9235 fatal and serious injuries in coming 20 years of time with net present value of economic benefits of 1.812 billion Indian Rupees and benefit–cost ratio of 5.53. The average cost of countermeasures per fatal and serious injury comes out to be 35,439 Indian rupees (INR).

7 Prioritization of Black Spots

Prioritization of black spots has been carried out on economic analysis parameters, i.e., cost saved per fatal and serious injury saved (FSI) and benefit–cost ratio (BCR) which is shown in Table 23. The black spots have been arranged in hierarchy of decreasing benefit–cost ratio in Table 23. The black spot with the highest priority

Table 22 Safer road investment plan for 20 years all 16 black spots

S. No.	Road safety countermeasures (In decreasing order of priority with regard to benefit–cost ratio of investment in the countermeasure)	Length/Sites	Fatal and serious injuries (FSI) saved	Present value (PV) of safety benefit (millions INR)	Estimated cost (millions INR)	Cost per FSI saved	Benefit–cost ratio (BCR)
1	Pedestrian fencing (highest priority)	3.10 km	876	171.95	0.06	755	260
2	Improve curve delineation	1.40 km	116	22.7	0.04	2427	81
3	Shoulder rumble strips	8.27 km	294	57.7	1.06	3615	54
4	Footpath provision passenger side	29.3 km	1683	330.16	31.62	18,794	10
5	Signalized crossing	26 sites	1620	317.82	34.21	21,124	9
6	Traffic calming	18.39 km	3300	647.6	4.75	14,402	14
7	Improve delineation	32.76 km	641	125.76	16.90	26,379	7
8	Signalized intersection	27 sites	715	140.27	27.2	40,755	5
9	Upgrade pedestrian facility	3 sites	19	3.7	0.8	41,101	5
10	Delineation at intersection	27 sites	680	133.5	36.97	54,358	4
11	Protected turn lane	9 sites	409	80.24	20.47	50,075	4

(continued)

Table 22 (continued)

S. No.	Road safety countermeasures (In decreasing order of priority with regard to benefit–cost ratio of investment in the countermeasure)	Length/Sites	Fatal and serious injuries (FSI) saved	Present value (PV) of safety benefit (millions INR)	Estimated cost (millions INR)	Cost per FSI saved	Benefit–cost ratio (BCR)
12	Motorcycle lane (painted logos)	3.4 km	12	2.4	0.58	46,553	4
13	Roadside barriers	6.07 km	251	49.28	14.84	59,107	3
14	Protected turn lane improvement	5 sites	127	24.95	8.07	63,511	3
15	Clear roadside hazards	4.70 km	123	24.20	8.80	71,377	3
16	Restrict/combine direct access points	2.5 km	98	19.28	6.46	66,039	3
17	Off-road bicycle lane	3.4 km	45	8.83	3.33	74,058	3
18	Road barrier (driver side)	0.5 km	19	3.79	1.32	68,591	3
19	Grade separated pedestrian facility	1	210	41.26	21.99	104,601	2
21	Signalize side road	8 sites	122	23.88	39.10	321,183	1
22	Segregated motorcycle lane	3.07 km	108	6.89	5.72	162,916	1
23	Shoulder sealing	0.47	5	1.25	1.07	35,439	1
Total			9235	1.812 billion	0.3273 billion	35,439	5.53

Table 23 Prioritization of black spots based on benefit–cost ratio

Prioritization hierarchy	Black spot	FSI saved	PV of FSI saved (million INR)	Estimated cost (million INR)	Cost per FSI saved	BCR
1	Shani Mandir	974	191.13	20.05	20,539	10
2	Bakoli bus stand	94	18.49	2.13	22,583	9
3	Azadpur bus stand	170	33.41	4.00	23,530	8
4	SGT petrol pump	487	95.58	12.78	26,247	7
5	Tikri Khurd village	462	90.67	13.09	28,334	7
6	Swaroop nagar	858	168.45	26.72	31,130	6
7	Khampur village	735	144.25	23.43	31,880	6
8	Nangli Budhpur	1430	280.5	48.93	34,233	6
9	Singhu border	655	128.61	23.13	35,295	6
10	Azadpur Sabzi Mandi	615	120.61	22.02	35,834	5
11	Budhpur Ganda Nala	538	105.65	22.56	41,914	5
12	Sai Baba Mandir	673	132.05	29.05	43,172	5
13	Azadpur Chowk	247	48.38	13.17	53,449	4
14	Mukarba Chowk	565	110.93	33.50	59,262	3
15	SGT nagar	104	20.36	7.62	73,474	3
16	Libaspur bus stand	227	18.25	6.02	80,254	3

has been shown to top followed by others and black spot with the lowest priority is shown at the bottom.

Inference

Prioritization of black spots suggests that Shani Mandir on NH-44 can be accorded with highest priority with cost per FSI saved 20,539 rupees and BCR 10. Nine hundred seventy-four fatal and serious injuries can be prevented in 20 years of time on NH-44 stretch near Shani Mandir. Table 23 presents the measures for prioritization in the form of hierarchy of all 16 black spots.

8 Conclusion

As shown in Fig. 2, the star ratings get improved for all kinds of road users on NH-44 after application of road safety improvement plan. Initially, on 16 black spots along NH-44 corridor, the 67% of road stretch exhibited 3 star rating for vehicle occupants while the 62% of the stretch experienced with 3 star rating for motorcyclists. All the black spots reflected 2 star rating for bicycle riders and 1 star rating for pedestrians. After application of countermeasures, it can be stated that nearly 20% of the road stretch along the black spots enjoys 5 star rating for four-wheeler riders and motorcyclists while the 80% of the stretches experienced a 4 star rating for pedestrian safety. As far as the 10% of the stretch of the 16 black spots, achieves 5 star rating. For bicyclists, the 50% of the stretch of black spots can attain 3 star rating while the 15% of the black spot stretches can be improved to 3 star rating along with 8% of the stretch gaining 5 star rating.

An investment of about 0.3273 billion Indian Rupees ($5 million US Dollar) Indian rupees on black spots of this 18 km stretch on National Highway 44 can save 9235 fatal and serious injuries in coming 20 years of time with net present value of economic benefits of 1.812 billion Indian Rupees ($28.46 million US Dollars) and benefit–cost ratio of 5.53. Prioritization of road safety countermeasures suggests that pedestrian fencing is accorded with the highest priority with BCR 260 and lowest FSI followed by improved curve delineation and others. Prioritization of road safety countermeasures suggests that pedestrian fencing would enjoy the highest priority with BCR 260 and lowest FSI, followed by improved curve delineation and others prioritization of black spots suggests Shani Mandir on NH-44 corridor experience the highest priority with cost per FSI saved 20539 supported with BCR 10. Nine hundred seventy-four fatal and serious injuries can be prevented in 20 years of time on NH-44 stretch near Shani Mandir. Table 23 presents the prioritization in the form of hierarchy of all 16 black spots.

The methodology used in this work with the help of *i*RAP for prioritization of road safety countermeasures as well as prioritization of accident black spots can serve as tool for comparison of safety measures and black spots at a global level on

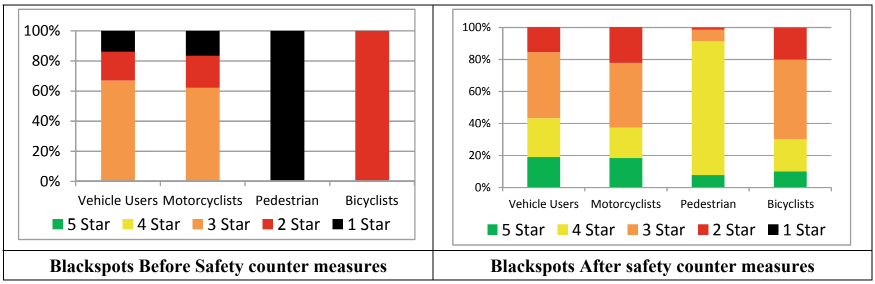

Fig. 2 Combined star rating chart for NH-44 road stretch before and after road safety countermeasures

a common platform. Further, the road investment plan and estimation of fatalities saved can help various government organizations to carry out road safety investment plans in an efficient manner through scientific approach.

9 Way Forward

Star rating is not sensitive toward assessing potential impact of intelligent transport system (ITS) measures. Research on impact of ITS in infrastructure can be conducted and integrated with road infrastructure rating to further enhance the values of road safety. Diurnal variation in star rating of road infrastructure components and corresponding countermeasures can further make roads safer.

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Risk Analysis for a Four-Lane Rural Highway Based on Safety Audit

Ayush Dhankute and Manoranjan Parida

Abstract With over 5.4 lakh kilometres of roads and the number of motor vehicles that have crossed the mark of 2.1 lakh thousand in 2015 along with urbanisation, road crashes emerge as complementary. But the fact that the number of crashes and injuries has dropped and on the contrary, the fatalities and severity of accident have inflated by 4.7% in 2016 with respect to the previous year, the severity is the factor that has been rising. Thus, the study focuses on determining Risk Index by proactive measure for rural highway which is a quantified value based on various factors such as exposure, crash severity and probability, as per various researchers. The study was carried out only on National Highway with reliable accident data as they are major contributors to road crashes, i.e. one-third of total crashes occur on National Highways which constitute only 2% of the road network in India. The factors such as geometric design and road infrastructure are considered for Safety Index calculation and further, the validation is based upon the historically available authentic accident data. The study results show a good correlation between ranking of the road segments as per Risk Index and based upon available road accident data. This validated technique can be used to determine risk on the road without relying on the unreliable accident data and safety measures can be incorporated on the road segments as per priority which is determined based upon the Risk Index of the highway segments.

Keywords Risk index · Road safety audit · Exposure · Probability · Severity

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1 Introduction

1.1 General

Conventionally, road traffic safety has been seen as an extent of the number of road traffic crashes and victims resulting out as consequence from crashes per time period, generally for the duration of heavy traffic flow. The frequency of crashes and fatalities are also conveyed in terms of rates, such as the number of casualties per number of registered vehicles, vehicle kilometres travelled and the human population. These current measures are mainly based upon unforeseen random incidents such as crashes and casualties. But a combination of factors related to the constituents of the system consisting of roads, the environment, vehicles and road users, and the way they interact lead to a road traffic crash. Hence, the term Road Safety Index (RSI) or simply Road Risk Index (RI) has come up which take all the above factors into consideration while evaluating a road. This Risk Index is mainly depended upon the Road Safety Audit (RSA) of the road.

Based on the review of literature, it is inferred that RSI has been defined in various ways. As such, RSI has been related to the hazard originating from road infrastructure's features such as density of intersections or accesses, road surface anomalies, road signs, road markings, and barriers. Rosolino et al. (2014) RSI in one of the literatures is established as a driver centred, subjective valuation of the possible road safety risks for in-service highways. RSI is also defined as the index that measures the relative safety performance of a road segment quantitatively and is expressed by merging different components or factors of risk such as

- Factors influencing exposure to risk such as length of trip and mode of travel which is influenced by land-use planning practices, mixture of exposed road users with high-speed motorised traffic, inadequate attention to incorporation of road function with decisions about road layout, speed limit and design, etc.
- Risk elements persuading crash contribution such as unsuitable and unwarranted speed, being a vulnerable road user in residential and urban areas, vehicle factors—such as handling and maintenance, braking, defects in road design, layout and upkeep, insufficient visibility because of environmental factors.
- Risk factor influencing crash severity such as unwarranted speed, unsatisfactory vehicle crash protection for occupants and for those hit by vehicles, roadside objects not crash protective, etc.

1.2 Need for the Study

As discussed earlier, there has been no development of any kind of RSI or RI based on Safety Audit at least in India. This index should be able to recognise potential threats, which are evaluated by measuring risk in relation to road features that may act as a consequence to future crashes, so that counteractive treatments may be executed prior to the crashes. From this index, the safety-related concerns and recommendations for improvement should be reached upon.

The feature of almost all the safety management agendas comprises of a "collision-prone location" program, where substantial accident history must be present and identified before improvements are suggested. Often, these programs are exclusively crash records dependent and thus program success is administered by the data quality. Unfortunately, in various jurisdictions in India or also in various parts of the world, the collision data have been degrading for several years both quantitatively as well as qualitatively. This mounting problem is endangering the accomplishment and endurance of various road safety agendas. To help diminish this problem, it is alleged that a subjective assessment modus operandi could be developed that will be irrespective of collision statistics and that could be used to detect and diagnose problematic areas. Investigation of crashes is a program of reactive approach, it examines former crashes and aims to eradicate or change the features that backed to those past collisions. RSI developed from safety appraisal or audit is a program of proactive approach, aimed at decreasing road crashes well in advance of their occurrence. Road crash investigation process has a tendency to focus on particular locations, while safety assessments are more analogous to mass action study methodologies. In countries with meagre accident figures, the significance of safety reviews as an accompaniment to accident investigation methodologies becomes more imperative. Undeniably, fewer the crash data figures, lesser is the information crashes can give about crashes to be vetoed.

Another approach may be "Traffic Conflict Technique". A common description of a traffic conflict is "an noticeable situation in which two or more road users approach each other in space and time for such an extent that there is a threat of collision if their respective movements remain unaltered" (Leur and Sayed 2002). Multiple observation methods have been established for the measurement of traffic conflicts. These methods can be categorised as subjective or objective. Subjective methods comprise of substantial decision by the conflict spectator and are disapproved by a number of researchers because the rating of severity of the oblique action can fluctuate prominently from one observer to another. Objective methods take account of a cardinal time-proximity or ordinal time-proximity dimension in the severity scale. The most extensively used measure is the time to collision, described as the time for two vehicles to collide if they continue at their contemporary speed and on the same path. However, it is noted that the traffic conflict practice necessitates the interaction of two vehicles and thus may not be useful for rural roads where many incidents are single-vehicle, off-road incidents.

1.3 Objectives of the Study

- To develop an analysis method for the development of Road Risk Index for a four-lane divided highway having a good correlation with certain objective measures of safety.

- Initially, the method has to be developed on the road where accident data is properly being maintained and updated regularly, so that the method developed could be validated with the help of the accident data.
- Once the mechanism of developing Risk Index is validated, the same modus operandi can be used on all other roads even with poor accident data as there is no need for revalidation.

2 Literature Review

According to various literatures, Road Safety Index (RSI) has been defined by various ways and Risk Index has been related to the risk arising out from road infrastructures such as density of intersections or accesses, road surface anomalies, road signs, road markings, and barriers (Rosolino et al. 2014). The proposed method was based on the identification of five classes of factors, viz. accident history, access density, road surface anomalies, problems in road signs, roadside safety barriers and weights to be assigned to each class. Risk Index in one of the other literatures was established as a driver centred, subjective evaluation of the impending road safety risks for in-service highways (Leur and Sayed 2002). The Risk Index here was developed on this basis, risk is function of likelihood, exposure and consequence where exposure is based on traffic volume, probability is evaluated separately for each road feature to assess its potential in causing accident, and consequence is mostly influenced by vehicle speed factor. The outcomes from the Risk Index were associated with outcomes from quantitatively derived road safety measures to evaluate the accomplishment of the Road Safety Risk Index.

RSI is also defined (Montella 2005) as the index that objectively measures the comparative safety performance of a road segment and is expressed by merging different components or factors of risk such. A methodical process to conclude which road features should be considered and how each feature should be assessed during the review is described. The technique discourses rural two-lane highways at non-intersections. From the procedure, a potential for a Safety Improvement Index (PFI) was designed. Authentication of the procedure was carried out by an association of the PFI values with the anticipated collision frequency. Another researcher defined Road Safety Index as the product of exposure factor, accident frequency factor and accident severity factor (Cafiso et al. 2007a, b). Exposure factor is based on length and AADT of the considered segment, accident frequency factor is based on road safety inspection factors such as accesses, delineation, cross section of road, etc. and geometric design factors such as curve radius and curve length. Accident severity factor is based on roadside hazards, operating speed of vehicles, design speed, etc. The SI was calculated in various segments selected from a sample of two-lane rural highways in the European country of Italy, and the authentic accident condition was obtained with the empirical Bayes (EB) procedure. To determine the level of agreement between the rankings obtained with the two techniques, Spearman’s rank correlation was put to use.

Harwood et al. (2000) gave the accident modification factors (AMF) which are applied to determine the comparative increase in the accident risk (ΔAF) due to issues such as access and road cross section as these factors were used by various researchers to determine the Road Risk Index. Direct accesses to roads can considerably increase crashes. Access point sites can be very hazardous (e.g. accesses on horizontal curves). AMFs that take into consideration driveway density have been established, they show the considerable effect of accesses on road safety. Report on Safety Audit of Existing roads New Zealand (2003) focuses on the area of effect of road marking, delineations, road signs and pavement skid resistance on crashes, i.e. the relative increase in accident risk due to above-mentioned factors. Detailed items considered are centre line missing or inadequate, edge lines missing or inadequate and no overtaking line missing in sections where overtaking sight distance is not provided. Report on Roadside Design Guide AASHTO (1996) reflects on the area of roadside safety while evaluating roadside items comprising of crash barriers, embankments, bridges, dangerous barrier terminals and transition, utility poles, trees, ditches, etc. This report also gives the relative surge in the accident severity due to roadside issue.

3 Methodology

See Fig. 1.

4 Data Collection

4.1 Site Selection

A 65 km stretch of a four-lane divided road of National Highway 6 (old numbering NH-53 new numbering) was selected as shown in Fig. 2. Start point was 20°50′11.00″ N and 77°42′54.20″ E near Amravati (Maharashtra) and endpoint was 21°6′13.00″ N and 78°10′55.68″ E near Talegaon (Maharashtra) about 100 km from Nagpur towards Mumbai. The complete section was divided into 13 segments of 5 km each. Site was selected such that complete record of accident data and every other data was maintained and updated timely.

4.2 Accident Data

Crash data from May 2013 to May 2017 was obtained from IRB infrastructure Ltd., the construction company, and also from National Highway Authority of India.

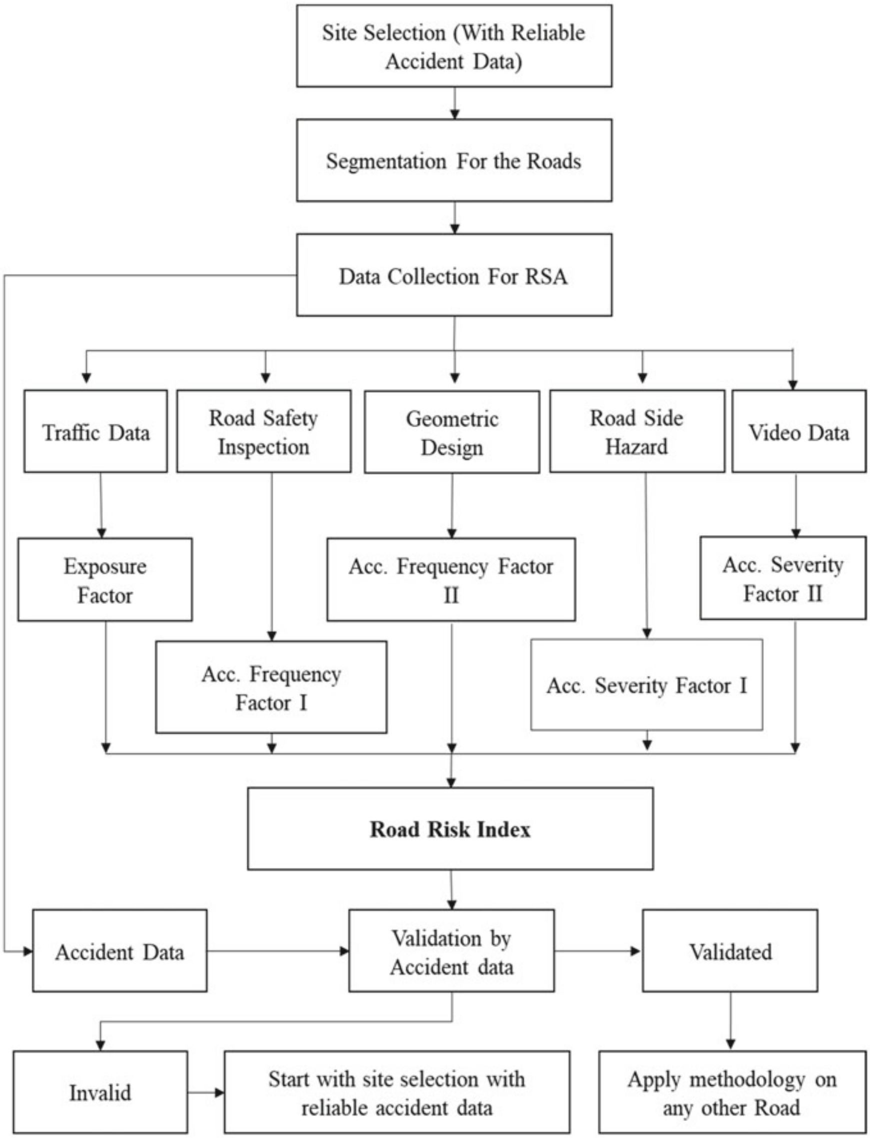

Fig. 1 Methodology flow chart

Detailed information of about all the crashes including accident date, time, chainage, location, nature of accident, nature of injuries, causes of crashes, road features, road conditions, weather condition, vehicles involved with their registration number, number of fatalities, minor, grievous injuries, animals killed, type of help provided, etc., was provided. From this data, few analyses are made as follows (Table 1; Figs. 3,

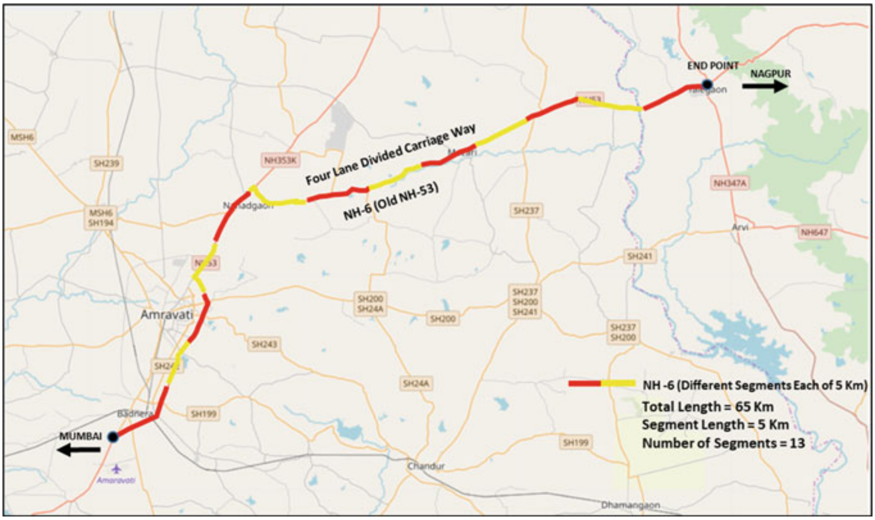

Fig. 2 Location of site selected for the study

Table 1 Site accident data for NH 53

Number of crashes						
Segments chainage (S. No.)	2013	2014	2015	2016	2017	Total
0—5 km (1)	5	4	4	3	0	16
5–10 km (2)	1	4	1	3	0	9
10–15 km (3)	5	3	4	1	1	14
15–20 km (4)	11	21	12	15	2	61
20–25 km (5)	14	14	12	10	4	54
25–30 km (6)	14	9	9	5	5	42
30–35 km (7)	7	6	10	6	2	31
35–40 km (8)	10	4	6	8	0	28
40–45 km (9)	5	6	4	2	0	17
45–50 km (10)	8	9	6	4	1	28
50–55 km (11)	0	0	2	1	0	3
55–60 km (12)	6	4	3	4	1	18
60–67 km (13)	7	2	3	2	1	15
Total	93	86	76	64	17	336

4 and 5):

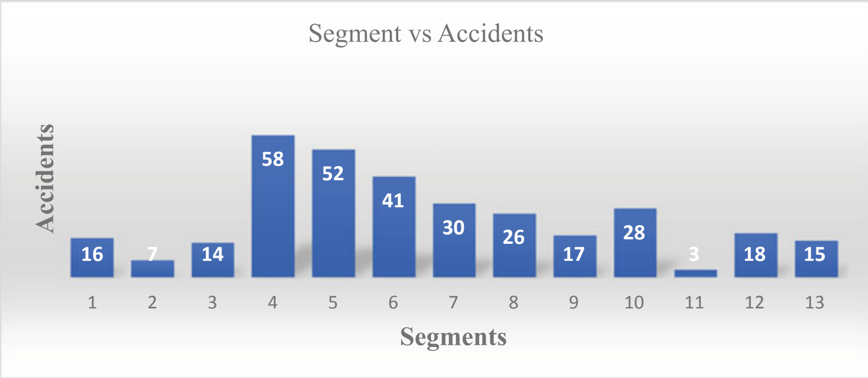

Fig. 3 Crash frequency versus road segments for 3 years

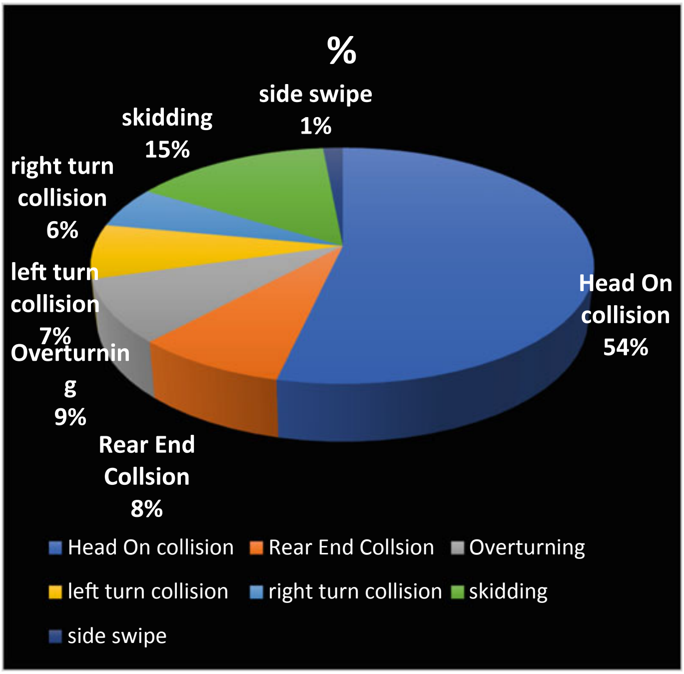

Fig. 4 Type of Crashes

4.3 *Traffic Data*

According to the traffic studies done while preparing Detailed Project Report in 2006–07, the count was done at three locations one within 15 km from start then

Fig. 5 Time of Crashes

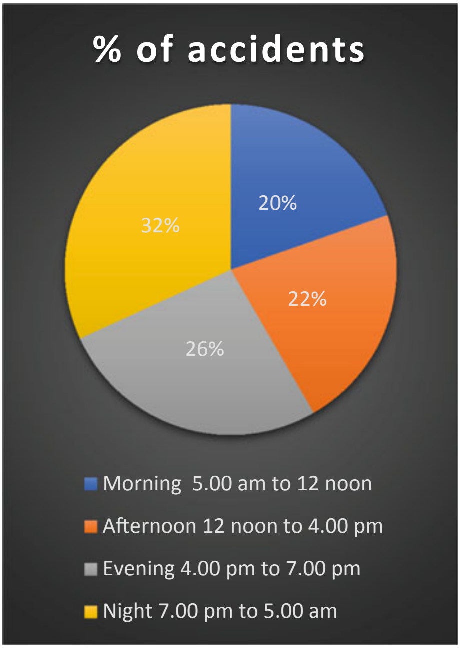

Table 2 Actual AADT in 2006–07 and projected AADT in 2017 in PCU

Locations	AADT in PCU in 2006–07	Projected AADT in PCU in 2017
25–65 km	21,956	37,553
15–25 km	25,293	43,254
0–15 km	16,794	28,704

between 15 and 25 km and finally between 25 and 65 km. The AADT is given in PCU including all fast as well as slow-moving vehicles as shown (Table 2).

4.4 Geometric Data

Geometric data such as lane width, shoulder width, median width, horizontal gradient, vertical gradient, radius of curve and length of the curve was obtained from AutoCAD files. AutoCAD files for each km for the entire stretch were studied and data was collected (Fig. 6).

This is the sample AutoCAD file for 2nd km stretch. It gives the following information regarding the alignments, profile, subassembly details, etc.

Lane width = 4.5 m; shoulder width = 2 m; median width = 4.5 m; and radius of curve = 1000 m.

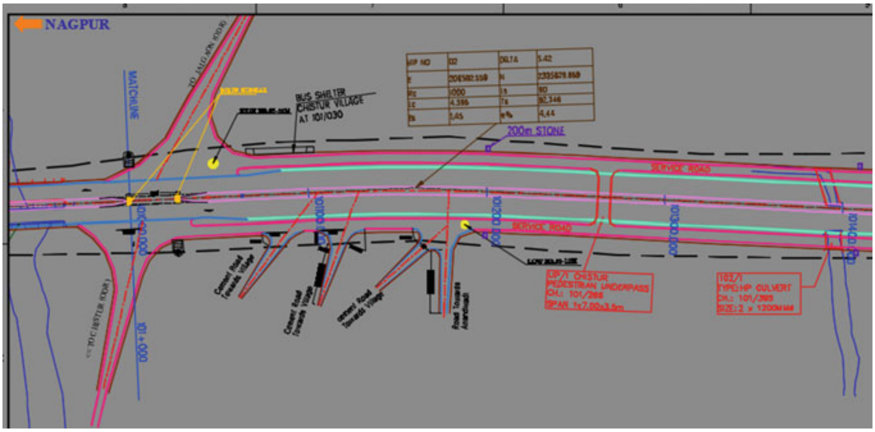

Fig. 6 Sample AutoCAD drawing of 2nd km of stretch

Length of curve = 20 m; centre line length = 3 m; access density; spacing between centre line = 6 m, etc. Similarly, data was collected for each kilometre stretch.

