Lecture Notes in Civil Engineering

Akhilesh Kumar Maurya
Bhargab Maitra
Rajat Rastogi
Animesh Das *Editors*

Proceedings of the Fifth International Conference of Transportation Research Group of India

5th CTRG Volume 2

Springer

Lecture Notes in Civil Engineering

Volume 219

Lecture Notes in Civil Engineering (LNCE) publishes the latest developments in Civil Engineering—quickly, informally and in top quality. Though original research reported in proceedings and post-proceedings represents the core of LNCE, edited volumes of exceptionally high quality and interest may also be considered for publication. Volumes published in LNCE embrace all aspects and subfields of, as well as new challenges in, Civil Engineering. Topics in the series include:

- Construction and Structural Mechanics
- Building Materials
- Concrete, Steel and Timber Structures
- Geotechnical Engineering
- Earthquake Engineering
- Coastal Engineering
- Ocean and Offshore Engineering; Ships and Floating Structures
- Hydraulics, Hydrology and Water Resources Engineering
- Environmental Engineering and Sustainability
- Structural Health and Monitoring
- Surveying and Geographical Information Systems
- Indoor Environments
- Transportation and Traffic
- Risk Analysis
- Safety and Security

To submit a proposal or request further information, please contact the appropriate Springer Editor:

– Pierpaolo Riva at pierpaolo.riva@springer.com (Europe and Americas);
– Swati Meherishi at swati.meherishi@springer.com (Asia - except China, and Australia, New Zealand);
– Wayne Hu at wayne.hu@springer.com (China).

All books in the series now indexed by Scopus and EI Compendex database!

More information about this series at https://link.springer.com/bookseries/15087

Akhilesh Kumar Maurya · Bhargab Maitra ·
Rajat Rastogi · Animesh Das
Editors

Proceedings of the Fifth International Conference of Transportation Research Group of India

5th CTRG Volume 2

Springer

Editors
Akhilesh Kumar Maurya
Department of Civil Engineering
Indian Institute of Technology Guwahati
Guwahati, Assam, India

Bhargab Maitra
Department of Civil Engineering
Indian Institute of Technology Kharagpur
Kharagpur, West Bengal, India

Rajat Rastogi
Department of Civil Engineering
Indian Institute of Technology Roorkee
Roorkee, Uttarakhand, India

Animesh Das
Department of Civil Engineering
Indian Institute of Technology Kanpur
Kanpur, Uttar Pradesh, India

ISSN 2366-2557 ISSN 2366-2565 (electronic)
Lecture Notes in Civil Engineering
ISBN 978-981-16-8258-2 ISBN 978-981-16-8259-9 (eBook)
https://doi.org/10.1007/978-981-16-8259-9

This Springer imprint is published by the registered company Springer Nature Singapore Pte Ltd.
The registered company address is: 152 Beach Road, #21-01/04 Gateway East, Singapore 189721, Singapore

Preface

The Transportation Research Group of India (TRG) has brought out this edited book titled “5th Conference of Transportation Research Group of India” in three volumes. Volume I of the book includes the papers submitted on themes, TCT-A01: Pavements and materials, TCT-D01: Travel behaviour and transport demand, and TCT-H01: Emerging travel