4.5 Road Safety Inspection Data

For road safety inspection, a complete drive through technique was used. Complete 65 km stretch was covered by a driving throughout 130 km (both sides). Information such as access density, dangerousness of accesses, lane width, shoulder width, shoulder condition, median width, chevron boards, road markings, delineations conditions, barriers, reflector condition, roadside hazards such as ditches, poles, trees, encroachment and dangerousness of terminals was extracted from the drive-through process (Figs. 7, 8, 9 and 10).

4.6 Video Graphic Data

The speed data for the project corridor was extracted from the videography data collected for the project corridor. The video data was collected at three locations one in first 3 segments, next one in further 2 segments and the last one in remaining 8 segments. The three locations were [20°55′19.42″ N and 77°47′7.43″ E], [21°1′24.28″ N and 77°49′27.20″ E] and [21° 1′34.84″ N and 77°55′25.90″ E]. The video was recorded such that a stretch of 54 m was captured, and then based on the frame rate of the video, travel time of vehicle was calculated and subsequently speed of vehicle was calculated. The speed here is space mean speed as distance remains

Fig. 7 12.5 km chainage (encroachment)

constant. As there are different vehicles with different PCU values, weighted space mean was calculated and thereby operating speed was also computed (Fig. 11).

Similarly, the operating speed of all the three locations was calculated as per the given Table 3.

5 Data Analysis

Data collected above was analysed as further to determine the following five factors which are tabulated as follows which were used in the computation of Risk Index which is the product of all these factors is figured further in the paper (Table 4).

5.1 Exposure Factor

Exposure factor indicates the probability of the traffic population to get exposed to the risk of crashes. Exposure factor depends on AADT of the segment and calculated as.

Fig. 8 20 km chainage (no median)

Fig. 9 34 km chainage (dangerous access)

Fig. 10 18 km chainage (median opening)

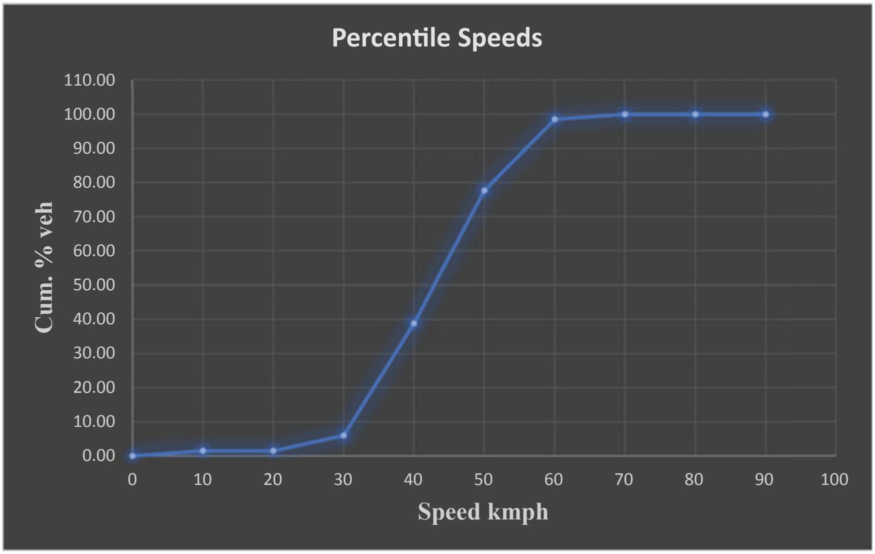

Fig. 11 Graph showing percentile speed of vehicles at video location 3

Table 3 85th percentile speed at all three locations

Location	85th % speed km/h		
	Towards Mumbai	Towards Nagpur	Average
Location 1	54	40	47
Location 2	77	73	75
Location 3	60	54	57

Table 4 Factors for risk index and their responsible elements

Factors for risk index	Responsible elements
Exposure factor	AADT, length of segments
Accident frequency factor I	Accesses density, shoulder width, median width, road markings, signs, delineations
Accident frequency factor II	Curve frequency, radius of curves, length of curves
Accident severity factor I	Terminals, poles, ditches, crash barriers and encroachments, etc.
Accident severity factor II	Design and operating speed, segment length
Risk Index	Exposure factor, accident frequency factor I, accident frequency factor II, accident severity factor I, accident severity factor II

$$\text{Exposure}_{\text{rural}} = \frac{\text{AADT(segment)}}{\text{AADT(max)}} * 5 \tag{1}$$

5.2 *Crash Frequency Factor I*

The Crash frequency factor I (AFFI) is depended on various safety-related parameters such as accesses, median width, lane width, shoulder width, delineations, road markings and signs. It is calculated as the product of accident frequency factors (AF) due to all the safety issues listed in Table 5. This factor for each safety factor j is calculated as

$$\text{AF} = 1 + \text{WTSC}_j * \Delta\text{ACF}_j * P_j \tag{2}$$

where WTSC is weighted score for each safety issue based on score given to the problem by the criteria given in table number 5, ΔACF is projected increase in accident risk as per AASHTO guidelines and P is share of accident typologies due to issue again as per AASHTO guidelines.

Table 5 Proforma for giving scores to the safety issues

Safety issue	Subissues	V. high-level problem (S_{ik} = 1.5)	High-level problem (S_{ik} = 1.0)	Low-level problem (S_{ik} = 0.5)	No problem (S_{ik} = 0)
Accesses	Dangerousness	N.A	Located on curves, crests, junctions, etc.	Unpaved accesses	No access
	Density	>20/km	15–20/km	10–15/km	<10/km
Cross sections	Lane width (L)	N.A	L < 2.75 m	2.75 < L<4.5 m	L > 4.5 m
	Shoulder (s) Paved (P) Unpaved (UP)	Absent	For open area Ws < 1 m (P) and < 1 m (UP) For B/U area Ws < 1 m (P)	For open area 1 m < Ws < 1.5 m (P) 1 m < Ws < 2 m (UP) For B/U area 1 m < Ws < 2 m (P)	For open area Ws < 1.5 m (P) Ws > 2 m (UP) For B/U area Ws > 2 m (P)
	Median (m)	Absent	For open area Wm < 3.5 m For B/U area Wm < 1.5 m	For open area 3.5 m < Wm < 5 m For B/U area 1.5 m < Wm < 2.5 m	For open area Wm > 5 m For B/U area Wm > 2.5 m
Delineations	Chevron boards	N.A	Missing on severe curves	Missing on severe curves	Present
	Barriers, reflectors	N.A	Missing or broken	Lower reflectivity	OK
Road markings	Edge/centre line	N.A	Missing	Faded	OK
	Chevron markings	N.A	Missing	Faded	OK
Roadside	Embankments, bridge	N.A	Unshielded on high slopes	Unshielded on bed slope	OK
	Trees, utility poles, ditches	N.A	Less than 3 m from C/w	Between 3 and 5 m from C/w	More than 5 m from C/w
	Encroachments	Less than 2 m from C/w	Between 2 and 3 m from C/W	Between 3 and 5 m from C/w	More than 5 m from C/w

(continued)

Table 5 (continued)

Safety issue	Subissues	V. high-level problem ($S_{ik} = 1.5$)	High-level problem ($S_{ik} = 1.0$)	Low-level problem ($S_{ik} = 0.5$)	No problem ($S_{ik} = 0$)
Signs	Warning/regulatory signs	N.A	Missing/broken	Faded/no reflectivity/improper placing	OK

5.3 Accident Frequency Factor II

The accident frequency factor II (AFFII) depends on another geometric element, i.e. curves. Length, radius and frequency of curves are required for calculation of weighted score of a particular element. Further, this weighted score, i.e. WTSC, relative upsurge in accident risk due to the geometric element, i.e. $\Delta \text{ACF}_{\text{GD}}$, and share of accident typologies due to this issue, i.e. $P_{\text{GD},}$ are used to determine AFF II as shown

$$\text{AFF II} = 1 + \text{WTSC}_{\text{GD}} * \Delta\text{ACF}_{\text{GD}} * P_{\text{GD}} \tag{3}$$

5.4 Accident Severity Factor I

The accident severity factor I (ASF I) depends on roadside element such as embankments, ditches, poles, trees and encroachments. Weightage of each element and score given to each element (based on AASHTO guidelines) are required for calculation of weighted score of the roadside element. Further, this weighted score, i.e. WTSC, relative upsurge in accident risk due to the roadside element, i.e. $\Delta \text{ACF}_{\text{RS}}$, and share of accident typologies due to this issue, i.e. $P_{\text{RS},}$ are used to determine ASF I as shown

$$\text{ASF I} = 1 + \text{WTSC}_{\text{RS}} * \Delta\text{ACF}_{\text{RS}} * P_{\text{RS}} \tag{4}$$

5.5 Accident Severity Factor II

The accident severity factor II depends on design speed of the facility and operating speed at that highway facility and is given as

$$\text{Accident Severity Factor II} = \frac{\text{Operating Speed}}{\text{Design Speed}} \tag{5}$$

6 Results

The above analysis procedure was followed for the selected stretch of 65 km. This analysis was done using MATLAB 2014 software by giving inputs regarding all

Table 6 Result table obtained after analysis in MATLAB

Seg. No.	EF	AFF I	AFF II	ASF I	ASF II	Risk Index	Acc. freq.
1	3.32	3.25	1.76	1.12	0.47	9.99	16
2	3.32	2.64	1.77	1.04	0.47	7.58	9
3	3.32	2.66	1.78	1.14	0.47	8.42	14
4	**5.00**	**5.29**	1.75	1.25	**0.75**	**43.39**	**61**
5	5.00	4.2	2.08	1.05	0.75	34.40	54
6	4.35	3.79	1.76	1.16	0.57	19.19	42
7	4.35	3.31	1.94	1.16	0.57	18.47	31
8	4.35	4.25	**2.14**	**1.26**	0.57	28.42	28
9	4.35	3.45	1.96	1	0.57	16.77	17
10	4.35	3.37	1.63	1.2	0.57	16.35	28
11	4.35	2.85	1.63	1.02	0.57	11.75	3
12	4.35	2.72	2.06	1.04	0.57	14.45	18
13	4.35	3.08	2.12	1.02	0.57	16.52	15

Bold values represent highest values amongst the respective columns

Table 7 Result table obtained after analysis in MATLAB

	EF	AFF I	AFF II	ASF I	ASF II	RI	Count
EF	1.00						
AFF I	0.68	1.00					
AFF II	0.31	0.13	1.00				
ASF I	0.14	0.61	−0.17	1.00			
ASF II	0.92	0.79	0.24	0.18	1.00		
RI	0.81	0.95	0.28	0.48	0.91	1.00	
Count	0.69	0.88	0.12	0.54	0.82	0.89	1

the required information about all the 13 segments as above, and the result was as Tables 6 and 7.

7 Important Findings

- From the table number 9, it is observed that the Risk Index is highest for the segment number 4 which indicates that this segment is most dangerous segment among all and it can be validated from the fact that the accident frequency is highest on that segment.
- From the correlation matrix, it is clear that there is a high correlation between the Risk Index and the accident frequency.

8 Conclusion

The segmentwise computed Risk Index in this study is determined by various factors which are based on various other elements such as traffic volume, condition of road furniture such as shoulder, median, road signs, road markings, barrier, chevrons, other elements such embankments, terminals, ditches, transitions, curves and encroachments. Risk Index for all the segments on the National Highway, i.e. NH 53 was calculated. Higher the value of Risk Index greater is the risk involved in that segment.

The Road Risk Index can be evaluated regardless of the availability of accident data. If along with other data, relevant accident data is available then it can be effectively used in combination with Risk Index as criteria for ranking of segments which is done in this study. On the other hand, if the accident data is not dependable, then the Risk Index remains only criteria for ranking.

The Road Risk Index expresses the overall safety condition of the segment on a highway facility unlike Black Spot Management Program. Thus, with the help of these index values, prioritisation of segments for safety treatment can be done along with identification of the type of safety treatment required.

9 Future Scope

- In case of safety assessment by this method of determining Risk Index, human factor is never taken into consideration, so in future efforts can be in this direction.
- Various safety assessment techniques can be associated with the safety assessment by the method of Road Risk Index.
- The determined Risk Index can be compared with the accident prediction model developed based on various proactive safety variables.

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A Study on Understanding the Factors of Non-compliance in Motorized Two-Wheeler Helmet Use in India: A Review of Literature

Rusha Das

Abstract Injuries related to road traffic are the major causes of death among people in the age bracket of 15–29 years (World Health Organization 2015). In many developing countries, motorized two-wheelers are the dominant means of transport and contribute to a more significant proportion of road fatalities. Leading causes of death are wounds to the head and neck that account for 88% fatalities in developing countries. India comprising of a significantly considerable percentage (70%) of two-wheelers with weak enforcement of helmet laws and poor compliance poses further challenges. This paper is divided into three sections. Section one reviews the literature on compliance behaviour from different theoretical disciplines to understand the behavioural factors that motivate compliance behaviour with the law. The second section reviews the empirical literature on helmet compliance across various countries with a mandatory helmet law. The overview brings to light that mere presence of a helmet law does not motivate compliance behaviour and more often lead to "token compliance". The meaning of helmet use should be properly understood through in-depth interviews to understand the motivations to comply. The third section focuses on the evolution of helmet law in India. The overview suggests that regulations focus heavily on penalty structure and withdrawal of license as the primary disincentive to motivate compliance. Finally, alternative approaches to increase helmet compliance are discussed, and a regulatory framework that recognizes the coexistence of law, morality and social norm is suggested.

Keywords Social norms · Instrumental and normative compliance · Helmet regulation in India

1 Introduction

Injuries related to road traffic are the major causes of death among people in the age bracket of 15–29 years (World Health Organization 2015). Risk of road crashes is

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significantly higher for young drivers than older ones according to research conducted in different countries (Massie et al. 1995; Panayiotou et al. 2008). Most of the government interventions designed to improve global road safety fall within five domains namely management of road safety, safer roads, mobility, vehicles, road users and post-crash response, respectively. The framework requires a significant shift of the responsibility from road users to those who design the road transportation system. However, according to the report, adequate laws that address the five underlying risk factors (speed driving, drunk driving, non-use of helmets, non-use of seat belts and child restraints) are present in only 28 countries that are seven per cent of the world population represented by four hundred and forty-nine million people. Besides, a fifteen per cent increase in the number of registered vehicles annually poses further challenges in ensuring safer mobility globally.

In developing countries, two-wheelers are the dominant means of transport and contribute significantly to the proportion of road fatalities. In India, two-wheelers constitute 70% of the vehicle population and account for 28.8% of total road injuries (MoRT 2015). According to World Health Organization's manual on Helmet (2006), injuries to the head and neck are the major causes of death and account for 88% fatalities in developing countries. The manual also highlights shared traffic space and lack of physical protection as factors that increase the risk of such injuries. Besides, head injuries create high social costs for the survivors, survivors' family and to the larger community. The medical costs are significantly higher for head injuries (Blincoe et al. 2000) and increase a nation's health care costs.

In India, the total number of fatal road crashes reported by the ministry of road transport and highways for the year 2016 was 136,071. Motorized two-wheelers accounted for 30.6% of the total fatal accidents. Further, productive age group between 18 and 35 years accounted for the highest share of 46.3% of the total number of people killed (150,785). The report also highlights based on a subjective measure that the fault of the driver accounted for 84% of the fatalities. Of the major responsibilities of the driver exceeding the speed limit was one of the primary causes (66.5%) of the total number of road crashes (480,652). The holder of a driver's licence accounted for 84.6% of the total road crashes signalling that mere presence of a law does not motivate people to comply. Of the total number of 150,785 deaths in the country, 6.7% were accounted for not wearing helmets by two-wheeler riders. However, the report should be clearer on how have they arrived at such calculations. Additional the annual cost of road crashes in India account for 3% of the country's GDP (Goyal et al. 2017).

Therefore, what motivates people to conform to rules and other than legal enforcement is critical to designing effective policies. In this attempt, the next section aims to understand the factors that motivate compliance/non-compliance behaviour from a cross-disciplinary perspective.

2 Cross-Disciplinary Research on Compliance Behaviour

Compliance is simply not the correspondence of behaviour with legal rules. It is a stand-alone concept that derives its meaning from different perspectives of why people obey or disobey the law. Different theories of law lead to significantly different conceptions of compliance. In the law literature, motivations to comply or not comply can be categorized into two factors, namely instrumental and normative motivations. Instrumental perspectives on compliance follow the same assumptions as that of the social control theory. The social control theory assumes that external rewards and punishments motivate behaviour. Thus, people shape their behaviour in response to tangible incentives and penalties associated with following the law. The social control theory also receives support from public choice theory where economic models of crime and punishment are applied to legal studies (Becker 1968; Stigler 1970). When public policymakers strive to obtain compliance, they adopt an instrumental perspective (Tyler 1990). However, the instrumental perspectives have failed to explain the concept of free-rider problem in cases like tax evasion where the actual percentage of tax evasion exceeded the predicted value as proposed by the expected utility model (Tyler 2006).

Another determinant of compliance is the normative motivations of what individuals consider as just and moral contrary in their self-interest. If people's attitude towards compliance with the law is based on personal or legitimate reasons of how they ought to behave or should behave, they are said to be normatively motivated to comply or not comply. Normative motivations based on morality imply that one feels the law to be just and hence one ought to comply, whereas normative motivations based on legitimacy implies that one feels that the enforcement authority has the right to enforce a particular behaviour (Tyler 1990, Easton 1958, Friedman 1975).

Moral normative motivations to comply or not to comply therefore have different outcomes to compliance with the law. For example, one may feel that it is morally right to engage in polygamy but may refrain from stealing. However, normative motivations driven by legitimacy will make people less likely to evade rules since they are motivated to comply because they feel that the government or the state has the right to enforce laws, giving discretionary authority with the government or the state regardless of the severity of punishment (Tyler 1990). On the other hand, people who behave based on their self-interest would be motivated to comply based on their estimate of the likelihood of detection of non-compliance. For example, people may not follow traffic rules in places where there is no traffic police but may refrain from shoplifting in a mall where there are higher chances of getting caught.

Empirical studies on legitimacy highlight attachment to the political system or support as a measure of perceived obligation to obey (Easton and Denis 1969). If a relationship between compliance and support is found, then it is assumed that people comply due to perceived obligations to law. Support, therefore, is an indirect measure of normative legitimacy as a motivation factor for compliance. Easton, 1969, distinguishes between the legitimacy of a particular authority and legitimacy of the institutions or procedures of the government. While legitimacy to the authority is based on short-term calculations of self-interest, legitimacy to the institutions and procedures are viewed as normative judgments that get developed during the political socialization process (Sears 1983). If support to legitimacy is driven by normative values, then it will have consistency and will not change as the immediate environment changes (Tyler 1990). However, both legitimacies will eventually need to meet some interests of the population in the long term to maintain the respective legitimacies (Kelman 1969).

Several empirical studies have found higher levels of support for the authority is strongly associated with behaviour for the system (Gibson 1967; Jaros and Roper 1980). Further Useem and (1979) have found that non-supportive attitudes are translated to behaviour when there are political groups that people could join. However, Wright (1976) found that low trust of government does not translate to civil disobedience or disruption of government. Some studies also highlight personal assessment of the morality of law as an important factor that affects obedience to the law (Grasmick and Green 1980; Jacob 1980; Tittle 1980). Thus voluntary compliance with the law is much more cost effective than compliance through force since the latter involves additional resource cost to ensure compliance.

Another determinant for compliance or non-compliance could be attributed to social norms studied in social psychology literature (Cialdini and Trost 1988). Social norms are accepted standards of behaviour within a particular group or society. Social norms tend to solve collective action problem by encouraging or discouraging certain behaviour that people would not engage in if the norm was absent. Many research studies have indicated social norms to be strong predictors of behaviour. (Ajzen 1985; Blanton et al. 2008). Furthermore, in social psychology, social norms are divided into descriptive and injunctive norms. Descriptive norms are patterns of behaviour that people engage in because they believe that individuals in their reference network follow it. Reference network is the set of individuals that matter to one's choice (Bicchieri 2014). On the other hand, injunctive norms are behavioural patterns that people engage in because they believe that other's think that other individuals in their reference network believe they ought to follow it (Cialdini and Trost 1988). Further, people's conception of 'self' is internally motivated and this gives rise to a set of personal norms (Cialdini and Trost 1988). According to Cialdini, the mere presence of a descriptive, injunctive or personal norm does not activate behaviour until the norms are salient (Cialdini et al. 1991).

The difference between attitude and behaviour has been highlighted by Ajzen in his latest work on the "Theory of Planned Behaviour". The theory postulated three belief systems that shape human behaviour. The behavioural beliefs stress on the probable consequences of a particular behaviour, the normative beliefs shape the expectations about social norms about and the control beliefs shape the facilitating and impeding factors to behaviour. Behavioural beliefs produce an attitude that is either favourable or unfavourable towards the behaviour; normative beliefs produce the perceived social pressure, and control beliefs produce the perceived behavioural control. Thus, attitude towards the behaviour, the subjective norm and perceived behavioural control are antecedents of behavioural intention that shape the behaviour in question (Ajzen 2014). In his latest work, he incorporates both descriptive and injunctive aspect in the subjective norm taking insights from Cialdini's work.

Models based on the economic theory focus on deterrence as a factor for compliance. Deterrence theory based on rational choice model assumes that individuals comply with the regulation if the likelihood of detection is higher than the benefits of violation. According to Becker (1968), violations are more likely when both the fines and the rate or likelihood of detection is lower. Becker's argument hints that penalties, sanctions and the likelihood of detection are under the society's control. However, he also mentions that after a point enforcement may become uneconomic and result in a social loss. The loss can be calculated as an aggregate of damages, cost of conviction, apprehension and carrying out the punishment, respectively. These social loss variables could be minimized with respect to the probability that an offence is discovered, and the violator is apprehended and convicted, the size of the punishment and the form of punishment.

Further to the deterrence model Allingham and Sandmo highlight that deterrence might have less effect if the probability of detection is higher. In other words, increasing the value of the penalty will not have a greater deterrent effect if the probability of getting caught is high (Alligham and Sandmo 1972). Felkinger and Walther extend the deterrence model by suggesting that a system of rewards and penalties could increase compliance than a system based on only penalties. However, their work focuses on tax compliance in the form of tax refunds or reduction rate in total payable tax (Felkinger and Walther 1991).

Monetary incentives in the form of rewards or penalties may have the price effect that reinforces the behaviour that is incentivized and/or the psychological effect. Sometime the psychological effect may work in the reverse direction and crowd out the incentivize behaviour (Gneezy et al. 2011). In the model proposed by Benabou and Tirole (2006), individuals have three variables that affect their total utility of exhibiting a particular behaviour. These are extrinsic rewards, enjoyment of doing a particular activity and their image about themselves and others. The image motivation is the value that an individual or others attribute to the individuals intrinsic and extrinsic motivation based on the effort level and incentives. The image motivation component is similar to the descriptive and injunctive norm component in social psychology literature. Gneezy Meire and Rey-Biel highlight that incentives could reduce image motivations to comply by diluting the signal to the self and others on whether a task is undertaken to do good or to receive compensation. However, not all incentives backfire even if the compliance rate does not significantly increase, for example, pricing garbage bag to reduce waste and encourage recycling Kinnaman (2006). Further compliance might also increase depending on whether an individual frames a situation as social or monetary (Heyman and Ariely 2004), since people may believe that monetary incentives or disincentives are included because the social norm is to violate.

In the public health domain, Lipinski and Rimal (2005) talk about the perceived norm, and this is similar to the subjective norm proposed by Azjen. The perceived norm has two components the subjective beliefs about the presence of a norm; this is similar to the descriptive norm in social psychology. The second component is the subjective belief about the pressure to comply; this is similar to the injunctive norm component in social psychology. Further modifications have been proposed by Storey and Douglas and Schoemaker in 2006 where they use collective behavioural norm and collective attitudinal norm as proxies for descriptive and injunctive norms. However, such aggregation of individual behaviour and attitude does not suggest what individuals believe about others.

Finally, Bicchieri (2006) combines the ideas from economics, social psychology and her contributions to game theory to come up with a unified definition of social norm that motivates people to comply or not comply. A social norm is said to exist and is followed by individuals if a sufficiently large part of a relevant group that matter to the individuals follow it. In other words, individuals have an empirical expectation that a sufficiently large part of their reference network conforms to a norm. The second criteria conditional on which one choses to conform to a social norm is the normative expectation that people have of what other individuals think they should do. Normative expectations are accompanied by sanctions where people in one's reference network are often prepared to sanction transgressing the normative behaviour. Bicchieri's empirical expectation and Cialdini's descriptive norm differ regarding whether expectations are unilateral or multilateral. In case of Cialdini's descriptive, norm of how others behave is informative and is similar to the unilateral expectation proposed by Bichhieri. On the other hand, Bichhieri proposes both unilateral and multilateral empirical expectation that matter to choice. For example, buying products from eBay is informative and form a unilateral empirical expectation that others buy from eBay. On the other hand, coordinating with others while driving to the left is a multilateral empirical expectation where non-coordination may invoke sanctions.

Bichhieri's definition of normative expectation contrasts with the injunctive norm defined by Cialdini. Normative expectation incorporates the possibility that a social norm can exist if individuals belief about the legitimacy of their reference group expectation of conformity which is not explored in economics and social psychology literature. However, such a conception bears striking similarity with the normative motivations to comply as explained in law literature. Further, Bicchieri also makes another classification of personal normative beliefs of what people think they should do. For example, if one believes that smoking is bad for one's health, then these are personal normative beliefs. The personal normative beliefs may/may not be connected to normative expectations.

Therefore, a careful synthesis of the above cross-disciplinary research on compliance reveals that people have varying motivations to comply. Instrumental motivations as proposed by law literature or deterrence theory in economics point out that people choose to comply or not based on the likelihood of detection. However, voluntary compliance is more cost-effective than enforcement because instrumental motivations can significantly increase the overall social cost. Thus, factors such as image motivations as proposed by economics literature, normative motivations as proposed by law literature and social norms proposed by social psychology, public health literature offer interesting insights into other factors affecting compliance.

Drawing from the theoretical review on factors affecting compliance and non-compliance, we now aim to critically analyse the studies conducted to understand the determinants of compliance and non-compliance with helmet laws across different third and second world countries with mandatory helmet law and poor compliance. The reason we chose third and second world countries is because two-wheelers are the dominant mode of transport and account for a significant portion of road fatalities. Also, since one of our objectives is to examine alternative approaches to increase compliance and analyse its applicability to increase helmet compliance in India, these countries fit into the sample characteristics we are looking for.

3 Review of Empirical Research on Factors Motivating Compliance/Non-compliance with Helmet Laws

Empirical studies of different second and third world countries are chosen to critically examine the methods used to identify factors affecting compliance and non-compliance with helmet laws.

In Iran, helmet law has been mandatory since 2002. Ghasemzadeh and Babazadeh (2017) use a cross-sectional study to investigate the cognitive and behavioural factors of helmet use among 150 motorcyclists in Iran. The sample was conveniently chosen at repair shops in a rural community in Iran. A written questionnaire on demographic and motorbike information was followed by a structured interview for collecting data on the theory of planned behaviour (TPB) constructs. Further self-reported interview questions were used to collect data on helmet use. Subjective norms and perceived behavioural controls were statistically significant predictors ($p < 0.0001$) of helmet use. There was no statistically significant relationship between positive attitudes and wearing a helmet. However, it is not quite clear whether both descriptive and injunctive norm questions have been used to study the influence of social norms.

In another study, Mehri et al. (2008) show that all the TPB constructs were statistically significant in predicting helmet use in Iran. Haqverdi et al. (2015) use structured interviews to understand the socio-economic and psychological factors affecting helmet use among 222 male motorcyclists in Mashhad city in Iran. Five robust factors using 24 item questions on attitudes, enforcement and social norms were created using PCA and CFA. Of the five factors, the perception of social norms and tendency to use helmets and perception of helmet use enforcement were having a p-value of <0.0001. However, this does indicate what kind of social norm (injunctive, descriptive or legitimacy of others) influence behaviour. Further, the perception of enforcement is heavily weighted as a factor by respondents who wear helmets without fastened chin strap rather than wearing them correctly. Therefore, enforcement may only lead to token compliance.

In Vietnam, helmet use had been mandatory since 2001. Hung et al. (2008) use structured interviews and observation technique to understand the barriers and determinants of helmet use in Hai Duong Province in Vietnam. Twenty-three selected petrol pump stations were randomly selected with 808 individuals as total sample. Twenty-five item attitude belief scales about helmet use and non-use were used that resulted in seven factors namely attitude towards features of the helmet, negative perception of helmet use, attitude towards universal helmet legislation, price, and storage problem, penalty, enforcement and risk, lack of helmet effectiveness. Further, results from multinomial logit highlight that trip distance greater than ten kms and attitude towards physical features of helmet use were statistically significant. However, the items representing the factors "helmet effectiveness" and "negative perception of helmet use" do not offer any contrasting explanation. Also, other factors, like peer influence, influence from family and friends, moral norms, etc. which may provide useful information regarding compliance, seem to be missing from the attitude belief scale. Further, the overall compliance rate was only 23% despite 95% people disagreeing with the statement that helmet use does not reduce the risk of head injury. Kumphong et al. (2018) use structured equation modelling (SEM) to investigate psychological factors affecting helmet use among 210 college students in Ho Chi Minh City. They show that out of several psychological models like the Health Belief Model, The Theory of Planned Behaviour and Traffic Locus of Control, the TBP is better at explaining the overall intention to use the helmet. However as pointed earlier all the dimensions of a social norm (injunctive, descriptive and legitimacy of others expecting compliance) have not been explored.

In Thailand, helmet law has been made mandatory since 1994. Ichikawa et al. (2003) analysed secondary data of 12,002 patients from the regional hospital for a period between January 1994–December 1997, to understand the effect of helmet act for a motorcyclist. The odd ratios of fatality for helmet users and non-users for single and multiple crashes were calculated. The result indicated that there was no significant difference between the proportion of fatalities pre- and post-enforcement of helmet use. Among injured motorcyclist, the helmet use rate increased fivefold and was also dependent on the time of the day. Effect of helmet use was non-significant for multiple crashes. Overall helmet act and helmet use revealed to be ineffective signalling that motorcyclist may use helmets but improperly or the meaning of helmet use still requires further research.

Jiwattanakulpaisarn et al. (2013) used face-to-face interview survey in 10 different locations in Thailand to understand whether law enforcement awareness affects helmet use. A total of 2429 drivers were randomly interviewed, and data was analysed using multivariate ordered logit regression. Their finding suggests that helmet use was higher for those who had the uncertain perception about time and location of motorcycle checkpoints and perceived a high risk of getting caught. However, the compliance was lower for drivers who could identify the exact time and location of checkpoints. Therefore, the paper suggests that increasing the perceived risk of non-use of helmets through more police presence is necessary. However, as seen in the compliance literature, strong enforcement requires additional resource cost and may result in a social loss. Thus, other voluntary factors need to be investigated. Further, self-reported helmet use is prone to reporting bias.

In Indonesia, helmet use has been mandatory since 1986. Conrad et al. (1996) use observational and open-ended interview questions to examine motorcycle helmet use and injuries in Yogyakarta, Indonesia. The data was supplemented by one-month injury data of patients from emergency departments in Yogyakarta. It was found that of the 9242 drivers who wore helmets only 55% wore them correctly and compliance was low during the night and in the absence of police surveillance. One of the major findings was that most people who wear helmets believe others would wear helmets if police enforcement is strong. Thus, the meaning of helmet use is influenced by external controls leading to token compliance. Such token compliance may result due to instrumental motivations to comply with the law. However, further research should carry out to validate such a claim. Susilo et al. (2015) used 3000 motorcyclists from three Indonesian cities to investigate the behavioural factors for disregarding traffic laws. The structured questionnaire on the type of reported violations was framed keeping in mind the violations that are outlawed or banned by the national traffic regulation. Violations were categorized into four groups. Group a—consisted of violations that had both jail and fine consequences, group b—had only fine consequences, group c—had factors that were strictly enforced by the community as an accepted norm and group d—had crashes that result in a lengthy jail term. It should be noted here that among the items that are considered in group c, helmet use is not an enforced social norm. The use of helmets is included in group b category that has only fine consequences. Moreover, most violations were self-reported scores that are prone to reporting bias. Consistent with previous results using the TBP model and SEM they found that attitudes, beliefs, norms and perceived behavioural controls can significantly predict traffic violation behaviour. However, the study only focused on positive norms of obeying traffic laws rather than exploring the possibility of disobeying traffic law as a social norm itself. Further what kind of social norm shapes behaviour still needs further investigation?

In Greece, helmet law has been mandatory since 1977. Germeni et al. (2016) use self-administered questionnaire among 523 randomly selected public school students from middle-income areas of Athens in Greece. Further, the Health Belief Model was applied to 12 focus groups of 70 students. The focus group was divided according to two criteria: 1. those who rode a TWMV always or most of the time and 2. those who wore the helmet always/mostly or never. Based on the criteria, three focus groups for every four categories were created—male users, female users and non-users. It was found that self-reported frequent use of helmets was characterized by the high threat of injury and the influence of “significant others”. Among students who reported non-use of helmets, there was low threat perception and risk perception. Also, the benefit of compliance sited by users was protection from road crashers whereas non-users sited avoiding tickets from traffic police as a benefit. Despite positive perception about the protective efficacy of helmet use (91%), 41.4% of TWMV reported helmet non-use. This suggests that the motivation to comply for non-users may be instrumental. For users, the motivation maybe social where one’s perception of wearing a helmet is either influenced by others or one’s own belief based on experience. Thus, without further research on the type of motivation designing interventions to increase compliance would be difficult for users and non-users.

A similar study conducted by Germeni et al. (2009) show that legal code is unlikely to change motorcyclist behaviour. In that attempt, Papadakkaki et al. (2013), uses self-administered questionnaire among 405 riders to understand the barriers and facilitators to helmet use. Two scales were developed to asses factors facilitating and impeding the use of helmets. Using PCA and multiple regression analysis, it was found that factors like imitation, experience, self-protection, regulation and environment were statistically significant predictors of facilitators. Discomfort and underestimation of danger were significant predictors of barriers to helmet use. Overall the compliance rate was very low around 25%. The study also uses self-reported data prone to bias. Further, items included in factors like environment and underestimation of danger and risky behaviour seem to be ambiguous. For example, "I wear a helmet when I am riding in an unknown area" can also mean high-risk perception. Similarly, "I am a risk-taking person" can also mean underestimation of danger. A better classification of factors would provide more useful insights.

Several other studies on helmet use have used similar methodologies and theoretical frameworks like the TBP, Health Belief Model, Motorcycle Rider Behaviour Questionnaire to understand the factors for compliance/non-compliance (Ahmed et al. 2013; Xuequn et al. 2010; Özkan et al. 2012; Elliott et al. 2007, Jingkang et al. 2018). It is evident that most studies have used self-reported measures and the many have not investigated all forms of motivations to comply. With the above critical review, now let us examine the helmet regulation in India and the empirical studies on helmet use conducted in India.

4 Helmet Regulation in India

Laws governing the use of motor vehicles in India are mandated by the Motor Vehicles Act of 1988, a modification of the act of 1939. The initial modifications to the act of 1939 are suggestive of the factors that gained primary importance. Factors that were included in the list were increase in the number of registered commercial and personal vehicles, adoption of the latest technology in the automotive sector, increased flow of passengers and freight with low obstacles to curb imbalances at regional or local levels, establishment of road safety standards, pollution control measures and policy aimed at regulating the flow of hazardous and explosive materials, private sector involvement in the road transport field and effective ways of tracking violators. Thus, the stress was more on economic growth rather than on safety.

Currently, the Motor Vehicles (Amendment) Bill 2019 has been passed. The new bill mandates a fine of Rs. 10,000 instead of Rs. 2000 for drunken driving, rash driving from Rs. 1000 to Rs. 5000. However, not wearing seat belts and helmets attract a meagre fine amount of Rs. 1000 with a three-month withdrawal of license as mention in section 194C of the Motor Vehicles Amendment Act 2019. However, the act does not specify any mandatory requirements on the educational qualification of the drivers to obtain licenses. Moreover, the variation of the law exists across different states, for example in Delhi women, and the Sikh community is exempted from the mandatory helmet requirement. The new bill finally recognizes pedestrian by giving them rights and protection and the bill also proposes separate bicyclers, skaters, pushcarts and other non-motorized vehicles, adult accountability for child restraint.

The evolution of rules and regulation regarding traffic suggests that the traffic culture was more oriented towards economic growth rather than assuring personal security. The recent amendments of 2017 cover rights and protection for pedestrians. Predominantly penalty has been the only economic disincentive to encourage compliance. As seen in the previous sections not all behaviours are driven by instrumental motivation where the people based on their self-interest would be motivated to comply based on their estimate of the likelihood of detection of non-compliance. The above overview of the traffic laws suggests a traffic culture India is economically utilitarian (Atchley et al. 2014). This is also reflected in the ways people approach traffic laws in India.

The empirical studies on helmet compliance conducted in India suggest that weak enforcement of law and location of the trip are the main reasons for non-compliance (Wadhwaniya et al. 2012; Dandona et al. 2006). Places like Calicut where helmet law was mandatory people sighted strong opinion of not wearing a helmet if someone drives slowly and carefully. The study further suggested that people wore a helmet only when it was enforced by the law rather than using the helmet as a protective gear (Karuppanagounder and Vijayan 2016). Grimm and Treibich (2016), surveying 1502 households in Delhi found that risk-averse educated drivers were more likely to wear a helmet. Further drivers who showed higher awareness were likely to wear a helmet and to spend less. Drivers who indicated a high level of unawareness took the highest risks. When controlled for risk awareness, drivers compensated for speed and helmet use. Mirkazemi and Kar (2014) surveyed 9014 individuals in Pune city from March 2008 to February 2009 and found that people who wore helmet were mostly influenced by the neighbourhood environment, social norms, family and peer influence less by education and economic status.

5 Approaches to Increase Compliance

After analysis of the theoretical and empirical literature on the motivations to comply/not-comply, we can see that a carefully designed intervention should be able to differentiate between the different motivations to comply. For example, in a study conducted by Parker and Makhubel (2010) to understand the existing norm regarding spouse violence, it was noticed that only a fraction of female and male respondents in South Africa think that it is acceptable that a husband may hit his wife in the event of a disagreement, whereas most people thought that it is bad for children to see such violence and many believed that it is morally wrong. However, half of the respondents told that men in their community hit their girlfriends. Thus, the problem here is more social than moral. An intervention designed that upholds the moral norm of spouse violence would facilitate the creation of new social norm.

In Bogota, Columbia traffic violation intervention designed under the administration of Antanas Mockus is an excellent example of differentiating motivations to comply. Antanas distinguished between three types of regulation based on the positive and the negative motivations to comply (Mockus 2002). The three forms of regulation were legal, moral and social norms, respectively. The positive reasons for obeying with legal norms were respect for the law or the perception of legitimate authority. The positive reason for obeying moral norms was good conscience and for social norms was social approval. The negative reason for obeying legal norms was authority's penalty, for moral norms, it was bad conscience and for social norms, it was social disapproval. Further, the emotion of a violator was categorized into fear, guilt and shame if the violation was legal, moral or social, respectively.

In Bogota, people disobeyed traffic laws, and there was a social norm of legal non-compliance, those who did obey the law were looked down upon. In such a scenario, increasing penalties would have made no big difference. Instead, the government used the moral norm technique to make it vivid that traffic regulation would decrease injury and death, corps of mimes sent social disapproval signals to violators. Thousands of thumbs up cards signalling social acceptance and thumbs down card signalling social non-acceptance were distributed to drivers who used them to signal their acceptance/non-acceptance of other drivers' actions. This resulted in a change of the normative expectation about one's behaviour, and as a result, the city's injury and death rate went down. This was then publicized in the city changing empirical expectation about obeying traffic laws leading to further decline (UNICEF 2015).

Carter et al. (2014) have highlighted that misconception about existing social expectations often lead to adolescent risk perception. In the same line, Bichhieri (2006) talks about pluralistic ignorance where a person's normative beliefs do not match with the normative expectation. As a result, bad norms thrive and lead to pluralistic ignorance. Many people privately condemn a behaviour but wrongly believe that their peers endorse it. For example, helmet usage rate among teenagers as sited in many empirical studies in the above section talk about peer influence. Thus, one way to increase compliance is to correct for the wrong normative expectations that an individual has about the "significant others".

Many studies have highlighted that pluralistic ignorance often lets bad norms thrive in case of paternity leave (Miyajima and Yamaguchi 2017), alcohol consumption (Prentice and Miller 1993; 1996; Schroeder and Prentice 1998) necessary help during emergency situations (Latane and Darley 1970), etc. Various intervention techniques like changing the descriptive norm or the empirical expectation have been successful in correcting the misperception regarding drinking norms, cigarette smoking (Lewis and Neighbors 2004; Hancock et al. 2002) and environmental conservation in hotels (Goldstein et al. 2008).

It has been found that the extent of motivation to comply depend upon other factors such as perceived level of similarity between an individual and his/her significant others, compliance level of an individual's reference networks and norms of an individual's local setting (Cialdini and Goldstein 2004; White et al. 2002; Goldstein et al. 2008). Thus, interventions that target such factors can significantly increase compliance.

6 Discussion

We can draw the following implications on India's motor vehicle rules based on our review of the literature on compliance behaviour, helmet compliance, helmet regulation in India and approaches to increase compliance.

From the perspective of the mandatory helmet law, one can argue that a simple penalty mechanism is unlikely to generate enough compliance for improving the rate of helmet usage in the country. This is quite evident in the recent amendment of 2017 to the Motor Vehicle Act, which still focuses on penalty structure and withdrawal of licence as the primary motivator to comply with rules. The theoretical and empirical review suggests that penalty is only one of the many ways to make people comply and often leads to a higher social cost of enforcement. Voluntary compliance through other regulatory mechanisms like moral and social norms may be effective in increasing overall compliance. Cities in India could be grouped into high and low compliance cities based on traffic violations database collected from regional RTOs and then based on survey data that incorporates all the motivations to comply, appropriate intervention based on the three regulatory frameworks—legal, moral and social could be used as "treatments" to check the overall change in compliance. Further to the intervention could be a design of a regulatory system that recognizes that motivation of compliance based on the written law on how it is applied can be very different from the motivation of compliance based on one's self-conscience and the motivation based on social approval.

The empirical review on helmet compliance further suggests that most studies have used self-reported data on helmet usage and compliance rate that is prone to social desirability bias. Further, not all conceptions on motivations to comply have been adequately investigated in the studies. The overview brings to light that mere presence of a helmet law and positive perception about the efficacy of helmet law does not motivate compliance behaviour and more often lead to "token compliance". The meaning of helmet use should be properly understood through in-depth interviews to understand the motivations to comply. Therefore, designing study that corrects the social desirability bias and incorporates all the different forms of motivations (instrumental, normative, prudential beliefs and pluralistic ignorance) can be more informative.

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Evaluation of Gap Acceptance Behavior for Pedestrian Crossing at Mid-Block Section of an Arterial Road

Raviraj Kacha, Dipak Rathva, Manish Jain, and Sanjay Dave

Abstract Pedestrian traffic crashes have become a very common scenario on urban arterial roads because of rapid growth in developing countries like India. Due to high speed of vehicles and continuous flow of traffic, the crossing of pedestrians particularly at mid-block of an arterial road becomes concern issue of safety. The crossing behavior of pedestrian is governed by age, gender, vehicle speed, pedestrian's speed, type of following vehicle, and gap acceptance. Microscopic analysis was carried out considering different parameters like age, gender, crossing pattern, pedestrian's crossing speed, vehicle categories, vehicle speed, and vehicular gap which were extracted from videos. Gap acceptance is an indicative measure of risk-taking behavior of pedestrian while crossing the road. It was observed that female pedestrians having less risk-taking practice for crossing the carriageway are compared to male. Study also estimated critical gap using Raff's method for different age group and gender and it was found to be less for young age group compared to middle and old age group pedestrian. Similarly, critical gap for female was observed to be more than that for males. For the study site, two types of crossing patterns were observed namely straight and oblique. Majority pedestrians were found to follow the straight pattern rather than the oblique pattern. The present study helps to understand the complex pedestrian road crossing behavior which can be useful for improving pedestrian safety measures at mid-block section and further planning of the facilities for pedestrians.

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Keywords Pedestrian · Crossing behavior · Critical gap · Crossing pattern

1 Introduction

Among the all transportation users, the pedestrian is considered as a vulnerable road user, and however, they also the most overlooked for transportation planning. For transportation planners to provide a good environment for pedestrian to cross the major streets is biggest challenge for the present rapid traffic. For the emerging countries like India, due to rapid urbanization, safety is a prime concern especially for pedestrians due to increasing growth of vehicles and less obedience to the traffic rules by both drivers and pedestrians.(Asaithambi et al. 2016) The urban road development has more emphasis on achieving the higher speed of vehicles and thereby neglecting facilities for pedestrian. In India, most of the pedestrian crosswalks on urban roads are uncontrolled or unsignalized, where pedestrians are forced to find a gap to cross the section of road under mixed traffic conditions. This is quite unsafe as vehicular speed varies with the surrounding conditions and categories of the vehicles on the roads.

Gap acceptance behavior of pedestrians is governed by critical gap. Particularly focusing on the factor that influence gap acceptance and rejection using critical gap, several studies have been carried out to understand gap acceptance behavior and to provide a gap size value which can be used as a threshold value for the planning of pedestrian facilities in cities. A single value termed critical gap,t_c, defined by Raff in 1950. It is adopted to represent the nature of acceptance and rejection of gaps. It is the vehicular gap for which a number of accepted gaps smaller than t_c are equal to a number of rejected gaps larger thant_c (Jain and Rastogi 2016). There are various methods of estimating critical gap namely Raff's method, Harder's method, Ashworth's method, logit method, maximum likelihood method, probability equilibrium method, HCM method, and different minimization method.

The pedestrian walking behavior at mid-block for crossing is different compared to other locations (Kadali and Vedagiri 2013). For designing any pedestrian facility, the walking speed is the prime parameter (Yannis et al. 2010). The parameters like age, gender, luggage, etc., influence the walking speed of the pedestrian especially during crossing (Laxman and Rastogi 2010). Researcher observed that in Jordan, the female pedestrian takes more time for crossing compared to male (Tarawneh 2001). Studies reveal that the vehicle category has also impact on pedestrian safety and it is observed that light vehicles are more unsafe compared to heavy vehicles (Lefler and Gabler 2004).

The present study examines the pedestrian crossing pattern, which is also compared with inherent characteristics of pedestrian like age, gender, and group behavior of the pedestrian. Pedestrian crossing pattern is classified as straight and oblique pattern. The study explores the acceptance of vehicular gaps by pedestrian using Raff's method. Study shows how critical gap varies with different vehicular and pedestrian characteristics. It is limited in nature as it focuses on one corridor in one city, though its findings can potentially be applied to the other cities.

- Objectives of the present study are-

- To analyze pedestrian gap acceptance and rejection behavior by critical gap.
- To analyze variation of critical gap with respect to pedestrian characteristics and vehicular characteristics.
- To analyze variation in pedestrian crossing pattern with respect to age, gender, and group behavior.

2 Method of Critical Gap

2.1 Raff's Method

Raff's method is a graphical method and can be attributed as the earliest method for estimating mean critical lag. The original Raff's method considered only lags (Jain and Rastogi 2016). Miller (1972) established that this method was biased and dependent on the probability density function of lags. The plot consists of the cumulative distribution of accepted gaps,$F(t)$, with reverse cumulative distribution of rejected gaps, 1-$F(t)$, and the intersection of these two curves indicates as the mean critical gap (Jain and Rastogi 2016).

3 Data Collection

3.1 Data Collection

The data was collected by videography of the pedestrian-vehicle conflict area and the corresponding traffic stream. The site selected for data collection was the mid-block section of a roadway of Surat city (Gujarat). The study section was selected where ample pedestrian crossing and vehicular movements were conflicting. For the present work, a videography survey was carried out to study the pedestrian crossing behavior, during the evening peak hours on a mid-block section of six-lane divided arterial road having effective width of 10 m on either side. The selected arterial road has a separate BRTS lane at the median and it witnessed the pedestrian flow of 8–10 pedestrian/minute crossing the road in either direction, i.e., from kerbside to median and median to kerbside.

3.2 Definition of Lag and Gap

It is essential to define the various microscopic events which are to be recorded for gap studies (Jain and Rastogi 2016). Pedestrians' crossing movement from kerbside to median and median to kerbside have been studied on one side of the divided carriageway. Figure shows the direction of the traffic flow (Jain and Rastogi 2016). Let the arrival time of a pedestrian in the study area ist_0, at timet_0, the distance between pedestrian and head of first vehicle (distance between A–A and B–B) is known as a spatial lag. Time taken by the first vehicle to reach conflict area A–A is defined as temporal lag or simply lags. If pedestrian rejects that lag, then the distance between tail of first vehicle and head of successive vehicle (distance between B–B and C–C) is known as a spatial gap. Time taken by the successive vehicle to cover a spatial gap is known as temporal gap or gap (Jain and Rastogi 2016) (Figs. 1 and 2).

Fig. 1 Study area of Surat city

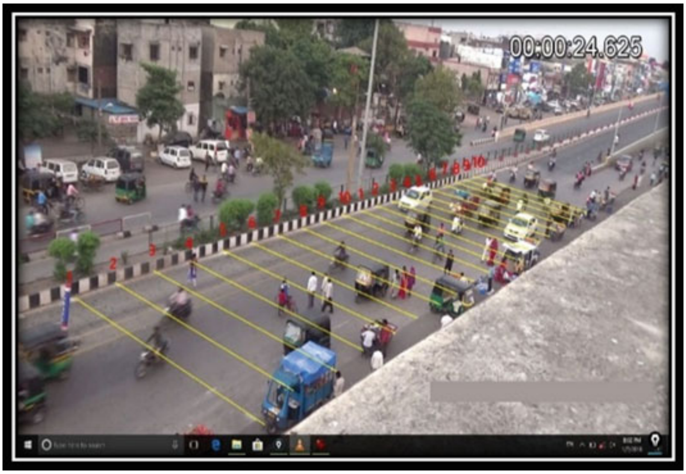

Fig. 2 Trap lines which useful in data extraction

4 Data Extraction

Data extraction is the most important task of the project. Data extraction starts with drawing trap lines on road points on both sides of the carriageway at 2.5 m interval along the length of road. So that it is helpful for data extraction like gap size and vehicle speed. Virtual lines are plotted on the marked points and embedded in video using software. Figure shows the camera view of the sites used for data extraction after embedding the virtual trap lines in the video. To extract the starting time, ending time and waiting time of pedestrian crossing, frame by frame technique was used. The video is played frame by frame several times to record time of vehicle head and vehicle's tail crossed a trap line. Both pedestrian and vehicular data were used to process the pedestrian decision of gap acceptance. Total of 2240 data points were extracted and consisted of pedestrian age group, gender, crossing time, crossing speed, gap size, acceptance/rejection of gaps, type and speed of the following vehicle.

5 Critical Gap Analysis

In Raff's method, the cumulative distribution of accepted gaps,$F_{\mathrm{ar}}\,(t)$, and the rejected gaps, 1-Fr (t), are plotted and their intersection is called critical gap. In the present study, variation of critical gap with respect to the inherent characteristics of pedestrian like age group, gender, and type of the following vehicle is analyzed.

5.1 Critical Gap as Per Age Groups

For the present study, the pedestrians are grouped into three age groups, as young age (10–25 years), middle age (25–50 years), and old age (above 50 years) (Fig. 3).

From the observations, it was found that critical gap varies with the change in the age group. As critical gap value reveals the risk-taking behavior, it was seen that as the size of critical gap increases, risk-taking behavior decreases. From the graph, it was observed that the critical gap for young age group pedestrians comes lower which reveals that this age group has high risk-taking behavior than middle age and old age group pedestrians.

5.2 Critical Gaps as Per Gender Classification

From the graph, it was observed that female pedestrians having a higher critical gap, which shows that female pedestrians are more concerned about safety while crossing the road compared to male pedestrians (Fig. 4).

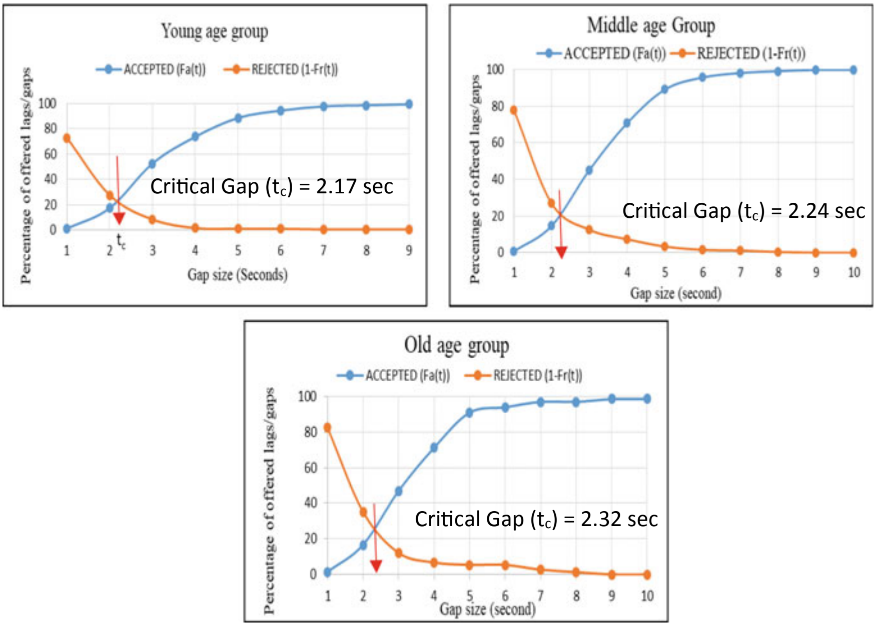

Fig. 3 Critical gap as per age group

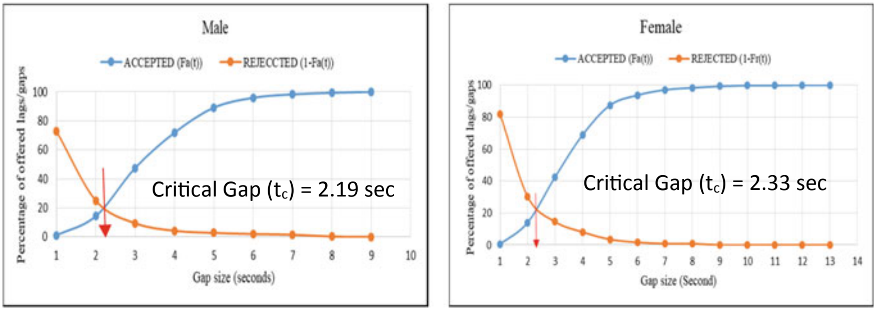

Fig. 4 Critical gap as per gender

5.3 Critical Gap as Per Single or Group of Pedestrian

In the present study, critical gap was analyzed for single pedestrian and for a group of pedestrians (platoon). Here, the group of two or more pedestrians with same crossing behavior like crossing start time, crossing end time, crossing speed, crossing pattern, and waiting time is considered as platoon.

In group behavior, the individual pedestrian is surrounded by other pedestrians and therefore their action is dependent on action taken by the whole group. It is

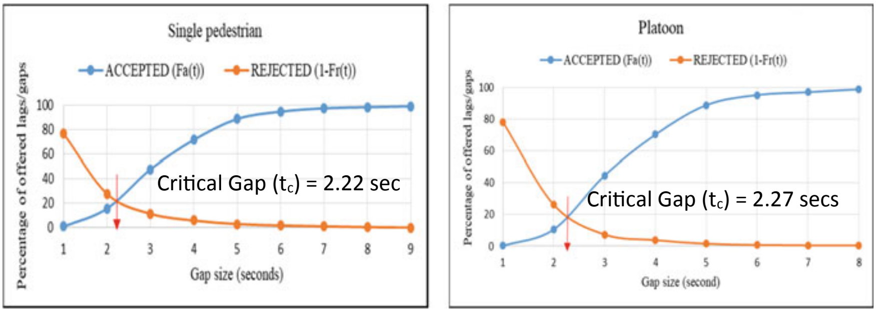

Fig. 5 Critical gap as single or group

also observed that the risk-taking behavior of platoon is lower compared to the single pedestrian as the decision of crossing the road is governed by the group of pedestrians as shown in (Fig. 5).

5.4 Critical Gap as Per Type of Following Vehicle

Various types of vehicles were found in the traffic flow stream, but the large share of motorized two-wheeler, three-wheeler, and car was found in the stream, and due to that, critical gap is found in these three different categories.

From the graphs, it is seen that critical gap varies with the type of the following vehicle. It was observed that the critical gap is found to be higher when the following vehicle is car compared to when the following vehicle is two-wheeler or three-wheeler, which reveals that pedestrian feels higher risk when the following vehicle is car compared to when the following vehicle is two-wheeler or three-wheeler (Fig. 6).

6 Critical Gaps and Pedestrian Crossing Pattern

6.1 Comparison of Critical Gap and Average Accepted Gap

Critical gap reveals the road crossing behavior of pedestrian which varies with various vehicular and pedestrian characteristics. Average accepted gap is much larger than the critical gap measured by Raff's method which suggests that pedestrian required more gaps to cross the road. Table 1 presents the variation of critical gap between the age groups, gender, type of following vehicles, and group behavior of pedestrian shows similar kind of behavior in both the methods.

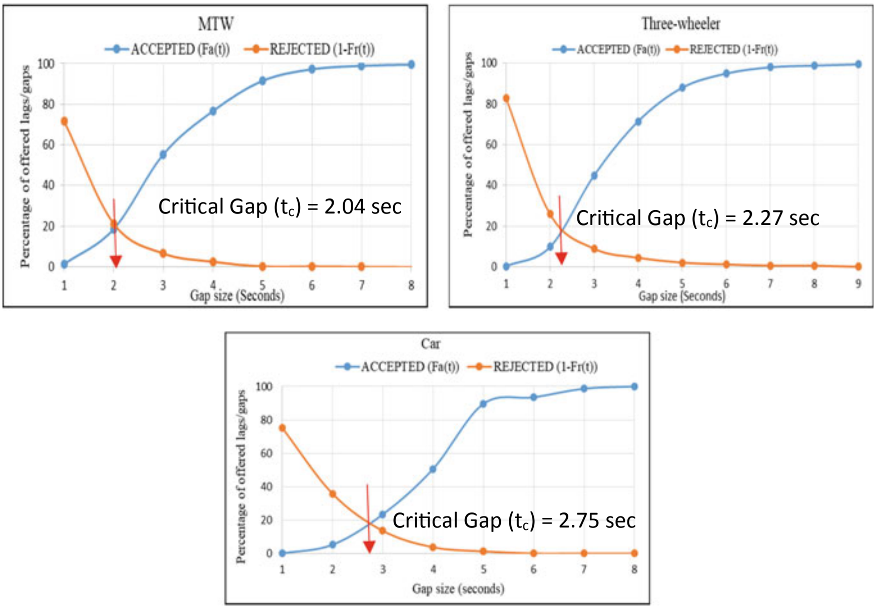

Fig. 6 Critical gap as per following vehicle category

Table 1 Comparisons of critical gap and average accepted gap

Characteristics	Range of characteristics	Critical gap (sec)	Average accepted gap (Sec)
Age group	Young age (10–25)	2.17	3.33
	Middle age (25–50)	2.24	3.37
	Old age (50+)	2.32	3.48
Gender	Male	2.19	3.36
	Female	2.33	3.39
Type of following vehicles	2-wheeler	2.04	3.15
	3-wheeler	2.27	3.47
	Car	2.75	3.86
Group behavior	Single	2.22	3.36
	Platoon	2.27	3.50

6.2 Statistical Analysis

The accepted and rejected gaps are extracted for male and female with different age groups and different group behavior. Variation in accepted gaps for male and female for different age groups is checked statistically using one way ANOVA at 95%

Table 2 one-way ANOVA test for gender with different age group and group behavior

Comparison	Characteristics	Statistics	Remarks
Male versus female	Young age (10–25)	$F = 0.014$, $P = 0.90$	Accepted
	Middle age (25–50)	$F = 4.24$, $P = 0.04$	Rejected
	Old age (50+)	$F = 8.00$, $P = 0.007$	Rejected
Single versus platoon	–	$F = 4.04$, $P = 0.04$	Rejected

confidence interval. The null hypothesis is considered as no variation in accepted gaps for male and female and different group's behavior (Table 2).

6.3 Pedestrian Crossing Pattern

Pedestrian crossing pattern varies with vehicular characteristics and pedestrian characteristics. Due to variation in size of gaps pedestrian, which always look to accept the gap, they change the path of crossing in oblique or rolling type. Pedestrians who are not looking forward to accept gap reject the gaps and wait for a suitable gap and also they are not interested in changing their path crossing the carriageway in straight path. So, it is observed that the crossing pattern is mainly dependent on inherent characteristics of pedestrian like age and gender (Table 3).

Table 3 shows the crossing pattern variation with respect to gender, age group, and single pedestrian or group of pedestrians. From the observations, it was seen that the pedestrian is more interested to cross the road in straight pattern as it reduces the effective time of crossing in the conflict area which leads to safe crossing behavior. It was also observed that the male pedestrian always looks for opportunity to accept the gap and doing that they also change their crossing pattern from straight to oblique or rolling. In case of female pedestrians, a high percentage of crossing in straight pattern was observed which suggests that they prefer to wait for a suitable gap rather than changing the crossing pattern. There is no variation observed in the crossing pattern for various age groups. Significant variation in the crossing pattern is observed when the individual pedestrian is crossing or group of pedestrian. In platoon, pedestrians feel safe so they are not interested in changing their crossing pattern thus forcing the following vehicle to lower their speed or to change the lane.

Table 3 Pedestrian crossing pattern

Crossing pattern	Gender		Age group			Single or group	
	Male (%)	Female (%)	Young (%)	Middle (%)	Old (%)	Single (%)	Platoon (%)
Straight	50.45	65.83	56	57	58	56.84	69.30
Oblique	49.54	34.16	44	43	42	43.15	30.69

7 Conclusion

- Critical gap varies with inherent characteristics of pedestrian, type of following vehicle, and group behavior.
- Critical gap for male found to be lower than the female which shows that male is having higher risk-taking behavior.
- It is observed that, pedestrians of younger age group have a higher risk-taking behavior than middle age and older age group.
- Critical gaps show that pedestrian prefers a larger gap to accept when the following vehicle is car in comparison with two-wheeler and three-wheeler.
- Statistical analysis shows that there is no effect of gender in accepted gap for the younger age group, while there is an effect of gender observed in accepted gap for middle and older age group.
- There is a significant difference in accepted gap value for a single pedestrian and platoon.
- It is also found that nearly 50% male pedestrian is having oblique crossing pattern which shows that they always look for an opportunity to accept a gap as compared to female (35%).
- When pedestrians are crossing in platoon, 70% platoon crossing pattern was straight which shows that they will not change their path, while 56% of single pedestrian cross the road in straight path.

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Simulation of Pedestrian Movement Over Different Facilities in Gangtok Using VISWALK Software

Vishal Kumar, Arunabha Banerjee, and Akhilesh Kumar Maurya

Abstract The main objective of the current study was to develop simulation models for the different pedestrian facilities (such as sidewalk, walkway and foot over bridge) in Gangtok near M.G. Marg, and estimate the capacity for such facilities. The data collected from the field was used as input for the development of the simulation models using VISWALK commercial software, and thereafter the models were calibrated and validated accordingly. The developed model was calibrated using VISWALK COM Interface programming in MATLAB. Genetic algorithm (GA) was used to calibrate the sensitive parameters affecting walking speed. The calibrated models were then validated using a new set of field data and the mean absolute percentage error (MAPE) of the models was computed. The results showed that male pedestrians moved with higher speed than female pedestrians by 3.5–12 m/min. Similarly, the pedestrians with luggage moved at a speed lower than the ones without luggage (~2–5 m/min). The average walking speed over the sidewalk was found to be more than the walkway. Using sensitivity analysis for the simulation, the reaction time of the pedestrians (tau B), direction-dependent force (B_soc_isotropic) and strength of the speed-dependent social force (A_soc_mean) were found to have a greater impact on the average simulated walking speed and were thus further considered for calibration and validation. It was also observed that the MAPE between observed and simulated speed ranged between 4.06 and 7.57% for the different facilities. Moreover, the correlation between simulated and observed walking speed was found to be between 0.81 and 0.98, and the *P*-values for both were found to be quite similar. The simulated capacity values for the different facilities were also estimated and it could be seen that the capacity was dependent on the width, location and type of the facility.

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Keywords Pedestrian · VISWALK simulation · Genetic algorithm · Sidewalk · Walkway · Foot over bridge

1 Introduction

Walking is the basic mode of transport, and may not represent a large part of our daily travelled distance, but pedestrians are an essential part of the traffic system. To reduce congestion over roads, instead of travelling by private cars, alternative modes such as walking, cycling and using public transport can be used. Walking is an important mode of transport as almost all trips performed using the mode. In the recent past, there has been a growing interest in understanding pedestrian behaviour which forms the largest single road user group. The walking behaviour of pedestrians along with flow characteristics are the basis for planning and designing of proper pedestrian facilities. Moreover, efficient multimodal travel would not be possible without effective pedestrian facilities connecting the different modes of transport.

To evaluate different layout scenarios or adapt to a sudden increase in capacity of a particular pedestrian facility in the near future, microscopic simulation modelling can be used. Using microscopic simulation, the interaction between pedestrians can be studied, as each pedestrian is considered as an individual. VISWALK microscopic pedestrians simulation software is used in this work, which is based on the Social Force Model (SFM) developed by Helbing and Molnar (1995). Social force model consists of several parameters known as walking behaviour parameters which affect the pedestrian walking speed significantly under different situations. Later, the model was further modified by Helbing et al. (1998), (Johansson et al. 2008 and Helbing and Johansson 2011) as per requirements for better applicability and performance.

Previously, Ishaque and Noland (2005) and Galiza et al. (2010), used VISWALK software in order to understand the relationship between observed (field) flow rate and walking speed based on simulation results. In another study by Galiza et al. (2011), it was seen that the pedestrian space was a function of flow rate and speed. Moreover, results also predicted that with the increase in the proportion of older pedestrians, the average space decreased. Yu et al. (2006) used Genetic Algorithm (GA) to arrive at optimal parameter values which minimized the sum of squared error (SSE) of a microscopic simulation model. Similarly, Tettamanti et al. (2015) used GA to calibrate and optimize the model parameters in VISSIM. Kretz et al. (2008) in a simulation study observed that with small changes in bottleneck width, large impacts were observed both in the model as well as reality. Similarly, a simulation study by Ko et al. (2013) predicted that with an increase of 10, 20 and 30 elderly pedestrians, the average speed decreased exponentially.

Table 1 Details of locations for data collection

Facility type	Location	Length (m)	Width (m)	Start time	End time
FOB (Site 1)	Sonam Gyatso Marg	10.0	2.50	10:55 AM	4:30 PM
Sidewalk (Site 2)	Near Babumoshai Restaurant	10.0	2.40	3:30 PM	7:10 PM
Sidewalk (Site 3)	Near Denzong Cinema	10.0	2.19	12:15 PM	4:45 PM
Sidewalk (Site 4)	Near Denzong Cinema	10.0	1.44	10:50 AM	5:30 PM
Walkway (Site 5)	New Market to MG Marg FOB	7.50	4.55	10:05 AM	4:30 PM
Walkway (Site 6)	MG Marg FOB to New Market	7.50	4.55	12:50 PM	5:15 PM

2 Data Collection and Analysis

The data was collected using videography techniques from the capital of Sikkim state, Gangtok. The locations were visited prior to data collection and pedestrian facilities that had a significant high flow of pedestrians were chosen for final videography.

2.1 Data Collection Sites

The data was collected during peak hours on weekdays using videography techniques. The camera with the tripod stand was fixed at an altitude of approximately 10 m from the ground. The duration of data collection ranged between 3 and 5 hours. Data was collected for pedestrians moving in both directions of travel. The length and width were marked in order to observe the pedestrian movement within the stretch. The effective width was considered after deducting the static obstacles. The details of the pedestrian facilities from which data were collected are given in Table 1. Table also includes the duration of data collection.

Figures 1 and 2 depict the location of the sites 1 and 2 from which data were collected.

Based on the data collected from different locations, the data were processed in the lab for analysis.

2.2 Data Extraction

The videography data collected from the six locations were processed in the lab to extract the speed, flow and density parameters. For speed (m/min) observation, random pedestrians were selected moving in both directions and their travel times were noted and trap length was used for estimation of the speed. Flow rate

Fig. 1 Location of FOB (Site 1)

Fig. 2 Location of sidewalk (Site 2)

(ped/min/m) was calculated by finding the ratio between the number of pedestrians crossing per minute (ped/min) to the effective trap width (m). Density (ped/m^2) was estimated from the fundamental relationship, $q = k*v$. Space or area module (m^2/ped) was also calculated as the inverse of density. Based on the extraction, the simulation models were developed using the VISWALK software.

2.3 Model Development

VISWALK software (PTV AG 2013) was used to develop microscopic simulation models for the different types of pedestrian facilities. The geometry of the foot over bridge, walkways and sidewalks were defined as measured from the field. The type of flow considered for this experiment was bidirectional. In order to define the geometry of the study area in the VISWALK, set scaling option was employed. Two types of pedestrian were defined in the model, man and women. Walking behaviour parameters, desired speed distribution, pedestrian inputs, routes and composition were added to the geometry. In addition to these, the pedestrian characteristic body size of males and females were also defined in the model. Figures 3 and 4 depict the simulated environment created for sites 1 and 2.

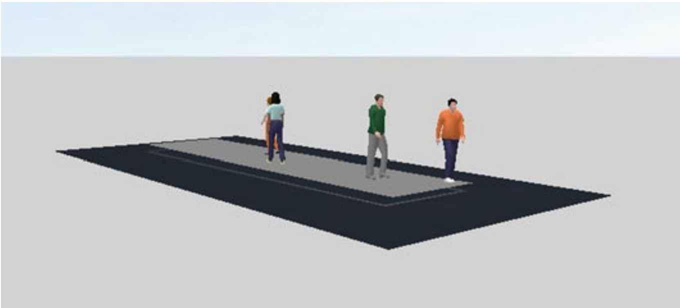

Fig. 3 Simulated FOB (Site 1)

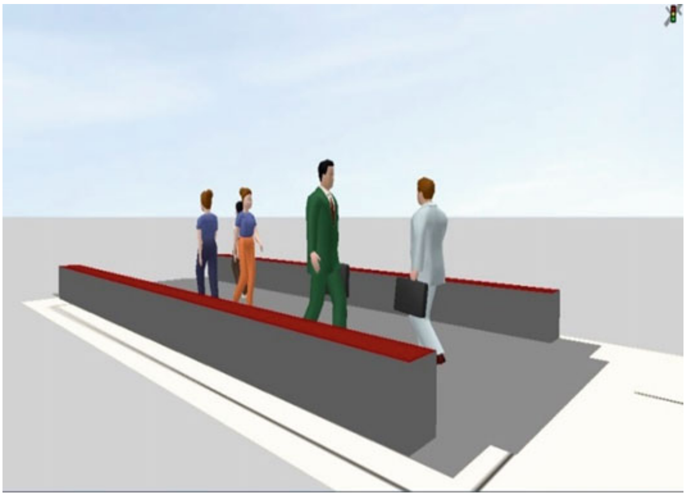

Fig. 4 Simulated sidewalk (Site 2)

2.4 Sensitivity Analysis

Sensitivity analysis was carried out to examine the sensitivity of simulated walking speeds were to change in selected parameters of the social force model. The investigated parameters were: tau (relaxation time in seconds), A_soc_isotropic and B_soc_isotropic (strength of force between pedestrians in m/s^2), A_soc_mean and B_soc_mean (range of force between two pedestrians in meters) and VD (impact of relative velocity between pedestrians). In order to find critical values of the parameters, minor changes were made repeatedly. The critical values which had a significant influence on the walking speed of the pedestrians in the simulated models have been explored by using both the minimum and maximum values.

2.5 Analysis of the Simulation Parameters

The results of the sensitivity analysis showed that parameter tau had a significant impact on the average walking speed observed in simulation. Higher values of tau produced lower average simulated walking speed and decreased the pedestrian acceleration as well. Similarly, higher values of A_soc_isotropic parameter corresponded to stronger repulsive force (F_1) and reduced average simulated walking speed.

Also, higher values of B_soc_isotropic produced lower average simulated walking speed. Thus increase in B_soc_isotropic parameter, increased the sizes of the pedestrians which resulted in pedestrians slowing down to avoid hitting each other.

Higher values of A_soc_mean (controlling the strength of force F_2) produced higher values of average simulated walking speed along with stronger force. B_soc_mean parameter had no influence on the average simulated walking speed. Finally, higher values of VD resulted in higher average simulated walking speed. Thus, when VD increased, d_m (i.e., the distance between two pedestrians) decreased and consequently F_2 increased.

2.6 Model Calibration

Models created in VISWALK were calibrated and adjusted to account for the different parameters of the simulation model so as to replicate the actual field conditions. The input parameter values were varied until and unless the error obtained between the simulated and actual measures like walking speed, flow, etc. were less than the pre-decided threshold value.

Manually calibrating VISWALK models is time-consuming, and so genetic algorithm (GA) technique was used for the calibration of walking behaviour parameters of the models. In order to find the mean absolute percentage error (MAPE) between the actual and simulated measures, GA was used to generate a random set for parameters (within specified bounds). Following Eq. 1 was used as a fitness function in terms of speed. V_{obs} and V_{sim} are the observed and simulated speeds, respectively.

$$f = \frac{1}{N}\sum_{i=1}^{n}\left|\frac{V_{\text{obs}} - V_{\text{sim}}}{V_{\text{obs}}}\right| \times 100 \tag{1}$$

Options were set in genetic algorithm command line such as population size, number of generations, crossover, mutation, level of display, etc. Initial population was defined and the population size was fixed at 16 with total generation of 20 while keeping other parameters of genetic algorithm as default values. Number of generations was set as stopping criteria. A close look at Figs. 5 and 6, revealed that

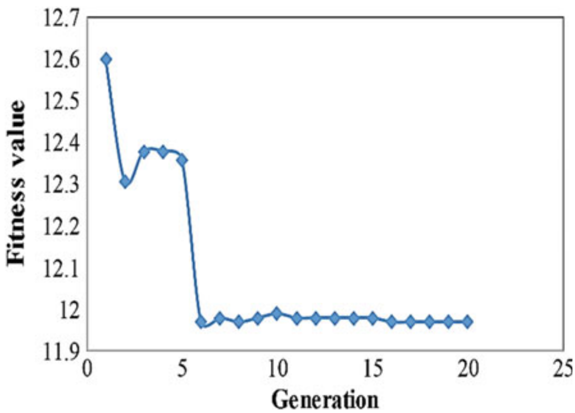

Fig. 5 Fitness versus generation (Site 1)

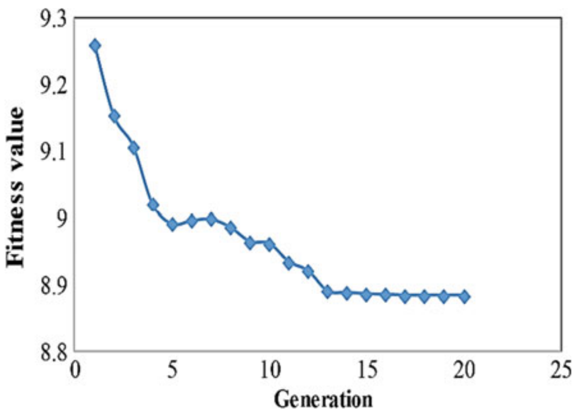

Fig. 6 Fitness versus generation (Site 2)

Table 2 Calibrated parameters for models of various sites

Site	Facility	Optimal parameter values				
		A_soc_isotropic	B_soc_isotropic	A_soc_mean	Tau	VD
1	FOB	1.339	0.107	0.555	0.104	7.655
2	Sidewalk	1.277	0.127	0.684	0.131	6.896
3	Sidewalk	1.386	0.139	0.578	0.128	7.589
4	Sidewalk	1.368	0.115	0.598	0.107	7.453
5	Walkway	1.416	0.124	0.579	0.118	7.333
6	Walkway	1.328	0.110	0.564	0.113	7.218

with an increase in number of generation average fitness, values improved and after a certain generation, it did not change much and became constant. Therefore, the sets of parameters corresponding to minimum fitness value were considered as optimized parameters.

Table 2 shows the optimal values of the calibrated parameters for using GA. The calibrated models were further validated with a different set of field data.

2.7 Model Validation

The calibrated models were then assessed using a new set of field data (with different input volumes, composition, desired speed, etc.). The optimal parameters obtained from the calibration were put in the model with new data set. Mean absolute percentage error of the models was computed (refer Table 3) and errors were within desired limits.

The relationship between observed and simulated data for basic fundamental parameters (speed, flow and density) was compared below through scatter diagram (refer Figs. 7, 8, 9 and 10). From the following figures, it was found that the calibrated model matched with the field observation within a small error range.

Table 3 Results of validation based on calibration for different facilities

Site	Facility	Width (m)	MAPE (%)
1	Foot over bridge (FOB)	2.50	7.57
2	Sidewalk	2.40	5.91
3	Sidewalk	2.19	4.06
4	Sidewalk	1.44	6.03
5	Walkway	4.55	4.94
6	Walkway	4.55	4.36

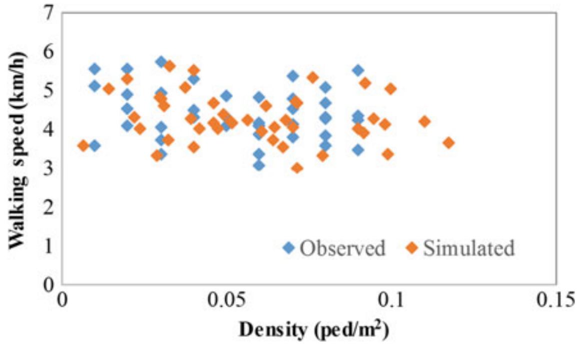

Fig. 7 Speed versus density (Site 1)

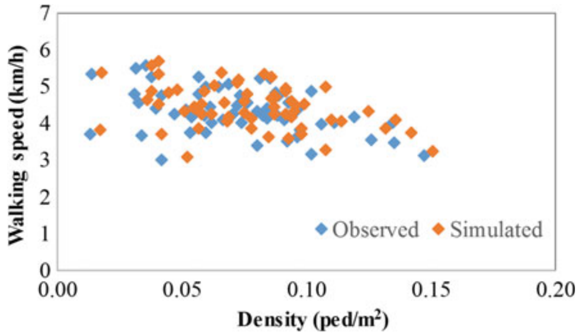

Fig. 8 Speed versus density (Site 2)

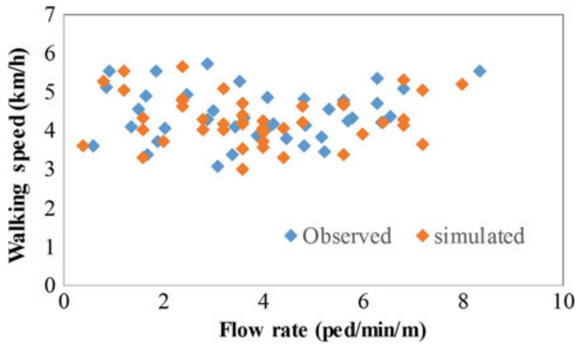

Fig. 9 Speed versus flow rate (Site 1)

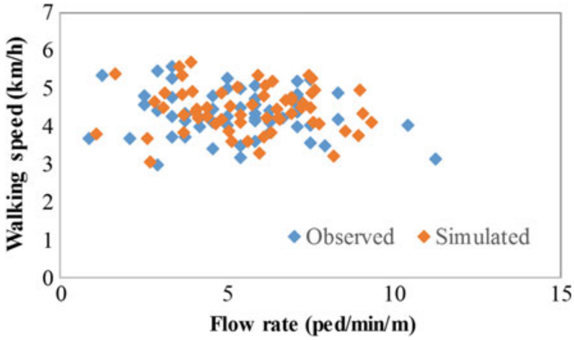

Fig. 10 Speed versus flow rate (Site 2)

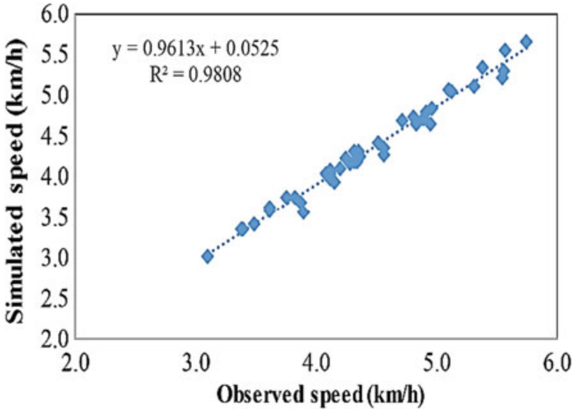

Fig. 11 Simulated versus observed speed (Site 1)

2.8 Linear Regression Modelling (LRM)

Scatter plots were developed between observed versus simulated speed as well as observed flow versus simulated flow and they are shown in Figs. 11, 12, 13 and 14.

2.9 Hypothesis Test for Regression Slope

Hypothesis testing was performed to determine whether any significant linear relationship exists between an independent variable (observed flow or observed walking speed) and dependent variables (simulated flow or simulated walking speed). The null hypothesis states that the slope of the regression line is equal to zero, which

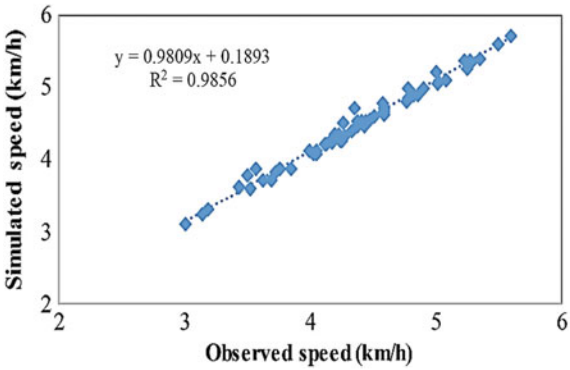

Fig. 12 Simulated versus observed speed (Site 2)

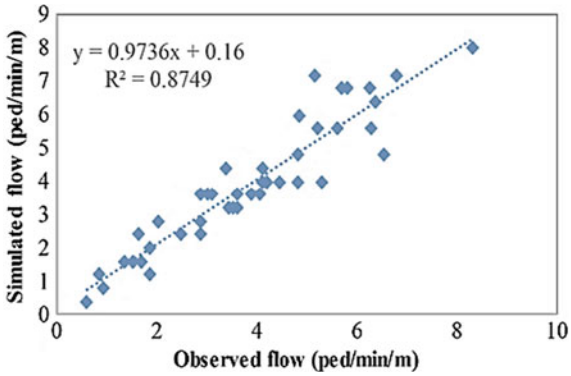

Fig. 13 Simulated versus observed flow (Site 1)

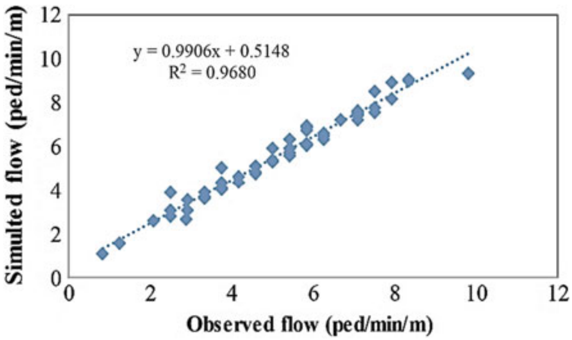

Fig. 14 Simulated versus observed flow (Site 2)

Table 4 Linear regression analysis results of observed speed versus simulated speed for different facilities

Site	Facility	Slope	Intercept	R^2	P value	Null hypothesis
1	FOB	0.9613	0.0525	0.9808	8.77E-38	Rejected
2	Sidewalk	0.9809	0.1893	0.9856	2.78E-55	Rejected
3	Sidewalk	1.0949	−0.3262	0.9381	2.31E-37	Rejected
4	Sidewalk	0.9473	0.3281	0.8139	3.17E-23	Rejected
5	Walkway	0.9575	0.3238	0.9807	1.62E-52	Rejected
6	Walkway	0.9728	0.1677	0.8405	3.28E-25	Rejected

Table 5 Linear regression analysis results of observed flow versus simulated flow for different facilities

Site	Facility	Slope	Intercept	R^2	P value	Null hypothesis
1	FOB	0.9736	0.16	0.8749	1.39E-20	Rejected
2	Sidewalk	0.9906	0.5148	0.9680	4.28E-45	Rejected
3	Sidewalk	1.1295	−0.8656	0.6248	3.55E-14	Rejected
4	Sidewalk	1.258	−2.1304	0.7423	4.91E-19	Rejected
5	Walkway	1.1162	0.2407	0.6791	3.40E-16	Rejected
6	Walkway	1.0349	0.8527	0.3981	5.00E-08	Rejected

means that the points between the independent and dependent variables are explicitly random and there is no relationship between them. In this case, the line drawn between them will be a flat horizontal line and the alternative hypothesis states that since the slope is not equal to zero, there is a significant relationship between those variables. From Tables 4 to 5, it is inferred that null hypothesis was rejected which implies that there is a significant relation between dependent and independent variables at 5% significance level. Moreover, P-values were less than 0.05, which indicated that these relationships are statically significant.

2.10 Capacity Analysis of Different Types of Pedestrian Facilities

Capacity is known as the maximum flow rate in speed—flow curve. The conditions at maximum flow represent the capacity of pedestrian facilities. After calibration and validation of the simulation models of different pedestrian facilities, the models have been employed to estimate the capacity at optimize model parameters and input parameters of pedestrian traffic. The simulated speed-flow rate and flow rate–density relationships were compared with observed speed-flow rate and flow rate–density relationships (refer Figs. 15, 16, 17 and 18).

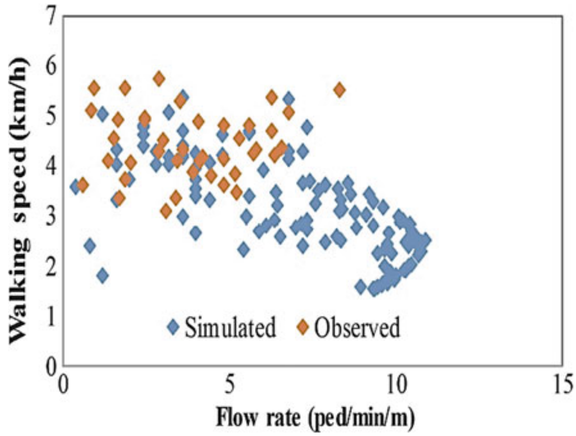

Fig. 15 Speed versus flow rate (Site 1)

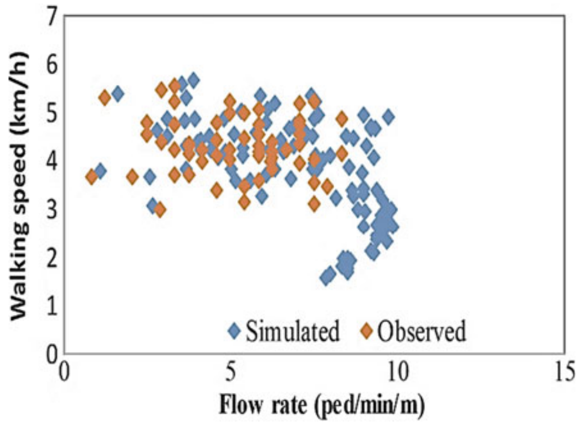

Fig. 16 Speed versus flow rate (Site 2)

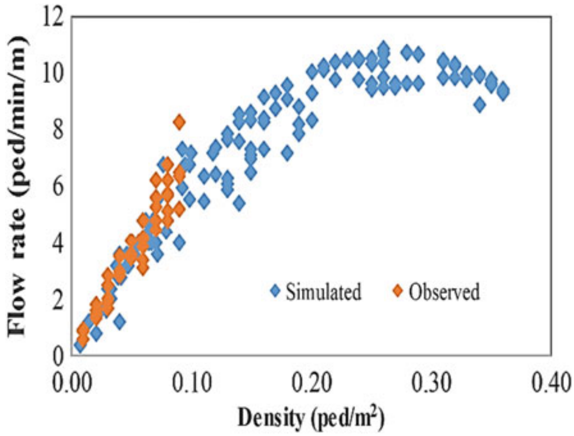

Fig. 17 Flow rate versus density (Site 1)

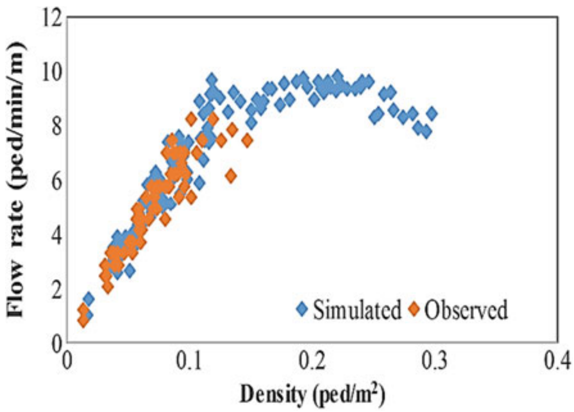

Fig. 18 Flow rate versus density (Site 2)

Table 6 Estimated simulated capacity (maximum flow rate) of different pedestrian facilities

Site	Type of facility	Width (m)	Capacity (ped/min/m)
1	FOB	2.50	10.65
2	Sidewalk	2.40	9.67
3	Sidewalk	2.19	27.94
4	Sidewalk	1.49	23.16
5	Walkway	4.55	18.12
6	Walkway	4.55	26.63

Table 6 shows the estimated simulated capacity of the different facilities considered in the study.

3 Results and Discussion

In this study, microsimulation software VISWALK was used. Further, VISWALK GUI was called into MATLAB through COM Interface programming. The salient features of the study included the investigation of sensitive walking behaviour parameters using sensitivity analysis. Based on the sensitivity analysis using VISWALK, it is inferred that lower values of Tau and B_soc_isotropic, and higher values of A_soc_mean produced higher walking speed. Similarly, lower values of A_soc_isotropic and higher values of VD produced higher walking speed.

The calibration using genetic algorithm showed the range of the sensitive parameters (Table 2). The validation of the models predicted the mean absolute percentage error (MAPE), and it was found that calibrated models matched with the field observations within small error range between 4.06 and 7.57%. Hypothesis test of regression slope indicated the existence of a significant relationship between observed and simulated data. Moreover, from Tables 4 and 5 it is evident that since the null hypothesis of the regression slope being rejected, it is inferred that significant relationship exists between observed versus simulated data.

The simulated speed-flow rate and flow rate-density relationships showed similar trends to the observed field speed-flow rate and flow rate-density relationships. Moreover, the simulated capacities predicted that other than width, the location and type of the facility also affected the maximum flow rate values for the different sites. The reasons for such observations could be because as at site 5, the flow being lower, pedestrians were able to choose their desired speed and hence variability of speed was more. While on site 6, flow being higher led to lesser chance for pedestrians to choose their desired speeds and hence somewhat forced them move with the crowd. For the three sidewalks observed, site 2 had a relatively lower capacity in comparison to sites 3 and 4. The capacity observed for sites 3 and 4 ranged between 23 and 28 ped/min/m, while that observed for site 2 was approximately 10 ped/min/m. The higher flow of pedestrians was observed at sites 3 and 4 as these facilities were located adjacent to public transport terminal, while site 2 was located in an area where there were very few pedestrians using the facility. Sites 5 and 6 which were walkways had different capacities even though they had the same width and located in similar location types (shopping area). Flow rate at site 6 was observed to be nearly 27 ped/min/m, while it was around 18 ped/min/m for site 5. The main reasons for such huge differences were due to the fact that site 6 was a continuous section consisting of various types of shops, which connected M.G. Marg to New Market, and hence more pedestrians were attracted to use this section. On the other hand, site 5 had a discontinuous section with many restaurants, hence only a particular group of pedestrians were attracted to this location. The capacity observed for site 1 was around 11 ped/min/m and was relatively low as it was a FOB near a Telephone Exchange office where less number of pedestrians used the facility.

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A Study on Understanding the Factors Influencing Pedestrian Inclination Towards Using Pedestrian Bridges

Arunabha Banerjee and Akhilesh Kumar Maurya

Abstract The study aimed to understand the factors profoundly affecting the present motive of the pedestrians to use the foot overbridges (FOBs) in two different cities of India and also put forward recommendation based on the pedestrians' suggestions to improve its future usability. An interviewer-administered questionnaire survey was conducted over two cities, namely Kolkata and Bengaluru covering various FOBs. The perceptions of seven different factors related to the condition of the FOBs were ranked on a five-point Likert scale from very poor to very good. The demographic survey showed that majority of the pedestrians using the FOBs were male pedestrians and aged between 23 and 45 years. Also, students and servicemen preferred to use the FOBs more than the others. Using multinomial linear regression (MLR) method, it was observed that in Bengaluru, factors such as security and surface affect the preference of the pedestrians towards using the FOBs. Similarly, in Kolkata, security along with width and comfort significantly affects the preference of the pedestrians. The survey results indicate that to improve future usability of FOBs, installation and maintenance of lift/escalators/ramps along with proper security (in the form of CCTV camera and security personnel) were essential. The study also put forward specific recommendations which provide valid information to the planners and designers, to improve the existing FOBs or construct FOBs in future to attract more pedestrians.

Keywords Foot over bridge · Pedestrian · Perception survey · Future usability · Multinomial logistic regression

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1 Introduction

As per IRC: 103 (2012), walking is a mode of transportation where pedestrians cover 1–2 km length of their urban trips. The pedestrians are known for their variability in choosing their trend of walking, which makes them unpredictable in comparison with motorized traffic. In India, the pedestrian facilities are either occupied by vendors, maintained or unavailable, which makes it difficult for the pedestrians to use such facilities. Due to the unavailability of proper pedestrian facilities, the pedestrians are forced to use the carriageway or use illegal means to cross the busy roads. This, in turn, becomes fatal sometimes as the pedestrians are exposed to the surrounding environment and are in higher chance of interacting with the motorized traffic, which makes them far more vulnerable than other road users.

The N.C.R.B. Report (2015) revealed that 22,375 out of 2,09,796 (~10.7%) of pedestrian road accidents took place at urban road crossings. Moreover, as per Road Safety in India Status Report (Mohan et al. 2016), it was reported that nearly 40% of the fatalities were related to pedestrians directly and mostly occurred in metropolitan cities like Kolkata, Bengaluru and Delhi. Thus, to improve the existing situation, it is imperative to separate the pedestrian and vehicular traffic interactions entirely through grade-separated facilities (foot overbridges or subways). Even though both subways and foot overbridges (FOBs) provide similar access to the pedestrians, yet the poor safety and security in subways makes it less preferable than FOBs. The limited number of perception studies conducted over FOBs in different cities of India motivates research in this particular area. The following section provides a detailed review of the available literature related to pedestrian foot overbridges.

2 Literature Review

Foot overbridges/overpasses are elevated pedestrian facilities which allow complete segregation of pedestrians from the motorized traffic, to enhance safety (Ribbens 1996). In India, FOBs are provided without proper due consideration of the need for the development of such facilities. Moreover, the absence of lifts/escalators/ramps, presence of unwanted people and absence of security personnel/CCTVs make it difficult for a pedestrian to use the facilities and encourage illegal at-grade crossing. It is thus extremely important to analyse the requirement of such facilities at proper locations. Moreover, adequately designed elevated facilities would also encourage pedestrians to use the facilities and avoid unauthorized crossings through median openings or cuttings.

In Uganda (East Africa), Mutto et al. (2002) observed that even though 77.7% felt that overpass was a better option to cross the busy streets, yet the presence of high stairs, extra walking involved and fear of security played a negative role over the psychology of the pedestrians. The researchers also suggested that the development of median barriers would prevent at-grade crossings. Räsänen et al. (2007) observed that in Ankara (Turkey), the use of FOBs varied between 6 and 63% depending on the location of the facility and the presence of escalators. Moreover, the safety and familiarity with the neighbouring also affected the pedestrian motivation towards using the facility. In Amman (Jordan), Abojaradeh (2013) used questionnaire survey to understand the factors influencing the use or non-use of pedestrian bridges, and factors such as posted speed limit, walkway width and existence of median barrier were seen to have a positive impact on the use of FOBs. In Bangladesh, Saha et al. (2013) and Pasha et al. (2015) observed that insufficient security especially at night time (due to the absence of proper lighting and presence of unwanted pedestrians) prevented the pedestrians from using the FOBs to prevent chances of mugging. Moreover, the presence of an obstruction (in the form of vendors and standing pedestrians) forced the users to cross at-grade crossings illegally instead of using the overpasses. Rankavat and Tiwari (2016) found that about 95% used the FOBs in Delhi (India) if the time taken over FOBs and at-grade crossings was same. The study also showed that with the increase in age, the usage rate of FOBs decreased due to the extra effort required in climbing. In Barraquilla (Colombia), a study by Oviedo-Trespalacious and Scott-Parker (2017) revealed that security was the most important factor which affected pedestrians' choice of using the elevated facility as robbery and other assaults were common over such bridges due to the absence of proper security. The study by Malik et al. (2017) in Karachi (Pakistan) also revealed that security along with comfort played a pivotal part in the selection of the overbridges as female pedestrians (61.4%) felt in-secured especially at night time. The researchers suggested that proper installation of lights along with proper placement of security personnel would encourage pedestrians to use the bridges even at night time.

To understand the actual scenario of the utilization of FOBs, it is essential to conduct questionnaire/perception surveys. Moreover, through a questionnaire survey, the actual condition of the bridges can be understood, and future proposals could also be put forward which might increase the use of the bridges. Therefore, the study aimed to understand the factors profoundly affecting the present motive of the pedestrians to use the bridges and also put forward recommendation based on the pedestrians' suggestions for increasing future usability.

3 Data Collection

This section presents the details of the data collection sites and procedure for data collection.

3.1 Data Collection Locations

The pedestrian questionnaire survey was conducted over two cities, namely Kolkata and Bengaluru covering seven FOBs across various land use types. Table 1 shows the

Table 1 Location details

City	Location Name	Type of Land Use	Width (m)	Number of steps	Stair width (m)	Tread (cm)	Riser (cm)	Facility for vertical movement
Kolkata	Ultadanga	Mixed	3.1	41	2.40	26	20	Stairway
	Lake Town	Residential	2.9	19	2.90	30	15	Stairway, ramp
	Sealdah	Mixed	2.4	53	2.10	30	15	Stairway
Bengaluru	Tin Factory Bus Stand	Commercial	2.6	50	2.09	25	12	Stairway
	Yeshwantpur Railway Station	PTT	3.1	54	1.10	26	9	Stairway, lift
	Marathahalli	Commercial	3.4	44	1.50	32	16	Stairway, lift
	Christ University	Educational	2.8	39	3.10	29	14	Stairway, lift

details of the locations considered for the study. The locations were chosen which had sufficient flow of pedestrians and were connected through a single entry and single exit. The data were collected between February and March 2017 during morning and evening peak hours on weekdays.

3.2 Questionnaire Survey

An interviewer-administered questionnaire survey was conducted at each location for a single weekday during morning and evening peak periods. The participants were randomly selected, and those who were willing to undertake the interview were finally interviewed. Due to heavy rush during the peak period, the participation rate was low (1 in 20 participants). Among all the requested participants, only 263 pedestrians participated in the final survey.

A five-point Likert scale (ranging from very poor to very good) was used to rank the perception of the pedestrians based on current existing condition of the FOBs. The following factors affecting the usability of the FOBs were considered in the study:

- Width (available effective walkway width both over the FOB and on stairways),
- Surface (in terms of broken/slippery tiles or well-maintained surface),
- End connectivity (in form of entry/exits along with type of vertical connectivity available),
- Safety and security (in terms of CCTV cameras and security personnel),
- Comfort (in the form of lighting and shade),

- Walk environment (surrounding existing conditions in the form of maintenance and cleanliness) and
- Obstruction (in form of hawkers and standing pedestrians).

All the above factors were considered after reviewing the existing literatures and finally selecting the best possible combinations for determining the current condition of the FOBs. The respondents were also asked about their user preference in using the FOBs under the present situation. Finally, the participants were also asked about the improvements they wanted to see in the existing condition which would improve the future usability.

4 Results and Model Development

The section discusses about the data extraction, analysis and model development.

4.1 Data Extraction and Analysis

The raw data collected from the field survey were manually entered into the excel sheets, according to specific requirements of final analysis. After that, data were analysed using IBM SPSS (Statistical Package for Social Sciences). Cronbach's Alpha (α) test was conducted to check internal consistency or reliability of the Likert scale. The results of the test are shown in Table 2. It can be seen that Cronbach's Alpha (α) was 0.758, which indicated a high level of internal consistency of the scale.

Also, Table 3 discusses about the "Cronbach's Alpha if Item Deleted". The value represents the importance of Cronbach's Alpha if any particular item was deleted. It could be observed that the removal of any factor from the study would result in lower Cronbach's Alpha.

The demographic data of the pedestrians who participated in the questionnaire survey are shown in Table 4. Also, Figs. 1, 2 and 3 represent the city-wise use of FOBs based on the profession, trip purpose and daily use.

From Table 3, it could be seen that majority of the respondents' who used the FOBs in both the cities were male pedestrians (~69–76%) and aged between 23 and 45 years (~40–42%). Moreover, the number of pedestrians with luggage or without luggage was quite similar.

Table 2 Reliability statistics

Cronbach's alpha	Cronbach's alpha based on standardized items	Number of items
0.758	0.737	7

Table 3 Cronbach's alpha value depending on different factors (item-total statistics)

Factors	Scale mean if item deleted	Scale variance if item deleted	Corrected item-total correlation	Squared multiple correlation	Cronbach's alpha if item deleted
Width	10.6383	14.904	0.388	0.166	0.708
Surface	11.0426	13.655	0.484	0.260	0.685
Obstruction	11.6454	14.559	0.579	0.351	0.672
Connectivity	10.9929	14.579	0.351	0.145	0.709
Security	11.4326	15.133	0.286	0.115	0.704
Comfort	11.2128	13.897	0.528	0.299	0.675
Walk environment	11.2482	13.688	0.523	0.310	0.675

Table 4 Demographic characteristics of respondents'

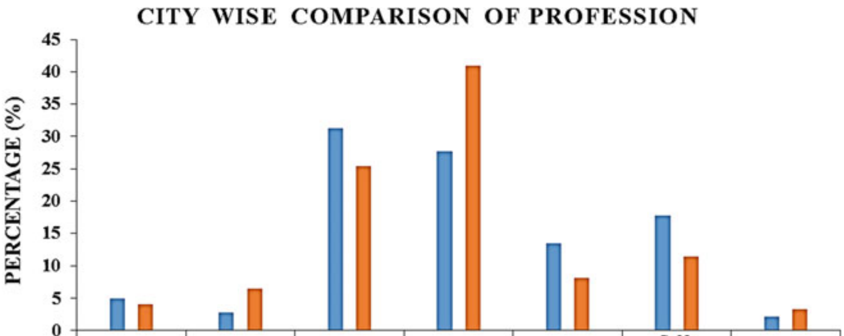
Characteristics		City	
		Bengaluru	Kolkata
Gender (%)	Male	69.5	75.4
	Female	30.5	24.6
Age (%)	<12	2.10	3.30
	13–22	27.0	19.7
	23–45	40.4	41.8
	46–59	22.0	25.4
	>60	8.50	9.80
Luggage (%)	With	45.4	54.9
	Without	54.6	45.1
Sample size		141	122

The pedestrians were mostly either students or servicemen who mostly used the FOB. There was a significant percentage of self-employed and businessmen who also used the facility (Fig. 1).

In both the cities, most of the pedestrians used the facility for travelling to their workplace (~34–36%) or colleges and schools (~17–24%). Some even used the facility for mode change and returning home as well (Fig. 2).

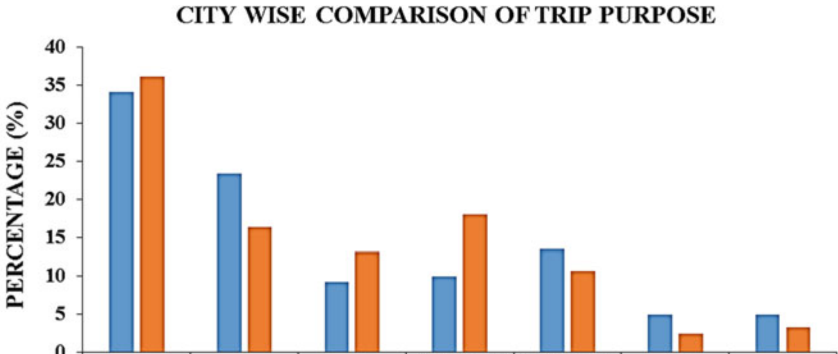
The results from Fig. 3 revealed that majority of the pedestrians using the FOBs used it daily twice (~32–45%) or more than daily twice daily (~13–16%). There were very few first-timers who were using the FOBs. Thus, it could be predicted that the perception of the respondents was mainly from those who were regular users of the FOBs and were aware about the existing condition of the FOBs.

The following Tables 5 and 6 show the descriptive statistics of the pedestrian perception in Bengaluru and Kolkata.



	Retired	Homemaker	Student	Service	Business	Self-employed	Others
Bengaluru	5.0	2.8	31.2	27.7	13.5	17.7	2.1
Kolkata	4.1	6.6	25.4	41.0	8.2	11.5	3.3

Fig. 1 Profession of respondents'



	Work	Education	Shopping	Change of mode	Returning home	Jay walker	Others
Bengaluru	34.0	23.4	9.2	9.9	13.5	5.0	5.0
Kolkata	36.1	16.4	13.1	18.0	10.7	2.5	3.3

Fig. 2 Trip purpose of respondents'

The descriptive statistics show the overall mean, standard deviation, variance, skewness and kurtosis of the factors primarily considered to define the perception of the pedestrians. The skewness measures the degree and direction of asymmetry, while the kurtosis measures the heaviness of tails of a distribution. From the data, it could be observed that the mean and standard deviation ranged between 0.8–2.39 and 0.6–1.07, respectively, for both the cities. Majority of the factors for both the cities were negatively skewed, indicating that the distribution was skewed to the left, which meant that the mean was less than the median. Similarly, in case of kurtosis,

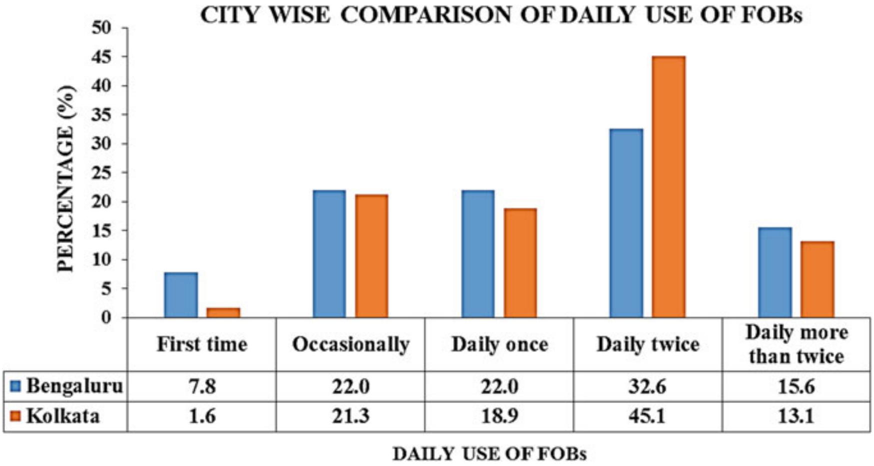


Fig. 3 Usage daily of respondents'

Table 5 Descriptive statistics of pedestrian perception in Bengaluru

Factors	Sample size	Mean	Std. deviation	Variance	Skewness		Kurtosis	
					Statistic	Std. error	Statistic	Std. error
Width	141	2.39	0.95	0.91	−0.03	0.20	−0.54	0.41
Surface	141	1.99	1.07	1.16	−0.19	0.20	−0.59	0.41
Obstruction	141	1.39	0.79	0.62	−0.82	0.20	−0.91	0.41
Connectivity	141	2.04	1.08	1.18	0.15	0.20	−0.62	0.41
Security	141	1.60	1.07	1.15	0.18	0.20	−0.55	0.41
Comfort	141	1.82	0.97	0.94	−0.01	0.20	−0.74	0.41
Walk environment	141	1.78	1.02	1.04	−0.05	0.20	−0.52	0.41

Table 6 Descriptive statistics of pedestrian perception in Kolkata

	Sample size	Mean	Std. deviation	Variance	Skewness		Kurtosis	
					Statistic	Std. error	Statistic	Std. error
Width	122	1.92	0.93	0.87	0.04	0.22	−0.51	0.43
Surface	122	1.86	0.92	0.85	−0.04	0.22	−0.39	0.43
Obstruction	122	0.86	0.64	0.41	0.12	0.22	−0.58	0.43
Connectivity	122	2.19	0.96	0.92	−0.12	0.22	−0.45	0.43
Security	122	1.39	1.07	1.15	0.14	0.22	−1.22	0.43
Comfort	122	1.74	0.99	0.98	0.04	0.22	−0.42	0.43
Walk environment	122	1.61	0.97	0.95	−0.12	0.22	−0.73	0.43

Table 7 Preferability sample size

City	Preferability sample size of using the FOBs under current existing conditions					
	Always (0)	Highly (1)	Sometimes (2)	Slightly (3)	Not (4)	Total
Bengaluru	69	53	15	4	0	141
Kolkata	33	41	17	15	16	122

all the factors had negative values, indicating lighter tails in comparison with the normal distribution.

The preference exhibited by the pedestrians to use the FOBs under the current existing conditions was ranked from "always preferable" to "not preferable". In Bengaluru (refer Table 7), none of the participants had responded that they did not want to use the FOBs. However, in Kolkata, there were a significant number of pedestrians who preferred either not to use the FOBs or were not slightly interested in using the FOBs.

Based on the preferability of existing conditions, Tables 8 and 9 represent the significance of factors which influence the pedestrians' perception. Multinomial linear regression (MLR) method was used as there was one outcome (preferability) variable and multiple predictors (width, surface, obstruction, connectivity, security, comfort and walk environment) were used.

The parameter estimates for preferability of the pedestrians in the two cities are based on the *B* (estimated coefficient), Wald statistics, odds ratio [(Exp *B*)] and *p*-values (significance). The estimated MLR coefficients for the models are represented by the "*B*" values. "Wald" is the chi-square test value which tests the null hypothesis that the estimate equals 0. Moreover, the odds ratio of the predictors is represented by "Exp (*B*)", which is the exponentiation of the coefficients. The risk of the outcome falling in the comparison group relative to the risk of the outcome falling in the referent group is represented by the odds ratio greater than 1, and it increases as the variable increases. The *p*-values of the coefficients are represented by "Sig", which is based on Wald test statistics of predictors.

It could be seen from Table 8 that in Bengaluru based on preferability "always" (code 1), the effect of surface was significant ($B = -1.587$, Wald $= 2.462$, $p = 0.097$) at 10% significance level, and the coefficient of B being negative indicated that better the surface of the FOB would be, the preferability would increase considerably. Similarly, for "always" preferability, the width was not significant as the *B*; Wald and *p*-values were lower at 90% confidence interval. Moreover, security along with surface was the most significant factors ($p < 0.1$) which were influencing the choice of the pedestrians through all the preferability choices.

Table 9 predicted that in Kolkata, security along with width and comfort was the most significant factors ($p < 0.1$) which influenced the choice of the pedestrians through all the preferability choices.

Table 8 Parameter estimates based on preferability of pedestrians in Bengaluru

Preferability[a]		B	Std. error	Wald	df	Sig.	Exp (B)	90% Confidence interval for Exp (B)	
								Lower bound	Upper bound
1.00 (ALWAYS)	**Intercept**	3.044	2.685	1.285	1	0.257			
	Width	0.525	0.823	0.407	1	0.524	1.690	0.437	6.538
	Surface	**−1.587**	1.011	**2.462**	1	**0.097**	0.205	0.039	1.080
	Obstruction	−1.237	1.172	1.114	1	0.291	0.290	0.042	1.995
	Connectivity	0.564	0.663	0.724	1	0.395	1.757	0.591	5.228
	Security	**1.534**	0.852	**3.243**	1	**0.072**	4.637	1.142	18.825
	Comfort	0.779	0.727	1.149	1	0.284	2.179	0.659	7.203
	Walk environment	0.129	0.752	0.030	1	0.863	1.138	0.330	3.922
2.00 (HIGHLY)	**Intercept**	3.955	2.690	2.161	1	0.142			
	Width	0.398	0.826	0.232	1	0.630	1.489	0.382	5.795
	Surface	**−1.809**	1.018	**3.157**	1	**0.076**	0.164	0.031	0.874
	Obstruction	−1.695	1.175	2.082	1	0.149	0.184	0.027	1.268
	Connectivity	0.715	0.670	1.136	1	0.287	2.043	0.678	6.156
	Security	**1.397**	0.856	**2.661**	1	**0.093**	4.041	0.988	16.523
	Comfort	0.466	0.733	0.404	1	0.525	1.593	0.477	5.317
	Walk environment	0.497	0.761	0.426	1	0.514	1.643	0.470	5.749
3.00 (SOMETIMES)	**Intercept**	0.357	2.894	0.015	1	0.902			
	Width	0.972	0.864	1.265	1	0.261	2.643	0.638	10.951
	Surface	−0.706	1.052	0.450	1	0.102	0.494	0.087	2.787
	Obstruction	**−1.016**	1.221	**1.392**	1	**0.095**	0.362	0.049	2.697

(continued)

Table 8 (continued)

Preferability[a]		B	Std. error	Wald	df	Sig.	Exp (B)	90% Confidence interval for Exp (B)	
								Lower bound	Upper bound
	Connectivity	0.103	0.706	0.021	1	0.884	1.109	0.347	3.539
	Security	**1.209**	0.886	**1.862**	1	**0.082**	3.350	0.780	14.386
	Comfort	−0.333	0.780	0.182	1	0.669	0.717	0.199	2.585
	Walk environment	0.629	0.787	0.637	1	0.425	1.875	.513	6.848

[a]The reference category is **4.00 (SLIGHTLY)**

Based on the B, Wald and p-value at 10% significance level, the most significant parameters were identified which influenced the choice of pedestrians through all the preferability choices. Thus these values were marked in bold against the most significant parameters

Table 9 Parameter estimates based on preferability of pedestrians in Kolkata

Preferability[a]		B	Std. error	Wald	df	Sig.	Exp (B)	90% Confidence interval for Exp (B)	
								Lower Bound	**Upper Bound**
1.00 (ALWAYS)	**Intercept**	−3.140	1.411	4.952	1	0.026			
	Width	0.125	0.843	0.022	1	0.882	1.133	0.217	5.913
	Surface	**−1.556**	0.920	**2.859**	1	**0.091**	0.211	0.035	1.281
	Obstruction	0.696	0.814	0.732	1	0.392	2.006	0.407	9.886
	Connectivity	−0.440	0.529	0.693	1	0.405	0.644	0.228	1.816
	Security	**3.324**	0.811	**16.795**	1	**0.000**	27.761	5.664	136.071
	Comfort	**1.139**	0.807	**1.994**	1	**0.088**	3.123	0.643	15.173
	Walk environment	0.505	0.711	0.505	1	0.477	1.658	0.411	6.684
2.00 (HIGHLY)	**Intercept**	−0.866	1.071	0.653	1	0.419			
	Width	0.115	0.703	0.027	1	0.870	1.122	.283	4.454
	Surface	−0.091	0.780	0.014	1	0.907	0.913	0.198	4.211
	Obstruction	−0.004	0.684	0.000	1	0.995	0.996	0.261	3.807
	Connectivity	−0.369	0.449	0.674	1	0.412	0.692	0.287	1.668
	Security	**2.172**	0.700	**9.626**	1	**0.002**	8.772	2.225	34.584
	Comfort	0.112	0.633	0.031	1	0.860	1.118	0.323	3.868
	Walk environment	0.584	0.579	1.019	1	0.313	1.794	0.577	5.579
3.00 (SOMETIMES)	**Intercept**	−1.088	1.162	0.875	1	0.349			
	Width	**1.298**	0.817	**2.528**	1	**0.092**	3.663	0.739	18.152
	Surface	−1.328	0.905	2.155	1	0.142	0.265	0.045	1.560
	Obstruction	0.933	0.731	1.629	1	0.202	2.542	0.607	10.651

(continued)

Table 9 (continued)

Preferability[a]		B	Std. error	Wald	df	Sig.	Exp (B)	90% Confidence interval for Exp (B)	
								Lower Bound	**Upper Bound**
	Connectivity	−0.652	0.493	1.748	1	0.186	0.521	0.198	1.369
	Security	**1.072**	0.762	**1.980**	1	**0.059**	2.921	0.656	12.996
	Comfort	**1.257**	0.700	**3.228**	1	**0.072**	3.516	0.892	13.860
	Walk environment	−0.576	0.614	0.881	1	0.348	0.562	0.169	1.872
4.00 (SLIGHTLY)	**Intercept**	−1.624	1.202	1.824	1	0.177			
	Width	**1.168**	0.778	**2.257**	1	**0.093**	3.216	0.701	14.760
	Surface	−0.969	0.835	1.348	1	0.246	0.379	0.074	1.948
	Obstruction	**0.099**	0.741	**0.018**	1	**0.094**	1.104	0.258	4.714
	Connectivity	0.083	0.477	0.030	1	0.862	1.086	0.427	2.766
	Security	**0.789**	0.761	**1.074**	1	**0.070**	2.201	0.495	9.790
	Comfort	0.443	0.678	0.427	1	0.513	1.557	0.413	5.877
	Walk environment	−0.027	0.615	0.002	1	0.964	0.973	0.291	3.250

[a]The reference category is **5.00 (NOT)**

Based on the B, Wald and p-value at 10% significance level, the most significant parameters were identified which influenced the choice of pedestrians through all the preferability choices. Thus these values were marked in bold against the most significant parameters

4.2 Model Formulation

Using the parameter estimates from Tables 8 to 9, the final models were developed both the cities based on preferability choice. The method of least squares which leads to best fitting of a set of data is used to form regression models between the dependent variable (Y_i) and independent variables (X_i). The multiple linear relationships between dependent and independent variables usually take the form of Eq. 1.

$$Y_i = \beta_0 + \beta_1 * X_1 + \beta_2 * X_2 + \beta_3 * X_3 + \cdots + \beta_n * X_n \quad (1)$$

During the formulation of the final models, the significant parameters were only considered, while the insignificant factors which could also have an impact on the model were not considered in the final models. The finals models are represented in Table 11 based on different preferabilities.

4.3 Future Usability

The current preferability from Table 10 showed the primary factors which the pedestrians felt affecting their perception towards using the FOBs. Further, a set of questions were put up to the pedestrians, and they were asked whether any improvement (Yes/No) in the following scenarios was necessary or not, which would change their perception towards using the FOBs in future. The six different categories are listed in Table 11.

The result from Table 11 shows that in both the cities, installation or maintenance of lift/escalator/ramp is the most important category which most of the pedestrians felt that needed immediate improvement. Apart from that, security installation or improvement is also extremely important to the majority of the pedestrians. Connectivity improvement and maintenance improvement were not important. In Kolkata

Table 10 A final model based on the preferability of the respondents'

City	Preferability	Final models
Bengaluru	Always	$Y_{\text{always}} = 3.044 - 1.587 * X_{\text{Surface}} + 1.534 * X_{\text{Security}}$
	Highly	$Y_{\text{highly}} = 3.955 - 1.809 * X_{\text{Surface}} + 1.397 * X_{\text{Security}}$
	Sometimes	$Y_{\text{sometimes}} = 0.357 - 1.016 * X_{\text{Obstruction}} + 1.209 * X_{\text{Security}}$
Kolkata	Always	$Y_{\text{always}} = 3.140 - 1.556 * X_{\text{Surface}} + 3.324 * X_{\text{Security}} + 1.139 * X_{\text{Comfort}}$
	Highly	$Y_{\text{highly}} = -0.866 + 2.172 * X_{\text{Security}}$
	Sometimes	$Y_{\text{sometimes}} = -1.088 + 1.298 * X_{\text{Width}} + 1.072 * X_{\text{Security}} + 1.257 * X_{\text{Comfort}}$
	Slightly	$Y_{\text{slightly}} = -1.624 + 1.168 * X_{\text{Width}} + 0.099 * X_{\text{Obstruction}} + 0.789 * X_{\text{Security}}$

Table 11 Choice of improvement requirement

Category	Choice	Bengaluru	Kolkata
Connectivity improvement	Yes	20	25
	No	121	97
Lift/escalator/ramp installation and maintenance	Yes	82	106
	No	59	16
Maintenance improvement	Yes	39	61
	No	102	61
Security installation or improvement	Yes	56	66
	No	85	56
Redesigning required	Yes	29	83
	No	112	39
Obstruction removal/relocation	Yes	40	66
	No	101	56

however, the pedestrians felt that redesigning of the facility was of utmost importance and the obstruction in the form of hawkers/ vendors/ standing pedestrians were also needed to be removed.

5 Conclusions

The current study focussed to understand the perception of the pedestrians towards using the FOBs under their existing conditions. Moreover, the improvements required in the current scenarios which could enhance the future usability were also explored. Thus, from the collected data and analysis, the following conclusions could be drawn:

- The respondents who participated in the survey used the FOBs, two (~32–45%) or more times daily (~13–16%), and were aware about the existing current conditions.
- Majority of the pedestrians using the FOBs in both the cities were male pedestrians (~70–75%) and aged between 23 and 45 years (~40–42%).
- The pedestrians who were either servicemen (~28–41%) or students (~25–32%) used the FOBs more often.
- Most of the pedestrians used the FOBs daily for travelling to their workplace (~34–36%) or educational institutes (~17–24%). Some even used the FOBs for mode change (~10–18%) and returning home (~10–14%).
- From the descriptive statistics, it could be seen that the mean and standard deviation for the pedestrian perception rating in Bengaluru ranged between 1.4–2.39 and 0.79–1.08, respectively. Similarly, for Kolkata, the ratings for mean and standard deviation varied between 0.86–2.19 and 0.64–1.07.

- Also, from the descriptive statistics, it was seen that the majority of the factors for both the cities were negatively skewed, indicating that the distribution was skewed to the left, which meant that the mean was less than the median. Similarly, in case of kurtosis, all the factors had negative values, indicating lighter tails in comparison with the normal distribution.
- As per preferability sample size, most of the pedestrians in Bengaluru (48.9%) preferred to “always” use the FOBs, while in Kolkata 33.6%, pedestrians said they would “highly” use the FOBs.
- The parameter estimates using multinomial logistic regression for Bengaluru showed that surface and security were the most significant factors which affected the preferability of the pedestrians of using the FOBs.
- In Kolkata, the preferability of using the FOBs mainly depended on security, width and comfort parameters.
- The installation or improvement of security along with installation or maintenance of lift/escalator/ramp was the most important parameters which required immediate improvement and which the pedestrians felt would significantly improve the future usability.

6 Recommendations

The following recommendations are required to improve the usability of the FOBs in the two metropolitan cities of India and encourage pedestrians to use the FOBs:

- Installing CCTV surveillance cameras or placing security personnel over the FOBs.
- Installing and maintaining lifts and escalators where required.
- Removal or relocation of vendors/hawkers/standing pedestrians from the FOB walkways.
- Proper maintenance of FOBs in form of repairing the surface, along with replacing lights and providing proper shade.

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Safety Evaluation at Urban Intersections Using Surrogate Measures

Aravindkumar Tigari, Satbir Singh Puwar, A. Mohan Rao, and S. Velmurugan

Abstract This paper aims to clarify the concept of surrogate measures of safety, summarizes the past research in the area and to investigate how simulation can be used in assessing the safety of intersections and midblock on the selected Project Corridor. In this context, four-lane divided carriageway connecting Gurugram with Faridabad and thus spanning a length of km 24.3 has been selected as the study stretch. For the above road corridor, traffic safety measures have been carried out based on the observed road crash data using different types of statistical approaches, mainly before–after comparisons of observed data, and forecasting studies based on road safety audits. This is primarily carried out on the assumption that there is direct correlation between the degree of safety on a road and the number of road crashes that occur there. Even though road crash data is a true representation of safety, its use in safety studies has many limitations. There are many other techniques which can be used for traffic safety evaluation in advance before the accidents occurs. These techniques are called traffic surrogate models. The word 'surrogate' means 'substitute' or 'replacement' which implies that by using surrogate measures to determine traffic safety as another factor to represent traffic safety. The surrogate measure model has been attempted on Gurugram–Faridabad road which is one of the major links between the two cities. It is regularly used by thousands of motorists commuting between Gurgaon and Faridabad. About 279 road crashes were reported on the study stretch during the last seven years which accounted for 165 road deaths and 250 persons injured.

Keywords Traffic safety · Surrogate safety assessment model · Time to collision · Post-Encroachment time

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1 Introduction

Road traffic crashes are one of the world's largest public health and injury prevention problems. The problem is all the more acute because the victims are overwhelmingly healthy before their crashes. According to the World Health Organization (WHO), more than 1 million people are killed on the world's roads each year [3]. A report published by the WHO in 2004 estimated that some 1.2 million people were killed and 50 million injured in traffic collisions on the roads around the world each year and was the leading cause of death among children 10–19 years of age. Hence, traffic safety has become a major area of concern for the authorities. Most of traditional analyses of traffic safety measures are carried out based on the observed road crash data, using different types of statistical approaches; mainly before and after comparison of observed data and anticipatory estimation studies based on safety audits. This is done mainly on the premise that there is direct correlation between the degree of safety on a road and the number of road crashes that occur there.

Road safety is a major concern in the developing world because of its impact on the global economy and people's welfare. Due to the rising population, the traffic risk has increased, especially in developing nations like India as the infrastructure is unable to cope with the increasing traffic. In India, the traffic situation is very grim as the rate of fatality is increasing every year. If one makes a close look at the Report 'Road Accidents in India-2016' published by Ministry of Road Transport and Highways GOI, it is observed that during the calendar year 2016, the total number of road crashes is reported at 4,80,652 causing injuries to 4,94,624 persons and claiming 1,50,785 lives in the country. This would translate, on an average, into 1317 road crashes and 413 road deaths taking place on Indian roads every day, or 55 road crashes and 17 deaths every hour.

Traffic safety assessment is generally based on the use of historical accident data records, which are reactive in nature; it is like waiting for any road crash to occur and then applying their countermeasures. These days researchers have proposed a new method for the appraisal of collisions at signalized and non-signalized intersections called proactive model based on the surrogate safety measures (SSMs). The main advantage of this method is that it can help predict the frequency of an impending road crash due to poor geometry of any road due to the aforesaid conditions and thereby serving as an efficient and more reliable proximal measure of traffic safety.

The word 'surrogate' means 'substitute' or 'replacement'. Hence, by using surrogate measures to determine traffic safety, we mean to substitute the need for crash data with another factor which would represent traffic safety. In the area of road safety engineering, expected outcome is the reduction or elimination of target crashes. (Jaehyun et al. 2014) Reduction of the frequency of events necessary for these crashes to happen is an appealing surrogate outcome that meets the first condition set forth in the medical sciences (Shekhar Babu and Vedagiri 2016). Thus, the frequency and other characteristics of such surrogate events may be considered promising surrogate measures of safety (Jennifer and McDonald 2011) As such, safety may also be affected by factors that are external to the surrogate measures. For example, traffic events reminiscent of crashes are believed to share many factors with crashes (Killi and Vedagiri 2014).

1.1 Objectives

This study mainly concentrated on the use of both traffic simulation model VISSIM and Surrogate safety assessment model (SSAM) to better understand causes of traffic crashes and further test to enhance the safety of the public. Specifically, this corridor selected for the analysis is the four-lane divided Gurugram–Faridabad road stretch spanning a length of km 24.3. Basically, this is a tolled interurban highway which is considered for the safety evaluation. However, the scope of the study confined to the safety evaluation of typical urban unsignalized intersections and midblock using surrogate safety assessment models.

2 Methodology

The methodology is adopted for the study based on the reviewed literature and the objectives identified. The road crash data for the study stretch were collected from the police stations. The study stretches fall under three police stations, namely DLF Phase I Gurugram, Surajkund, and Sector 55 Faridabad. The road crash data were collected from the year 2008 to 2014. The identified black spots on the Project Corridor were at km 8.05, *i.e. Valley View Apartment, an unsignalized intersection*; km 9.10, *i.e. Gawal Pahari unsignalized intersection;* and km 23.5, i.e. Hanuman Murti midblock. The above locations were simulated in VISSIM 7, and calibration was done by appropriately modifying the driver behaviour parameters (Pirdavani et al. 2010). The methodology adopted for calibration of VISSIM is presented in Fig. 1.

3 Study Corridor

The four-lane divided carriageway connecting Gurugram with Faridabad and thus spanning a length of km 24.3 has been selected as the study stretch. About 279 road crashes were reported in the study corridor during the last seven years resulting in the death of 165 persons and injury to 250 persons. The identified black spots on the Project Corridor were at km 8.05, *i.e. Valley View Apartment, an unsignalized intersection*; km 9.10, *i.e. Gawal Pahari unsignalized intersection*; and km 23.5, i.e. Hanuman Murti midblock. Out of the above three crash-prone locations, crash data analysis and other associated studies carried out pertaining to km 8.05 and km 23.5 are presented in this paper.

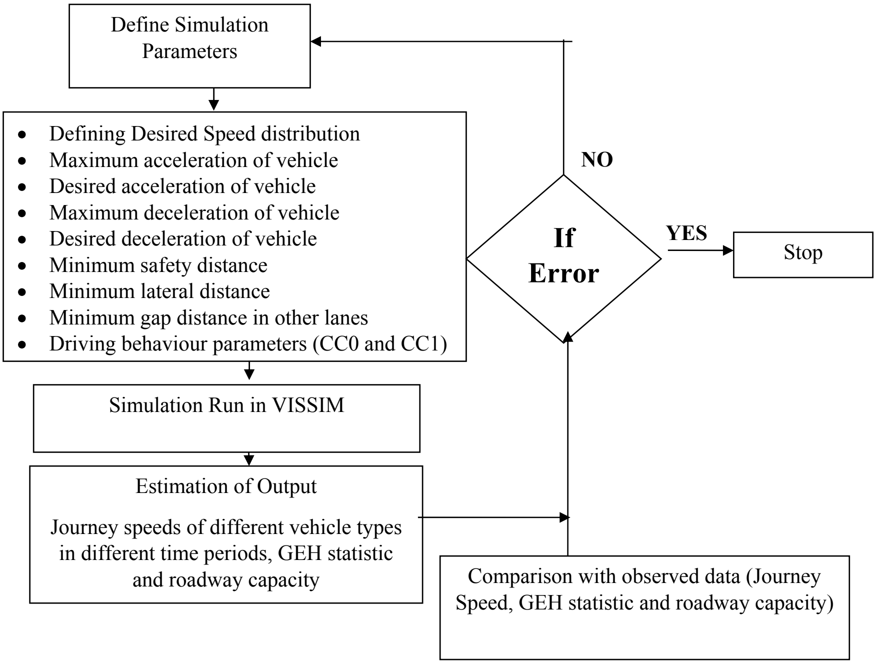


Fig. 1 Study methodology for calibration of model

4 Data Collection

Road crash data were collected for the years 2008–2014 from the respective police stations for km 8.05, i.e. Valley View Apartment unsignalized intersection and for Hanuman Murti midblock located at km 23.50. Videography method of data collection was adopted, and the traffic was recorded for Valley View Apartment which is an unsignalized intersection located at km 8.05 from Gurugram. Classified traffic volume data at km 8.05 intersection were collected by using videographic method spanning for 2 h from 9.15 am to 11.15 am, whereas at km 23.50 data were collected from 9.40 am to 11.40 am in order to get the data on traffic volume. Speed data were collected by using laser gun, 100 metres away from the intersection (Sorate et al. 2015). Figure 2 depicts the data collection arrangement and the snapshot of recorded video.

5 Data Extraction

The classified vehicle volumes of the intersections were extracted manually from the video at an interval of 5 min. Tables 1 and 2 present the sample of the classified traffic

Fig. 2 Videographic data collection and speed data collection using radar gun

Table 1 Sample of classified turning flow handled at km 8.05, unsignalized intersection near valley view apartment

Direction	Car	Motorized two-wheelers	Auto	Bus	LCV	HCV	MAV	Others	Total	Relative flow
Straight	6246	1432	173	79	438	67	0	14	8449	0.981
Left	125	26	2	0	0	6	0	0	159	0.018
Total	6371	1458	175	79	438	73	0	14	8608	

volume collected at Valley View Apartment intersection (at km 8.05) and Hanuman Murti (at km 23.50) respectively.

6 Traffic Flow Characteristics

The traffic flow at km 8.05 and at km 23.50 is presented in Tables 1 and 2, respectively. The composition of vehicles observed at km 8.05 intersection is shown in Fig. 3, and it is evident that cars followed by two-wheelers dominate the flow, accounting for 74% and 17%, respectively.

The cumulative plot of speed of the vehicles is shown in Fig. 4. The speed of cars was maximum and truck at low speed, whereas in two-wheeler and bus, the speed varied from 30 to 100 kmph.

Table 2 Classified traffic volume at km 23.05 midblock section, near Hanuman Murti

Direction	Car	Motorized two-wheelers	Auto	Bus	LCV	HCV	MAV	Others	Total	Relative flow
Gurugram to Faridabad	1238	563	53	33	184	57	3	13	2144	1
Faridabad to Gurugram	1745	1114	49	47	276	42	0	13	3286	1

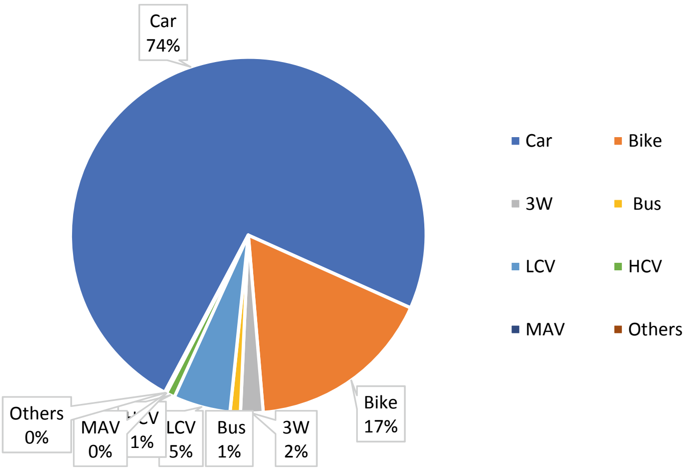


Fig. 3 Hourly traffic flow on Valley View Apartment at km 8.05

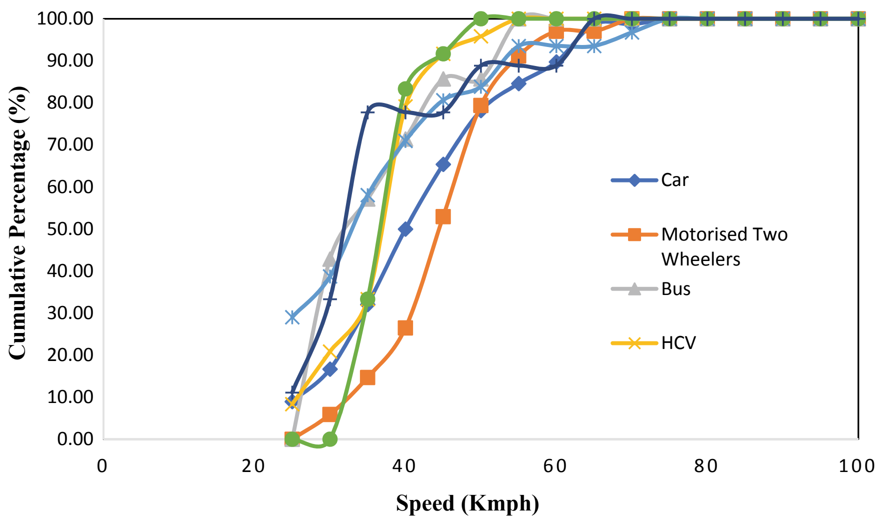


Fig. 4 Cumulative distribution of speed

7 Model Development and Testing

VISSIM and SSAM were the simulation tools utilized to simulate the conflicts between vehicles. The primary objective in this section is to determine whether VISSIM and SSAM models collectively provide reasonable results of the surrogate safety measures for vehicle-to-vehicle conflicts. It is not the intention of the model to validate or calibrate these two models but rather to substantiate the ability of the model to produce trends that are consistent with the increasing flow of vehicular and vehicles traffic.

7.1 VISSIM and SSAM Overview

VISSIM is a microscopic multi-modal traffic flow simulation software, traffic system operational analysis. VISSIM model consists of a set of cross-linked sub-models that depend on a number of parameters to describe traffic control operation, traffic flow characteristics, and drivers' behaviour.

VISSIM, version 7.00, was used in this study to develop simulation model for vehicles. A midblock was established in VISSIM to substantiate the model ability for simulating the conflicts between vehicles. The research methodology is first presented, followed by the analysis of VISSIM with regards to its various processes such as data collection, building of the road network, calibration, validation and more (Datta 2018). Finally, concluding is the assessment of the merits and demerits when using VISSIM for modelling. A hypothetical unsignalized and midblock crossing was simulated in VISSIM, as shown in Figs. 5 and 6.

Surrogate safety assessment model (SSAM) is a software application designed by the Federal Highway Administration (FHWA) to execute conflict analysis of vehicle trajectory data from microscopic traffic simulation models, such as VISSIM,

Fig. 5 Representation of intersection at Valley View Apartment km 8.05

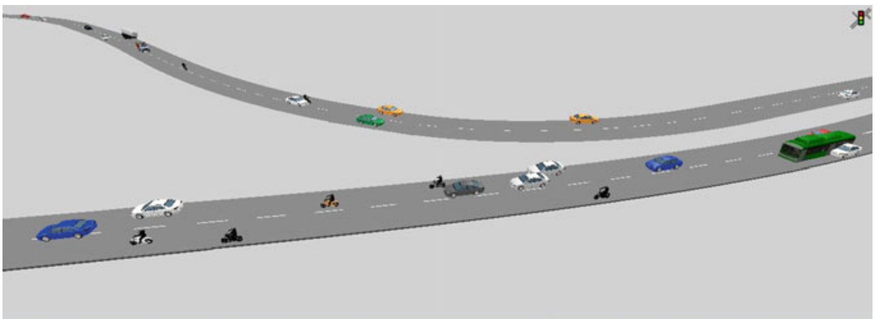


Fig. 6 Representation of mid-block at Hanuman Murti km 23.50

AIMSUN, PARAMICS, and TEXAS. It can provide a summary of the total number of conflicts broken down by type of conflicts (Ciro and Guida 2012). In SSAM, there are three types of conflicts: crossing, rear-end, and lane-changing. Classification of conflicts is based on the conflict angle, which these vehicles converge to a hypothetical collision point. The conflict angle is classified as follows:

- Crossing: $\|\text{conflict angle}\| > 85°$
- Rear-end: $\|\text{conflict angle}\| < 30°$
- Lane-changing: $30° \le \|\text{conflict angle}\| < 85°$.

SSAM is mainly used for vehicle-to-vehicle conflicts to assess the safety of traffic facilities, and it can detect the vehicle trajectory. SSAM can successfully import the vehicle trajectory file and detect the vehicle-to-vehicle conflicts. (Essa and Sayed 2015) SSAM software helped in the analysis of conflict by directly processing vehicle trajectory data which are collected from VISSIM output. It can provide a summary of the total number of conflicts broken down by type of conflicts. The analysis focused mainly on vehicle-to-vehicle conflicts and vehicle-to-vehicle rear-end conflicts caused by sudden braking to yield to vehicle at midblock crossings (Charly and Mathew 2014). The vehicle-to-vehicle crossing conflict and the vehicle-to-vehicle rear-end conflict can be selected from the SSAM output based on the length of the vehicle and the type of the conflicts. The numbers of vehicle-to-vehicle crossing conflicts and vehicle-to-vehicle rear-end conflicts were collected for each scenario. Furthermore, SSAM also calculates surrogate safety measures for each conflict. In this study, two measures were applied to evaluate the traffic safety: time-to-collision (TTC) and post-encroachment time (PET). TTC is defined as the value of time at the moment where one of the road users reacts and starts braking, renamed as 'time to accident' (TA) The shorter the TTC is, the more dangerous the situation is. The PET is defined as the difference between the time at which the leading vehicle enters a collision point and the time at which the following vehicle enters the same point.

8 Results and Analysis

The vehicle-to-vehicle conflict data were from SSAM when the TTC threshold is set as 5 s and PET threshold is set as 9.5 s. The statistical frequency distributions were developed covering TTC and PET, for business As usual (BAU) scenario at Valley View Apartment (unsignalized intersection), i.e. km 8.05 and Hanuman Murti (midblock), i.e. km 23.50.

Figure 7 shows that the TTC distribution for all conflicts observed at a single intersection, namely Valley View Apartment. It is evident from the above figure that for the design alternative of 'without gap in median' has the least number of conflicts and possessing highest mean TTC value, indicating thereby that the above alternative is the safest design. Based on the cumulative frequency distributions, it appears that there are inflection points at approximately TTC = 2.19 s.

	Existing Model	Junction Improvement	Without Median	U Turn
Total Conflicts	234487	205416	136041	239289
Mean TTC	1.94	1.98	2.19	2.11
Std.Dev.TTC	4.4	1.83	1.80	1.78
15th % TTC	0	0	0	0
50th % TTC	1.7	1.8	2.2	2.1
85th% TTC	4.4	4.4	4.4	4.3

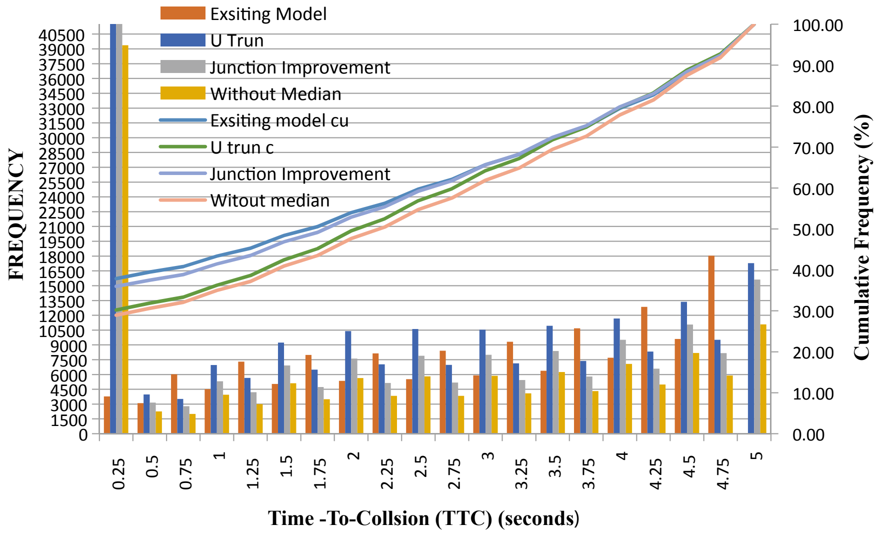


Fig. 7 TTC frequency distribution comparison for all design alternatives including BAU alternative (Valley View Apartment) at km 8.05

Figure 8 shows that the PET distribution of all conflicts is similar for all three design alternatives, making it difficult to infer as to which alternative is the safest design based on PET alone. While without gap in median design alternative has the least number of conflicts, that alternative also has the lowest mean PET value. Based on the cumulative frequency distribution, it appears that there are inflection points at approximately PET = 1.38 s. To investigate the relationship between TTC and PET, a simple linear regression model has been developed.

Figure 9 illustrates the relationship between PET and TTC. According to the above linear regression results, it is found that the p value of independent variable is 0.00, indicating that TTC is significantly associated with PET. In addition, the R^2 value for the model is 0.683 which implies that 68% of the variability in PET can be explained by the variation in TTC. It is noted that as the TTC increases, the PET also increases.

Figure 10 shows that the TTC distribution for all conflicts is similar in case of all design alternatives. Similarly, Fig. 11 shows the PET distribution of all conflicts is similar for all three design alternatives, making it difficult to conclude which alternative is the safest design based on PET parameter alone. At the same time, straight design alternative has the least number of conflicts as well as highest mean TTC value coupled with lowest mean PET value, indicating thereby that it is the

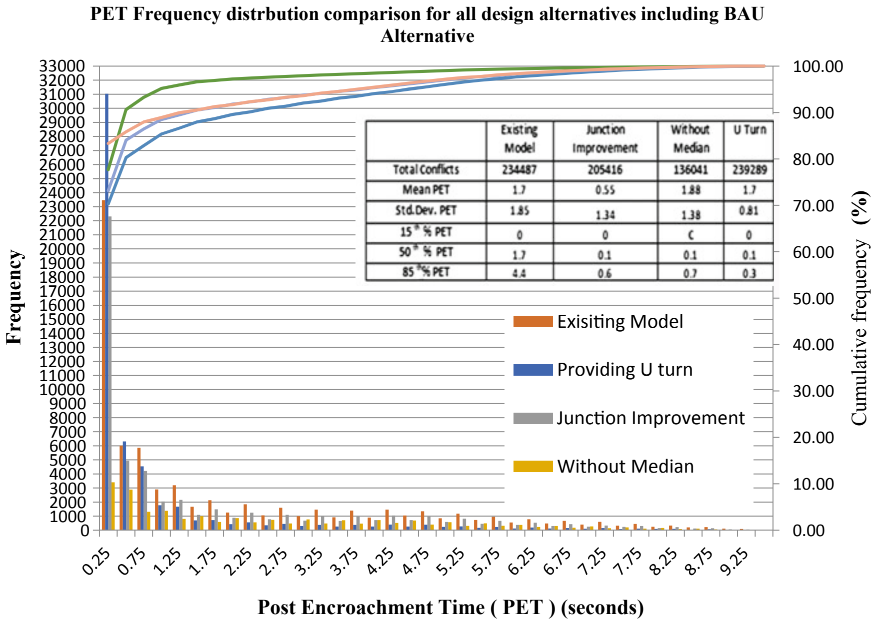


Fig. 8 PET Frequency distribution comparison for all design alternatives including BAU alternative at km 8.05, *i.e. Valley View Apartment*

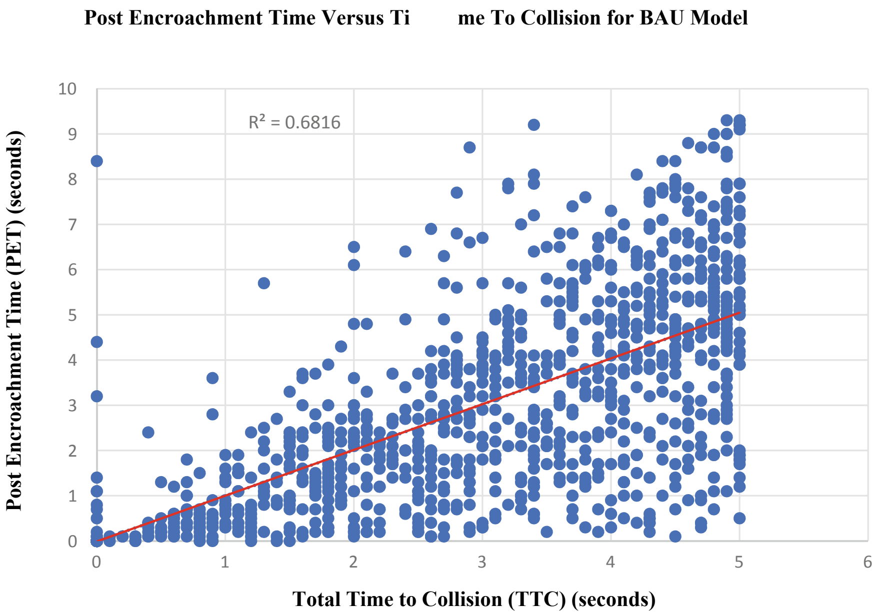


Fig. 9 PET versus TTC for BAU scenario

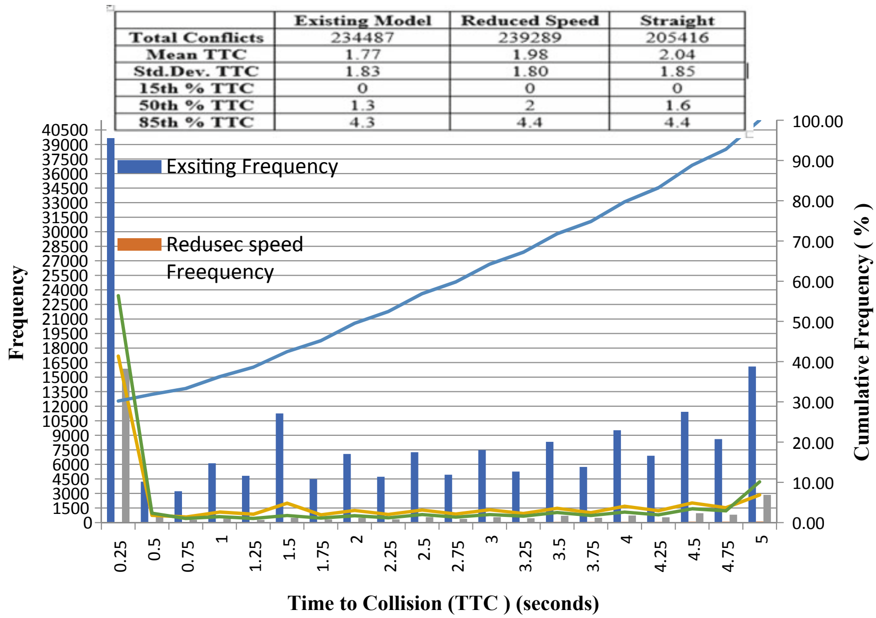


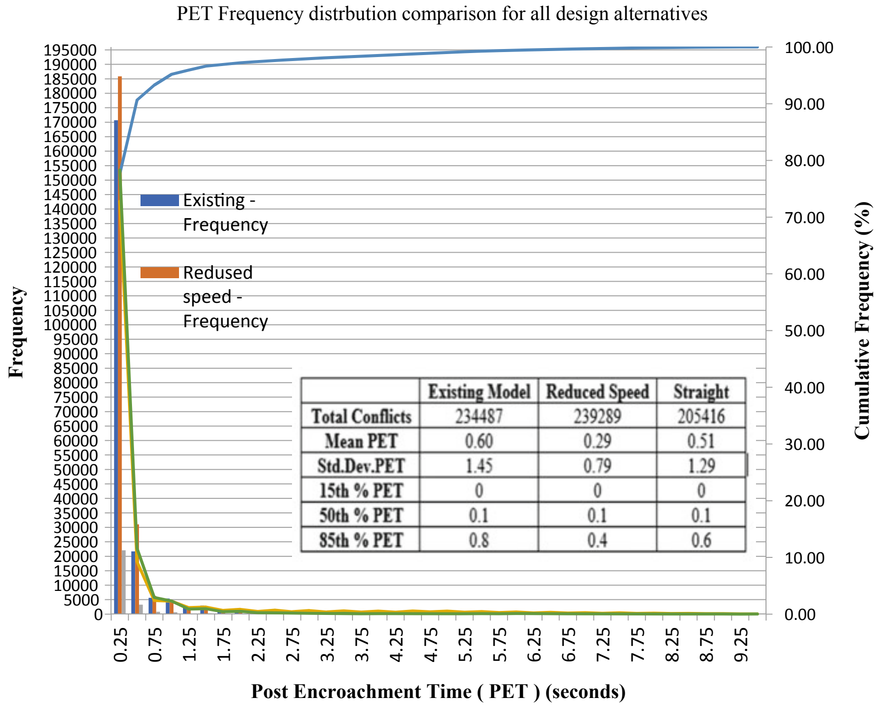
Fig. 10 TTC frequency distribution comparison for all design including BAU alternative at km 23.50 near Hanuman Murti

safest design. Based on the cumulative frequency distribution, it appears that there are inflection points at approximately TTC = 2.04 s.

9 Concluding Remarks

The main purpose of this study is towards the deployment of simulation models for understanding causes of vehicle-related traffic crashes and assess selected countermeasures to enhance the safety of the public. The appropriate simulation tool was then deployed to test the vehicle-to-vehicle conflicts. VISSIM and SSAM models developed in this paper were utilized to explore their abilities to provide reasonable results of surrogate safety measures. A virtual unsignalized intersection and midblock crossing was tested in VISSIM to simulate different levels of vehicular traffic volumes and spot speed. SSAM software was used to extract the surrogate safety measures for each conflict by directly processing vehicle trajectory data from VISSIM. The findings emerged from the study provide abundant evidence that VISSIM and SSAM models can be used to estimate vehicle-to-vehicle conflicts.

It was necessary to develop a calibrated and validated VISSIM model for the Project Corridor considered in this paper. The identified black spots on the Project Corridor were at km 8.05, *i.e. Valley View Apartment, an unsignalized intersection*;



	Existing Model	Reduced Speed	Straight
Total Conflicts	234487	239289	205416
Mean PET	0.60	0.29	0.51
Std.Dev.PET	1.45	0.79	1.29
15th % PET	0	0	0
50th % PET	0.1	0.1	0.1
85th % PET	0.8	0.4	0.6

Fig. 11 PET frequency distribution comparison for all design including BAU alternative at km 23.50 near Hanuman Murti

km 9.10, *i.e. Gawal Pahari unsignalized intersection*; and km 23.5, i.e. Hanuman Murti midblock. Out of the above three crash-prone locations, the crash data analysis and other associated studies carried pertaining to km 8.05 and km 23.5 are presented in this paper. Subsequently, SSAM was used to extract the vehicle-to-vehicle conflicts by processing the vehicle trajectory data from the calibrated and validated model.

A linear regression model was developed to identify whether the simulated conflicts are able to replicate the observed conflicts. Based on the model developed in the study, it was found that the number of simulated conflicts is significantly related to the number of observed conflicts.

By analysing the simulated conflict data, it was found that in the case of unsignalized intersection at km 8.05, i.e. Valley View Apartment, the mean of TTC is 2.19 s with a standard deviation of 0.80 and the mean PET is 1.88 s with a standard deviation of 0.38. Hence, the best engineering intervention conceived for the above unsignalized intersection is to develop intersection without gap in median, as we have less number of conflicts compared to other alternatives.

Similarly, for midblock at km 23.50, i.e. near Hanuman Murti, the mean of TTC is 2.04 s with a standard deviation of 0.85 and the mean PET is 1.51 s with a standard deviation of 0.29. Hence, the best engineering intervention considered for the above midblock is to remove the curvature at this very location and thus make the road straight which would reduce the potential for conflicts.

Lower TTC indicates higher probability of collision, and similarly lower PET indicates higher probability of collision. In summary, it can be inferred from this study that the TTC is ranging between 1.5 and 2.00 s which implies moderate risk of collision for the two locations considered in this study.

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Acceleration and Deceleration Behavior in Departing and Approaching Sections of Curve Using Naturalistic Driving Data

Suresh Nama, Gourab Sil, Akhilesh Kumar Maurya, and Avijit Maji

Abstract Driver safety is one of the prime concerns of traffic engineers in the design and construction of highways. Highway collision causes both social and economic losses. So the primary focus is always there in understanding the driver behavior over various geometric sections for the development of safe geometric roads. So in this study, an attempt was made to understand the acceleration and deceleration behavior over the horizontal curved sections. To do so a four-lane divided highway was considered and continuous data of the vehicle was collected using GPS techniques with the help of 30 distinct drivers. Each of these drivers drove around a stretch of 40 km with the attached GPS device to the vehicle. Using this data six distinct acceleration and deceleration regression-based models are developed corresponding to the cases of maximum, minimum and average acceleration and deceleration. From these models, great insight on how the geometric parameters are influencing this driver's acceleration and deceleration behavior is obtained. With these models, it is observed that radius and previous tangent length are the most influential parameters affecting driver behavior while approaching the curve. Radius and vertical gradient are found to be the highly correlated parameters with the acceleration behavior on the curve.

Keywords Acceleration · Deceleration · Four-Lane · Horizontal curve and modeling

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1 Introduction

One of the goals of transportation professionals is to provide the possible level of safety over the entire road network. Safety concerns on highways are increasing drastically. As per GHO (Global Health Observatory), an estimation of 1.25 million deaths occurred due to road accidents in 2013. In that, the share of India is about 16.66%. The transportation research wing of the Ministry of Road Transport and Highways (MORTH) stated that the accident severity has been raised from 21.1% in 2003 to 28.3% in 2013. Among these accidents, 89% (Mohan 2004) is due to human error. The accident data available at National Highway Traffic Safety Administration (NHTSA) says that the accidents on rural roads are higher than that of urban roads. It has been stated that over 50% of the total fatalities are attributed to the accidents in the curved section (Lamm et al. 2002). In addition to that, the accident rates observed on curves are 3–5 times higher than in straight sections.

Therefore, to understand the cause of accidents on horizontal curves, researchers have studied the driver's speed, acceleration, and deceleration behavior on horizontal curves. Researchers have developed operating speed models (V_{85}) to predict the free-flow speeds of the vehicle at critical sections on the highway such as tangent and curve (Banihashemi et al. 2011; Dimaiuta et al. 2011; Montella et al. 2015 and Sil et al. 2018). Many of the speed models are developed by collecting operating speeds of the vehicles mostly by using the radar gun (Montella et al. 2014). Many of the researchers studied on predicting operating speed models found that the geometric parameters such as radius, CCR, curve length, and tangent length are highly correlating with the operating speed of vehicles.

Additionally, to develop the speed profiles over the horizontal curve the acceleration/deceleration behavior of the drivers was also studied. But most of the speed models were developed (Psarianos et al. 1998; Jacob and Anjaneyulu 2013; Sil et al. 2018; Maji et al. 2018) considering the speed variations over the curve segment as constant (i.e. zero acceleration) and there will be constant acceleration/deceleration in the remaining part of the horizontal curve (tangent).

Lamm et al. (1999), Krammes et al. (1995), and Ottesen and Krammes (2000) considered acceleration/deceleration on horizontal curves on two-lane highways as 0.85 m/s^2 (constant) irrespective of the configuration of the curve. Hassan et al. (2003) collected speed data at five locations of each curve and measured acceleration and deceleration between the locations. The findings show that the speed of the vehicle is not the same throughout the curve and the vehicle is traveling at an acceleration/deceleration less than 0.85 m/s^2. Fitzpatrick and Collins (2000) estimated the typical values of acceleration on horizontal curves in the range of 0–0.54 m/s^2, similarly for deceleration are 0.00–1.00 m/s^2. Whereas Echaveguren et al. (2003) suggested the deceleration values as 0.15–0.65 m/s^2 on two-lane highway curves.

Using a driving simulator, Montella et al. (2014) have developed acceleration/deceleration models by collecting acceleration/deceleration data on two-lane roads at horizontal curves. The generated modes pointed out that the radius of the curve has a significant negative effect on acceleration and deceleration of the vehicle over the curve. This influence is high on deceleration compared to acceleration. Bella (2014) also used a driving simulator to collected data from horizontal curves. In this, the models for acceleration/deceleration rates are developed using the radius of the curve and the speed at the tangents. The models represent that the deceleration is directly related to the speed of the vehicle on the tangent section and inversely related to the radius of the curve. A similar relation between the deceleration and the radius is also observed by Pérez Zuriaga et al. (2010). Earlier studies on predicting acceleration/deceleration found that radius, CCR, tangent length, and deflection angle are more significant. Xu et al. (2017) stated that the deceleration rate is greater than the acceleration rates for the passenger cars on two-lane mountainous highways. However, there is very limited research are available on mountainous terrains.

The recent studies (Pérez-Zuriaga et al. 2013; Nama et al. 2016) showed that the speeds are not constant over the curve, the collected continuous data on acceleration/deceleration over the curve showed that the deceleration values don't stop at the point of the curve (PC), whereas it extends into the curved portion, similarly, the acceleration behavior doesn't start from the point of tangent they emerge from within the curve section only. So it is necessary to explore the acceleration/deceleration behavior of the driver over the horizontal curve to clearly understand its true behavior.

However, little research has been done in predicting acceleration/deceleration rates using speed profiles. According to the literature, the studies on vehicle acceleration and deceleration on mountainous highways are still in the rudimentary stage and the studies on four-lane highways are also very little. With the increase in the traffic demand on highways, nowadays the highway authority is converting the two-lane highways into four-lane highways. In addition, available models on acceleration/deceleration are restricted to two-lane traffic only. Hence there is a need to study the acceleration/deceleration behavior on four-lane highways in mountainous terrains.

Hence the major objective of this study is the modeling of acceleration and deceleration behavior of drivers on four-lane highways over mountainous terrains.

2 Data Collection

Unlike most of the studies on acceleration/deceleration where data was collected using radar gun at specific predefined locations, in this study a continuous speed and vehicle position data were collected. For data collection, a clear stretch unaffected by surrounding structures was selected having a length of 40 km on NH-3 outside the urban area. The 40 km stretch covers a wide range of gradients, curve radius, curve length, etc. This stretch is on mountainous terrain having a four-lane divided highway. For the data collection process, 30 number of well-experienced drivers who were also familiar with the study stretch is selected. Each driver used their own vehicle. All the vehicles used in this study are new with no more than 3 years old. All these vehicles used in data collection are equipped with a highly accurate GPS device which is having a frequency of 10 Hz. This GPS device can give a vehicle position and the corresponding speed of the vehicle at that particular position. The longitudinal vehicle speed is measured with the help of the Doppler Shift technique using five or more satellite data simultaneously. The GPS device is also attached with a camera, which helps in studying the surrounding conditions of the vehicle at a given position of the vehicle. The data collection process is started on 15th November 2017 and extended up to 30th December 2017. During this period, each day, one driver was picked among 30 multiple drives and ran the GPS equipped vehicle over the 40 km study stretch, of a four-lane highway (NH-3) in mountainous terrain.

3 Data Processing, Reduction, and Understanding

The GPS along with speed data was collected at a frequency of 10 Hz. The GPS data collected shows the latitude and longitude in minutes and the data logger file also consists of UTM time with 0.1 s intervals. This helps in extracting the necessary data from the GPS data logger file.

In the first stage to make all the driver's position at the same location and to relate one driver data with the other, the data was extracted at intervals of 4 meters. That is, at every four meters interval, the details of UTM time, Latitude and Longitude positions, and Longitudinal speed data were extracted. From this, all the non-free flow data was removed. Thereafter, a program was developed in MATLAB 2018a to filter outliers and smoothen the extracted speed values. Figure 1 shows the filtered speed values of all the drivers over a 90 m stretch. Thereafter all the driver's speed at a given position of every 4 m intervals are averaged. So that position based average speed profile for a curve consisting of all free-flow speed vehicle data is achieved as shown in Fig. 2a. Since the objective is focused on acceleration/deceleration, these

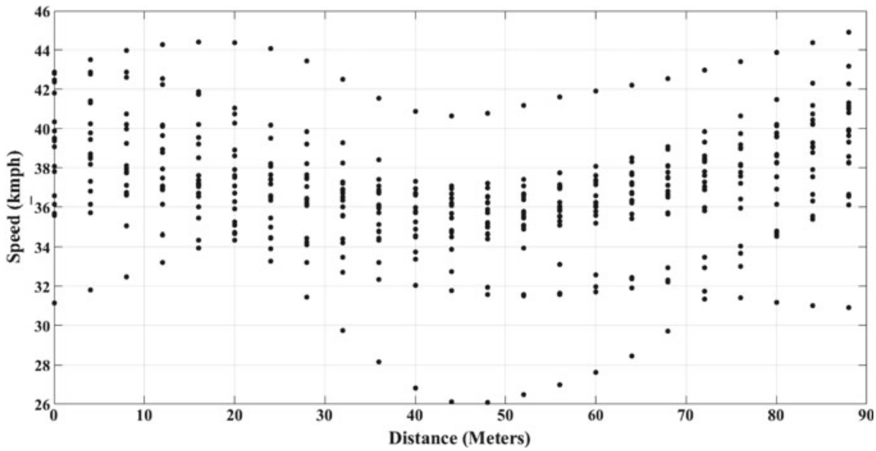

Fig. 1 Speed values of all the drivers collected from GPS device at 4 m interval

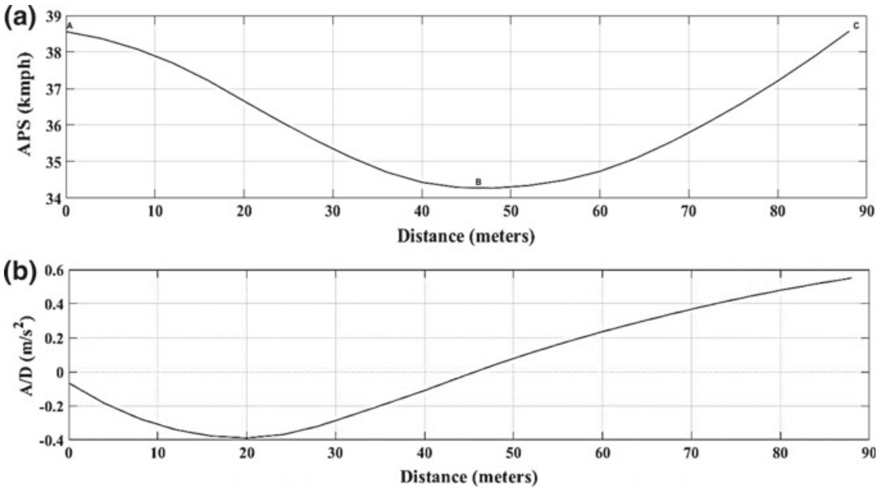

Fig. 2 **a** Average position speed (APS) profile. **b** Acceleration/deceleration (A/D) profile obtained from APS

values are measured using the following formula.

$$\text{acceleration/deceleration} = \frac{V_t^2 - V_{t-1}^2}{2 * d} \quad (1)$$

where

V_t and V_{t-1} are the average speeds of all the drivers at positions d_t and d_{t+1} during t and $t + 1$ times, respectively.

$d = d_{t+1} - d_t$, in this study d is fixed as 4 meters.

The other geometric data of the curves were obtained from the plan and profile drawing provided by the highway maintenance authority. The data from the drawings are verified in the field by collecting data using Laser Distance Meter and Trundle Meter. The selected 40 km stretch consists of various types of curves such as reverse curves, combined curves, isolated curves, etc., The present study focuses on simple curves instead of covering all the types. A total of 34 such sections are identified within the study stretch. The detailed descriptive statistics of the sections are presented in Table 1.

4 Deceleration and Acceleration Models for the Approaching and Departing Vehicles

It is observed from the average position based speed profile diagrams like in Fig. 2a, that there are two phases in which drivers cross a curve from one tangent to another.

Table 1 Descriptive Statistics of considered geometric data

Geometric parameter[a]	Minimum	Maximum	Mean	Standard deviation
Radius (*R*)	20.00	325.00	109.22	81.98
Curve Length (CL)	10.00	250.00	57.48	44.94
Spiral Length (SL)	0.00	60.00	29.12	13.51
Vertical Gradient (VG)	−6.26	6.69	1.07	3.89
Preceding Tangent Length[b] (PTL)	8.00	250.00	70.37	59.03
Succeeding Tangent Length[b] (STL)	8.00	400.00	77.48	80.49

[a] All units are in meters and VG is in percentage
[b] Doesn't include spiral length

The first one is the deceleration phase where the vehicle tries to reduce its speed so as to pass the curve safely, where the design of the horizontal curve plays a crucial role. The second one is the acceleration phase in this the driver accelerates after attaining their safe (minimum) speed over the curve. In Fig. 2, the region between A–B is in phase one, and B–C is in phase two. The acceleration/deceleration values are measured from average passion speeds (APS or V_t) using Eq. 1, for all the 34 sections considered in this study. The average speed acceleration/deceleration profile obtained for one of the curves is shown in Fig. 2b.

The maximum, minimum, and the average acceleration and deceleration values are measured for all the 34 sections and the details of them are presented in Table 2.

where "a" and "d" represents acceleration and deceleration.

With the help of the data available in Table 2, It is observed that the average deceleration values d_{Avg} can be simply obtained by the athematic average of both d_{Max} and d_{Min}. It implies that the rate of deceleration is constant in case of the approaching portion A to B. Whereas in the case of acceleration the values of a_{Avg} are very much nearer to a_{Max} for all the statistical cases of maximum, minimum, and mean as shown in Table 2, which clearly defines the advancements in the vehicle acceleration power of the vehicle, where the drivers reach their maximum acceleration in spit seconds

Table 2 Descriptive statistics of position based acceleration–deceleration values

	Minimum	Maximum	Mean	Standard Deviation
d_{Max}	−1.013	−0.013	−0.359	0.302
d_{Avg}	−0.516	−0.008	−0.182	0.152
d_{Min}	−0.148	−0.001	−0.034	0.037
a_{Max}	0.011	0.684	0.368	0.196
a_{Avg}	0.011	0.558	0.224	0.135
a_{Min}	0.001	0.200	0.030	0.036

and continuing with that acceleration for the rest of the section. This implies that the slope of accelerations are much steeper than the decelerations (0.224 > 0.182).

To develop the models, it was observed from the earlier studies that these models of acceleration/deceleration can be developed using Ordinary Least Square Regression technique. So in the current study also the models are developed using regression techniques for 95% confidence interval. In this method acceleration/deceleration is considered as dependent variables and the geometric parameters of the curve as independent parameters. Various statistics of the geometric parameters considered and are presented in Table 1. Out of 34 sections considered for data extraction, thirty are utilized in the development of the models and the remaining four sections are used in validation.

In the process of model development, using stepwise regression technique, different subsets of 26 sections are taken from the available 30 sections, for each case of acceleration and decelerations (d_{Max}, d_{Avg}, d_{Min}, a_{Max}, a_{Avg}, and a_{Min}) and the rest of the 4 data points are considered as outliers or redundant data points for model development. Since the selection of subsets is iteration process to simplify the method the process of elimination was used (similar to stepwise regression method). In this, the final four sections in the subset providing highest residual values from the model are replaced with the outside remaining sections one by one. This way it reduces the total number of iterations in order to achieve the best fitting model. The best obtained R^2 of top 4 models of accelerations and decelerations of all the six cases are presented in the following Table 3.

4.1 Deceleration Models

In Table 3, Eqs. 2(a, b, c & d), 3(a, b, c & d), and 4(a, b, c & d) represent the deceleration models for maximum, average, and minimum respectively.

In the case of maximum decelerations, the parameters radius (R) and previous tangent length (PTL) geometric variables are found to be the most influential parameters at 95% confidence interval. Since all the models 2(a)–(d) consists of these parameters R and PTL, it can be suggested that the model 2(d), having the highest R^2 is the best representation of maximum deceleration (d_{Max}).

In Table 3 Eqs. 3(a)–3(d) represents the prediction models of average deceleration. In this case also the parameters R and PTL are found to be the statistically significant parameter to represent the average deceleration values. Since all the models 3(a)–(d) consists of these parameters R and PTL, it can be suggested that the model 3(d), having the highest R^2 is the best representation of average deceleration $\left(d_{\text{Avg}}\right)$ model. Model 3(d) also represents that the magnitude of deceleration increases with the longer tangents length and smaller radius. So in order to improve the safety of a curve, the parameters PTL and R are to be selected in such a way that the deceleration values are in the safer limits.

Unlike the d_{Max} and d_{Avg}, the models for minimum deceleration doesn't depend on the PTL where as its starts depending on the vertical gradient (VG) of the curve and R.

Table 3 Developed models of deceleration and acceleration

Response variable	Regression models	adj R^2 (%)	
Maximum declaration (d_{Max})	$-0.753 + 0.00934\ R - 0.002639\ \text{PTL} - 0.000023\ R^2$	50	(2a)
	$0.2047 - 0.003100\ \text{PTL} - 23.61\frac{1}{R}$	63	(2b)
	$-0.8199 + 0.01030\ R - 0.003049\ \text{PTL} - 0.000025\ R^2$.	64	(2c)
	$-0.8073 + 0.01000\ R - 0.003090\ \text{PTL} - 0.000024\ R^2$	65	(2d)
Average declaration $\left(d_{\text{Avg}}\right)$	$-0.3853 + 0.004614\ R - 0.001200\text{PTL} - 0.000011\ R^2$	57	(3a)
	$-0.4084 + 0.004908\ R - 0.001422\ \text{PTL} - 0.000012\ R^2$	60	(3b)
	$0.0873 - 0.001375\ \text{PTL} - \frac{12.011}{R}$	62	(3c)
	$0.0952 - 0.001441\ \text{PTL} - \frac{12.011}{R}$	62	(3d)
Minimum Declaration (d_{Min})	$-0.0861 + 0.000707\ R - 0.000001\ R^2$	42	(4a)
	$-0.00391 + 0.00342\ \text{VG} - 2.230\frac{1}{R}$	45	(4b)
	$0.00730 + 0.00341\ \text{VG} - 3.107\ \frac{1}{R}$	61	(4c)
	$0.00653 + 0.00304\ \text{VG} - \frac{3.1341}{R}$	64	(4d)
Maximum acceleration (a_{Max})	$0.6454 - 0.00337\ R - 0.03562\ \text{VG} + 0.000007\ R^2$	60	(5a)
	$0.5248 - 0.001137\ R - 0.02832\ \text{VG}$	60	(5b)
	$1.057 - 0.00665\ R - 0.03232\ \text{VG} - 0.0588\ \text{L/R} - 9.881/R + 0.000014\ R^2$	71	(5c)
	$0.778 - 0.00525\ R + 0.00341\ \text{SL} - 0.03342\ \text{VG} - 8.261/R + 0.000011\ R^2$	74	(5d)
Average acceleration $\left(a_{\text{Avg}}\right)$	$0.3101 - 0.000730 - 0.02328\ \text{VG}$	51	(6a)
	$0.3120 - 0.000702\ R - 0.02318\ \text{VG} - 0.0000965\ \text{STL}$	55	(6b)
	$0.3591 - 0.000715\ R - 0.02236\ \text{VG} - 0.000426\ \text{STL}$	57	(6c)
	$0.3457 - 0.000646\ R - 0.02003\ \text{VG} - 0.000365\ \text{STL}$	60	(6d)
Minimum acceleration (a_{Min})	–	–	–

Note None of the geometric parameters are able to predict d_{Min}

This is because of the maximum deceleration (considering only the magnitude)d_{Max} occurs way before the point of the curve and in the case d_{Avg}, it depends on the PTL because the majority portion of the deceleration (based on length of the road used for deceleration) occurs on the tangent/Spiral path, it can be assumed that there is a possibility of the influence of PTL on these parameters. Whereas in the case of the minimum deceleration (d_{Min}) it occurs almost at the center of the curve in most of the cases so it is reasonable what the models in Eqs. 4 representing, that R and VG are the best geometric variables to predict the behavior of d_{Min}.Since model 4(d) provides a better fit having the highest a R^2 of 64%, so this model is the best to predict the minimum deceleration values occurring almost in the middle of the curve.

4.2 Acceleration Models

The models for maximum acceleration are presented in Eq. 5. From these equations, it is observed that the radius and vertical gradient acts as predominant influencing parameters over the maximum acceleration values in all the cases. In model 5(c) and 5(d) the parameters curve length and spiral length are also found to be influential. The model 5(d) gives a better fit $\left(R^2 \text{ value}\right)$ compared to 5(c) for the same number of parameters, hence it can be concluded that the model 5(d) can able to predict the maximum acceleration behavior of the driver.

It is observed from models 6(a)–(d) that the average acceleration depends on Radius, vertical gradient, and succeeding tangent length. The model 6(d) is having the highest R^2 value of 60 consisting of parameters R, VG, and STL.

In the case of minimum acceleration, it hasn't been found any geometric parameter that it is influencing the driver. Since this minimum acceleration comes once the driver decides to rise up the speed and reaches the maximum speeds at split seconds. In the case of minimum deceleration between A and B phase the drivers are forced to reduce their speed to go through the curve safely. Whereas in the case of minimum acceleration the drivers are free to choose where to accelerate, hence this is the free choice of the driver the stochasticity in the data increases and no other geometric parameters are unable to predict this. In the maximum acceleration case, it is observed that all the drivers are in the stage of reaching their maximum speed since they already passed the curve and approaching the straight section, the model 5(d) also explains that the spiral length provided clearly helps in achieving their maximum speed.

The acceleration models reveal that the parameter R, VG, and STL are the most influential parameters in departing phase B to C. Similarly, in the approaching phase the deceleration values are statistically dependent on R, VG, and PTL. So from these models, it can be seen that the acceleration and deceleration behavior depends totally on 4 geometric parameters PTL, R, VG, and STL. So it is necessary to provide proper tangent lengths and curvature for a given gradient to improve the safety of a horizontal curve.

5 Model Validation

In this section, the chosen developed acceleration/deceleration models are validated with the field data kept for validation.

To validate the models, the method described by Esposito et al. (2011) was used. This involves the calculation of I-value. If the I-value is between 0 and 0.2, then the model is said to be acceptable. The formulation to estimate I-value is given in Eq. 7.

$$I = \frac{\left(\sqrt{\frac{\sum_i^n (\text{Observed Value}-\text{Predicted Value})_i^2}{n}}\right)}{\left(\sum_i^n \text{Predicted Value}_i\right)/n} \tag{7}$$

where i refers to the section number and n refers to a total number of sections considered.

On calculation of I value for each type of chosen models (2 (d), 3 (d), 4 (d), 5 (d) and 6 (d)). It is found that the developed prediction models are giving an I-value in between 0 and 0.2. Hence, the developed models are valid for calculation of d_{Max}, d_{Avg}, d_{Min}, a_{Max} and a_{Avg}.

6 Conclusions

Unlike most of the studies where accelerations are derived from the spot speed data, the present study focuses on collecting continuous speed data more accurately by attaching GPS devices to the moving vehicle.

The outcomes of the continuous data in the present study show that the speed doesn't remain constant throughout the curve and acceleration/deceleration also occurs within the curve. These findings on four-lane highways are in align with the studies conducted on two-lane highways (Figueroa Medina and Tarko 2007; Pérez Zuriaga et al. 2010). This indicates that the similarities in driving behavior patterns between two-lane and four-lane highways.

It is also found that the rate of change in accelerations are much higher than the corresponding deceleration values, which might due to the drivers spending much time in decelerating during the approaching zone A–B. Whereas in the departure zone B–C the vehicles accelerate in split seconds because of availability of high horsepower engines in the latest vehicles.

It was found that the acceleration/deceleration values obtained are not constant over the horizontal curve in mountainous terrains. The deceleration shows a strong relation to the approaching tangent length of the curve. The acceleration depends on the length of the departure tangent. In addition to that, both the acceleration and deceleration also depends on the geometric parameters such as radius and gradient of the curve. So there is a possibility of driver choosing unappropriated acceleration or deceleration if proper tangent length and radius are not provided for a given gradient.

The present developed models can also help in the prediction of the critical acceleration and deceleration values for given geometric parameters. With the help of this, the vehicle acceleration and deceleration behavior can be altered by finding suitable geometric parameters, if the values of acceleration/deceleration values fall beyond the safer acceleration/deceleration limits.

Further studies can be conducted on four-lane highways with other types of terrains such as a plane and rolling terrains. The acceleration and deceleration behavior of vehicles for the 85th percentile speed vehicle can be studied. The same experimental procedure can also be repeated to study the driving behavior under rainy and night conditions can also be explored.

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Evaluation of Road Safety Audit Implementation Using Crash Reduction Factor and HDM-4

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Abstract Road Safety Audit (RSA) is a formal and independent safety performance review of road transportation projects by an experienced team of safety specialists to address safety. Noida–Greater Noida (NGN) Expressway is aneight-lane divided carriageway connecting Noida and Greater Noida. This Expressway has witnessed high crash rates and therefore, a comprehensive RSA was conducted and suggestions for improving safety on this expressway were recommended. These included replacement of concrete guard post on median by double row metal beam crash barrier, redevelopment of entry and exit points as per Indian Standards, marking and road studs on each of the traffic lanes and particularly near exit/entry points of auxiliary lanes. The Speed Enforcement Cameras and Close Circuit TV with Variable Message Signs were also installed at every 2 km. As the RSA recommendations were implemented in the year 2013 by the concerned stakeholders, an analysis was carried out to understand whether the measures had contributed towards the reduction in road crashes if any in the subsequent years 2014, 2015 and 2016. It was evident from this analysis that the total crashes after RSA had exhibited 84% reduction coupled with 91% reduction in fatal crashes. The analysis further revealed that the average speed of all types of vehicle had marginally increased after implementation of RSA recommendations, despite an annual increase in traffic in addition to crash reduction. The paper culminates with the analysis of the crash data to arrive at crash reduction factor followed by the use of HDM-IV wherein the economic benefits derived based on crash reduction was deduced.

Keywords Crash reduction factors · Business as-usual scenario

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1 Introduction

Road Safety Audit (RSA) is a formal procedure for assessing crash potential and safety performance of new road schemes as well as schemes for the improvement and maintenance of existing roads. The principle issue that would be addressed from RSA is to ensure that any road facility should operate as safely as possible. This implies that safety should be considered throughout the entire cycle of design, construction and pre-opening of any project facility as well as during operation and maintenance of any road facility.

In this paper, evaluate the RSA implementations crash reduction factors (CRF) are developed to estimate the reduction of crashes in the study stretch area for a certain period of time and can be used for implementing the countermeasures. Before-and-after method is used for calculating the CRF's.

Whether the implementation of RSA recommendations has indeed benefitted in terms of economic aspects Highway Development and Management (HDM-4) tool is used. The HDM-4 system is a software tool that is used to appraise the technical and economic aspects of road investment projects.

2 Study Area

Figure 1 Noida–Greater Noida (*NGN*) Expressway (*shown in Figure*) is a 3-lane dual carriageway connecting Noida, and Greater Noida the two major industrial cum

Fig. 1 Study stretch area Mahamaya to Pari chow

residential township developments located in the state of Uttar Pradesh. Since both these townships have grown rapidly over the years due to their proximity to Delhi and also due to the fact that they are falling under the National Capital Territory (*NCT*) of Delhi New Okhla Industrial Development Authority (*NOIDA*) have constructed the above expressway aimed at providing fast track road connectivity between the above two townships, the National Highway 2 connecting to the city Agra which was already congested as it ran through the heart of cities like Faridabad, Ballabgarh and Palwal. This expressway also connects Yamuna Expressway and also plans on the anvil to connect the expressway to Taj Economic Zone, International Airport and Aviation Hub.

3 Road Safety Audit (RSA) Recommendations

Road Safety Audit (RSA) is a formal procedure for assessing crash potential and safety performance in the provision of new road schemes and schemes for the improvement and maintenance of existing roads. The RSA suggested several safety recommendations such as the provision of double metal beam crash barrier (MCB's) of two rows, removal of barbed wire fencing on the median and placing them on Service Roads, host of Intelligent Transport System (ITS) solutions for effective road surveillance and speed monitoring and enforcement, Suggested deploying optical speed bars placed perpendicular to the median edge line with thermoplastic tapes, providing the auxiliary lanes, chevron marking and erection of channelizes before and after merging or diverging, etc. Most of the recommendations were implemented.

4 Data Collection

As part of the Operation and Maintenance Stage Road Safety Audit Check List conforming to Manual on Road Safety Audit (IRC: SP-88 2010),[1] various traffic studies such as Classified Volume Counts (TVC), Spot speed survey and Speed and Delay Studies (2 runs each covering morning and evening peak hours on both directions of travel) were conducted before and after RSA to understand the traffic characteristics on the study corridor. 24-hour TVC studies were carried out at strategically two locations namely near Mahamaya Flyover (Bus Stop No. 1) and near the end of the expressway (Bus Stop No. 8 and 9) to cover both directions of travel.

As the RSA recommendations were implemented in the year 2013 by the concerned stakeholders, an analysis was carried out to understand whether the measures have contributed towards the reduction in road crashes or not during the years 2014, 2015 and 2016, the above crash data have been compared with those

[1]IRC: SP-88 (2010), "Manual for Road Safety Audit" Indian Roads Congress, New Delhi.

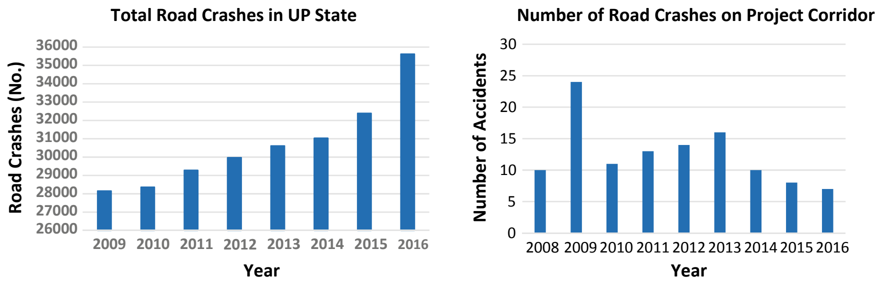

Fig. 2 Trend of Year-wise Road Crash Statistics in the State of Uttar Pradesh and Project Corridor

collected for the period from 2008 to 2012 which is period prior to the implementation of RSA action plan. Road crash statistics for the project corridor were compiled from First Information Records (FIRs) of the police records from 2008 onwards. At the same time, road crash data for the state of Uttar Pradesh were collected from the *Ministry of Road Transport and Highway* (MoRT&H 2012–2015) for the same period (refer Fig. 2).[2] It is evident that the number of crashes on the project corridor has reduced since 2014 which shows the effectiveness of RSA. In this context, the above crash data were compared with those collected for the period between 2008 and 2012[3] which is basically prior to the implementation of RSA action plan (refer Fig. 2). It was evident from this analysis that the total crashes after RSA had exhibited 84% reduction coupled with 91% reduction in fatal crashes. The analysis further revealed that the average speed of all types of the vehicle had marginally increased after implementation of RSA recommendations, despite an annual increase in traffic in addition to crash reduction.

Moreover, the traffic data was collected based on existing traffic conditions like Annual Average Daily Traffic (AADT) based on volume counts, spot speed studies and total travel time of the study stretch area and road data based on the geometric conditions of the study stretch area for both before and after RSA implementations.

Profile of Road Safety Audit (RSA) Implementation Scenario

Several safety associated recommendations were implemented on the study corridor conforming to the RSA recommendations. Some of the typical scenarios depicting 'before' and 'after' scenarios of RSA implementation are briefed here.

Implementations of safety recommendations

(1) The central verge (median) was having concrete guard post fenced with barbed wire fencing before RSA which was identified to be hazardous for the errant vehicles in case of road crash. Further, intermittent openings provided on the

[2]"Ministry of road transport and highways" statistics of road accidents in India 2008–2016.

[3]"Ministry of road transport and highways" statistics of road accidents in India 2008–2015.

Fig. 3 Before and after the condition of central verge

above fencing system prompt various road users including motorized two-wheelers as well as pedestrians to use the same for illegal crossing. It was recommended to remove the guard rails and install the double row metal beam crash barrier as shown in Fig. 3.

(2) Three entry/exit points were located on the project corridor without proper entrance angle as well as absence of acceleration or deceleration lanes. As a result, many road crashes were taking place due to the haphazard movement of vehicles. The entry/exit points were redeveloped conforming to Indian Road Standards (IRC: SP-99 2009) leaving no space for confusion and thus improving safety. Typical treatment is shown in Fig. 4.

(3) There were no road markings, lane markings, mixing zone area markings at entry/exit points of the auxiliary lanes. The audit recommended for the placement of all types of relevant markings (including lane demarcation as emergency lane at every 2 km on the Left most lane as well as lanes for Heavy vehicles, Two-Wheelers and cars), installation of road studs on each of the traffic lanes for enhanced visibility during night time as per IRC: 35 (2015). These provisions were also implemented (Fig. 5).

(4) The audit team observed improper placement of road signs. The locations and size of the signs were not meeting the operating speed requirements. All the irrelevant signs were replaced by proper signs as per IRC: 67 (2015). Some of the traffic signs before and after the implementation are presented in Fig. 6.

(5) There was no form of Intelligent Transportation System (ITS) for managing the high-speed traffic before RSA. Subsequent to RSA Speed Enforcement Cameras and Close Circuit TV with Variable Message Signs were implemented at every 2 km as shown in Fig. 7

Fig. 4 Redesign of entry and exit points

Fig. 5 Road markings and road studs for better nigh visibility

Fig. 6 Traffic signs as per IRC: 67 (2012)

Fig. 7 Intelligent transportation systems application on expressway

Cost Incurred for the Project Implementation

Almost all recommendations of the RSA were implemented by the Expressway owning agency. The details of the cost incurred for safety implementations including the overlay and service road augmentation are presented in Table 1.

Table 1 Comparison of CRF methods

Parameter	Total number of road crashes		Persons killed		Persons injured	
	Method-1	Method-2	Method-1	Method-2	Method-1	Method-2
CRF	0.58	0.84	0.62	0.91	-0.03	-0.55
CMF	0.42	0.16	0.38	0.09	1.03	1.55
Values of R	0.346	0.346	0.338	0.338	0.382	0.382

Impact of Safety Implementation

The parameters evaluating the impact of RSA implementation included the quantum of traffic flow, speed characteristics, number of road crashes, and the severity of road crashes before and after RSA scenarios. In this regard, duration of 5 years is taken as the time period before RSA and 3 years of duration after RSA implementation.

Number of Road Crashes

The road crash data (FIRs) and its severity were collected from various police stations located along the study corridor. The data was collected before and after RSA implementation period as explained earlier. The summary of road crashes on the study corridor is presented in Table 2. The crash rate after implementation of RSA reduced but the injury rate has registered a sudden increase primarily due to bus collision in 2014, whereas the fatal crashes have decreased considerably after implementation of RSA.

Traffic volume

Traffic volume data collected on study corridor during road safety audit and after implementing the road safety recommendations indicated that traffic on the road has increased at the rate of 5% per annum.

Table 2 Road safety project cost details for 20 km of NOIDA greater NOIDA expressway

S. No.	Description of work	Amount (in Rs crore)
1	Cost of overlay and traffic control devices such as road signs, road marking, studs, removal of redundant signs, Km stones, OHMs, for the entire road stretch	120
2	Cost of service road strengthening/augmentation	28
3	Provision of auxiliary lanes for facilitating entry and exit movements (including geometric design improvements)	13.8
4	Foot over bridge	2.5
5	Improvement of central verge	31
6	Cross drainage works	2
7	Provision of intelligent transport system	28
8	Others (junction improvements, watering of plants, road maintenance, and gantry boards cost, etc.)	7.5
Total project cost		232.8

Table 3 Road crash scenario on the study corridor

Year	Total road crashes	Persons killed	Persons injured
2008	10	15	10
2009	24	24	23
2010	11	14	13
2011	13	16	10
2012	14	15	12
2013	16	7	10
2014	10	11	27
2015	8	6	9
2016	7	5	8

Traffic speeds

Spot speed surveys were conducted for 24 h at two locations before RSA implementations. The survey was repeated at the same locations after RSA recommendations were implemented. The speeds observed for different types of vehicles are compared in Table 3.

As may be seen, the average speed of all types of vehicles in general has improved after the implementation of RSA recommendations. The posted speed limit on the study corridor is 100 Kmph for light vehicles (car and two-wheelers) and 60 Kmph for bus and Light Commercial Vehicles (LCV). The 85th percentile speed of all vehicles (except for bus in direction from Mahamaya to Pari Chowk) has reduced after RSA.

Travel time

The journey time of a vehicle to travel from the starting point to ending point was determined using the floating car method. Runs conducted on the study corridor to acquire the average travel time with the comparison of before and after data, it was observed that the travel time reduced by about 29.73% in the direction Mahamaya to Pari Chowk and 22.28% in the direction Pari Chowk to Mahamaya.

Evaluation of Crash Reduction Factor (CRF) and Crash Modification Factor (CMF)

Crash reduction factors (CRFs) are used in road safety studies to predict safety benefits due to the reduced number of crashes. NCHRP Report 162 emphasizes the necessity of the CRF studies. A crash reduction factor (CRF) is the percent crash reduction that might be expected after implementing a given countermeasure at a specific site. Before-and-after method is used for calculating the CRF. In this method; the effects of interventions on safety are determined by number of crashes that occurred on the study stretch before and after implementations using CRF. The methods of estimation of CRF are discussed in detail in the subsequent sections.

Method-1: CRF by number of crashes

This method is simple for calculating the crash reduction factor for the safety improvements of a study area. The formula used for calculating the crash reduction factor is as follows:

$$\mathrm{CRF} = 1 - \frac{N_a}{N_b} \tag{1}$$

N_a Number of road crashes after implementation
N_b Number of road crashes before implementation.

Method-2: CRF by exposure of crashes

It is based on the assumption that if nothing has changed, the crash experience before improvement is a good estimate of what would have happened during the after period without improvement (Shen et al. 2003). The basic formula for deriving a CRF in actual application, exposure to crashes is often considered in order to account for any changes in crash exposure that may have occurred between the before and after period. Accordingly, the CRF is calculated based on crash rates as follows[4]:

$$\mathrm{CRF} = \frac{(\mathrm{CR}_b - \mathrm{CR}_a)}{\mathrm{CR}_b} = 1 - \frac{\mathrm{CR}_a}{\mathrm{CR}_b} \tag{2}$$

where CR_a and CR_b are the crash rates at a treated site before and after improvement.

$$\text{Crash Rate} = \frac{\text{Total Number of crashes}}{\text{Exposer}} \tag{3}$$

$$\text{Exposure} = \frac{(\text{Total kilometers}) * (\text{No.of days}) * (\text{Mean AADT})}{1,000,000} \tag{4}$$

$$\text{Mean AADT} = \frac{\text{Sum of AADT from each crash}}{\text{Total no of Crashes}} \tag{5}$$

Test of significance

Statistical test is necessary to determine whether crash reduction is significant or negligible (FDOT 2012). The Poisson comparison of Mean Test is used to determine if the crash reduction is statistically significant (i.e. significantly better, significantly worse, or no significant change). The formula for the Poisson Test based on a 95% confidence level is given by Eq. (6).[5]

[4] *Reference* 1: Joan Shen and Albert Gan "Development of Crash Reduction Factors".

[5] *Reference* 3: FDOT State Safety Office Crash Reduction Analysis System Hub (CRASH) Information Guide, June 4, 2012.

$$R = \left(\frac{\left(2.326 * \sqrt{b - 0.16}\right) - 0.35}{b} \right) \tag{6}$$

where

R Minimum significant percent reduction, and
b Total number of crashes before project implementation.

A CRF is said to be significant when it is equal or higher than R. When a positive CRF is greater than R, it is said to be "significantly better" on the other hand, when the absolute magnitude of a negative CRF is greater than R, it is said to be "significantly worse". A CRF is said to result in "no significant change" if its absolute value is less than R.

Impact of RSA on Crash Reduction

The crash reduction factors calculated by the two methods are given in Table 4. The CRF values using method-1 are constantly lower than those calculated by method-2. It is due to the number of parameters considered in two methods while calculating the CRF. Method-1 relies on number of road crashes in before and after scenario, while method-2 considers parameters like traffic volume and exposure. Therefore method-2 provides more realistic and higher values of CRF.

Crash Reduction Factor (CRF) for total road crashes is computed as 0.84 which indicates that the total crashes have exhibited 84% reduction subsequent to implementation of RSA recommendations. At the same time, CRF values for fatal road crashes or persons killed was found to be 0.91, which implies that the fatal crashes exhibited a significant reduction trend of 91%. The CRF value for persons injured has registered a negative sign which indicates the increase in number of persons injured. This is attributed to the one major crash that occurred in 2014 involving rear-end collision of a Truck and a Bus, resulting in injuring 12 persons. Table 4 also shows values of R, the R values for the total number of road crashes is 0.346 and persons

Table 4 Speed percentile data before and after

Direction of flow	Mode	April 2013, Kmph			August 2016, Kmph			% increase or decrease		
Mahamaya to Pari Chowk	Percentiles	15	50	85	15	50	85	15	50	85
	Car	52	73	88	61	75	88	17	3	0
	Bus	45	55	70	52	65	78	16	18	11
	LCV	40	50	70	45	55	65	13	9	-8
	Two wheeler	40	55	80	45	56	69	13	2	-13
Pari Chowk to Mahamaya	Car	68	85	105	66	79	91	-2	-7	-13
	Bus	50	65	80	70	59	79	40	-10	-1
	LCV	45	55	70	48	58	68	8	5	-3
	Two wheeler	45	55	75	43	57	74	-4	4	-1

killed is 0.338 which are lower than the CRF values of respective crash types. The *R*-value reinforces the fact that the reduction of road crashes as compared to the trend of road crashes before RSA is significant implying the fact that the total and fatal crashes have declined after RSA on the project corridor.

Crash Modification Factor

A Crash Modification Factor (CMF) is a multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site. The main difference between CRF and CMF is that CRF provides an estimate of the percentage reduction in crashes, while CMF is a multiplicative factor used to compute the expected number of crashes after implementing a given improvement. It is important to note that the CMF represents the long-term expected reduction in crashes. CMFs are used by several groups of transportation professionals for various reasons. The primary user groups include highway safety engineers, traffic engineers, highway designers, transportation planners, transportation researchers, and managers and administrators. CMFs can be used to:

- Capture the greatest safety gain with limited funds
- Compare safety consequences among various alternatives and locations
- Identify cost-effective strategies and locations
- Check reasonableness of evaluations (i.e. compare new analyses with existing CMFs)
- Check validity of assumptions in cost-benefit analyses.

Crash Modification Factor is used to calculate the number of accidents occurred after the implementation of safety measures at a specified length of road. The CMF is calculated according to the reduction of the accidents before-and-after the implementation (Bonneson and Zimmerman 2007). The CMF's are calculated as follows[6]:

$$\text{CMF} = 1 - \text{CRF} \tag{7}$$

here, CRF = Crash Reduction Factor.

If the CMF is greater than "1.0" then it is considered as crashes have increased, while CMF less than "1.0" indicates that crashes have increased. The values of CMF for three cases are given in Table 4. As may be seen, the CMF for the persons injured is more than one indicating the increase in this type of crashes. However, CMF for total number of accidents and total persons killed 9 s a 0.16 and 0.09, respectively.

[6] *Reference* 2: Bonneson and K. Zimmerman "Procedure for using accident modification factors in the highway design process".

5 Evaluation of Crash Reduction Factor

Crash Reduction Factors (CRF) was developed to estimate the reduction of crashes if any on the project corridor by considering the traffic flow data to understand the effectiveness of the countermeasures. Before-and-after method was used for calculating the CRFs. Crash occurred on the study stretch were considered at overall section level, i.e. macro level without giving any importance to the specific location data as the safety measures are taken at that level. Moreover, the study stretch area is linear and shorter span, individual geometric countermeasures were not considered. CRF computed for the study corridor was deduced as 0.84 which indicates that the total crashes have reduced 84% subsequent to implementation of RSA recommendations. At the same time, CRF for fatal road crashes or persons killed was found to be 0.91, which again implies a reduction in fatal crashes.

6 Application of HDM-4 for Road Safety

The Highway Development and Management (HDM-4) system is a software tool that can be employed for a critical appraisal economic aspects of road investment projects. It provides facilities for storing characteristics of road networks, vehicle types and road works. The available analysis types are:

- Project analysis: the economic evaluation of individual road projects or investment options.
- Programme analysis: the preparation of prioritized work programmes in which investment alternatives are defined, and selected subject to resource constraints.
- Strategy analysis: the analysis of a whole road network.

In this study, the project level analysis was carried out.

6.1 Data Incorporated for HDM-4

Data incorporated by creating two projects with implementation and without the implementation to compare whether it is economical or not. Data incorporated to run analysis are like traffic flow patterns, speed flow types, climatic zone of the study area. Type of currency is also given for the output costs according to countries currency, vehicle type based on vehicle fleet, unit costs in the network analysis and improvements of surface conditions come under the maintenance standards. After giving every detail in the software projects folder is selected to run the analysis and results are generated in report form.

Configuration of data incorporated in HDM-4

- Traffic flow patterns are defined as a set of flow periods. A flow period represents the hours of the day with the same traffic flow. For each flow period should be specified as total hours per year that the period occupies and the amount of yearly traffic occurring during the flow period based on the proportion of AADT that occurs during each hour of the flow period or the percentage of AADT that occurs during each flow period.
- Speed flow types are defined based on the capacity characteristics in terms of ultimate capacity and free flow capacity. The speed flow type is created for both before and after analysis.
- Climate Zones are used to represent the climatic conditions found in different parts of a road network which effect pavement deterioration. Climatic zone data are divided into two categories that are moisture and temperature.
- Vehicle fleets are used to store the details of the vehicle types to be included in HDM-4 analysis, the vehicle fleet is also used to define traffic growth sets. A traffic growth set defines how the traffic grows over time and is assigned to sections within an analysis HDM-4 includes 20 default vehicle types fall into the following two categories motorized category and non-motorized category.
- A HDM-4 road network stores details of the roads that are taken for analysis. Each road network consists of several sections. A section typically corresponds to an identifiable length of road, but may also be a representative section created solely for analysis. HDM-4 uses the concept of "homogenous sections", where each section has uniform strength, geometry, traffic and condition characteristics over its entire length.
- For improving networks, improvement standards define the road improvement works to be carried out when the triggering condition is met. Each Improvement Standard is defined in terms of the road surface class to which it applies a triggering condition, an improvement type, the costs & duration of the works, and the resultant effect on the pavement in terms of its condition, geometry, strength, etc.

6.2 *Crash Data Analysis in HDM-4*

The base year for the crash data analysis was taken as 2013 as the safety recommendations were implemented during the above year. Hence, an analysis has been carried out to understand whether the measures have contributed towards the reduction in road crashes or not during the years 2014, 2015 and 2016 and beyond. Subsequently, crash rates per 100 million vehicle-km are calculated by considering total number of vehicles, length of the study section, fatal crashes and injuries. These were given as total crashes as well as in terms of their severity like fatal crashes, serious injurious crashes and property damage. This is calculated as follows:

$$\text{Crash classes} = \frac{100\,\text{million vehicles per km} * \text{Number of accidents}}{\text{Vehicles per km} * \text{length of the study section}}$$

The crash analysis in HDM is considered as the rate of crashes. The term crash rate is defined as follows:

$$\text{ACCRATE} = \frac{\text{ACCYR}}{\text{EXPOSURE}}$$

where,

ACCRATE	Crash rate in terms of crashes per 100 million veh-km
ACCYR	Number of road crashes per year
EXPOSURE	Annual exposure to road crashes.

The exposure is usually expressed in terms of 100 million veh-km and is calculated as follows:

$$\text{EXPOSINT} = \frac{\text{AADT 365}}{10^8}$$

$$\text{EXPOSSEC} = \frac{\text{AADT 365 SECTLEN}}{10^8}$$

where

EXPOSINT	Crash exposure at intersections in million vehicles
EXPOSSEC	Crash exposure between intersection in million veh-km
AADT	total traffic entering the intersection or using the section in veh/day
SECTLEN	length of the section in km.

The basic equations given in HDM-IV for modelling traffic safety are as follows:

$$\text{ACCYR} = \text{EXPOSSEC} * \text{ACCRATE}$$

$$\text{ACCOST} = \text{ACCYR} * \text{UNITCOST}$$

where

ACCYR	number of road crashes per year
EXPOSEC	annual exposure to road crashes
ACCRATE	Crash rate in terms of crashes per 100 million veh-km
ACCOST	Crash cost for the type of road crash
UNITCOST	unit cost for the type of road crash.

6.2.1 HDM-IV Associated Results

Overall investments envisaged for maintaining the safety measures for upcoming analysis period costs bases on economic and financial costs are generated based on descriptions like Median Plantation maintenance, Traffic signboards, Road Markings, ITS Maintenance.

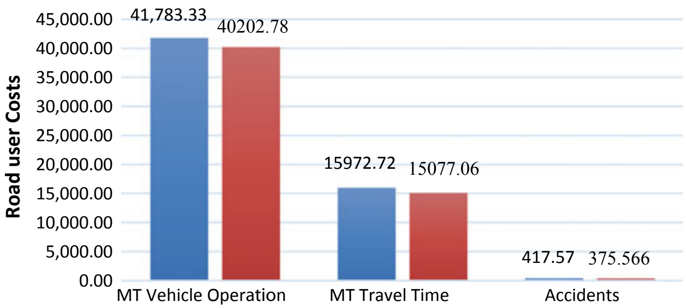

Fig. 8 Comparison of road user Costs for do-nothing and safety improvements

6.2.2 Share of Road User Costs

The share of road user cost mainly depends upon the total transport costs which are divided into road user costs and road agency costs as shown figure. The comparison of results derived 'before', i.e. Business-as-usual scenario (BAU) scenario and 'after' the incorporation of safety features including routine maintenance and overlay (as per the need) is shown in Fig. 8.

6.3 Calculation of Economic Internal Rate of Return Using HDM-4

The internal rate of return for the investments done on the project corridor including routine maintenance, overlay and RSA investments is shown in Table 5

The internal rate of return after analysis should be more than 12% as suggested by World Bank. However, the output of the economic analysis yielded an Internal Rate of Return (IRR) of ***23.1*** which is much more than 12% implying thereby the investments on the project corridor are beneficial purely from safety viewpoint.

7 Conclusion

- Crash Reduction Factor (CRF) for total road crashes is computed as 0.84 which indicates that the total crashes have reduced 84% subsequent to implementation of RSA recommendations. At the same time, CRF for fatal road crashes or persons killed was found to be 0.91, which again implies a reduction in fatal crashes. However, CRF value for persons injured has registered a negative sign which indicates the increase in the number of persons injured. This is attributed to one major crash that occurred in 2014 injuring 12 persons involved rear-end collision of truck with a bus, resulting in about 12 persons. The Crash Modification Factor

Table 5 Economic internal rate of return due to safety investment (In Rs. Lakhs)

Alternative	Present value of total agency costs (RAC)	Present value of agency capital costs (CAP)	Increase in agency costs (C)	Decrease in user cost (B)	Net present value (NPV = B + E − C)	NPV/cost ratio (NPV/RUC)	NPV/cost ratio (NPV/CAP)	Economic internal rate of return (EIRR in %)
Business as-usual scenario (BAU)	74.202	72.614	0.000	0.000	0.000	0.000	0.000	0.000
After implementation of safety	1,071.527	1,010.115	997.33	2,517.62	1,520.3	1.419	1.505	23.1

(CMF) was observed to vary between 9 and 16%. Statistical test was conducted to understand whether the reduction in CRF values is significant or not. The R values for the total number of road crashes are 34.59% and for persons killed is 33.83% which is lower than the CRF values of respective crash types. It supports the fact that the reduction in road crashes as compared to the trend of road crashes before RSA is not significant implying that the total and fatal crashes have declined after RSA on the project corridor.

- Using HDM-4 software life cycle of the project was predicted for 15 years. Using the HDM-IV tool, the investments done on the study stretch was computed. The output of the economic analysis yielded an Internal Rate of Return (IRR) of 23.1 which is much more than 12% implying thereby the investments on the project corridor are beneficial purely from a safety viewpoint.

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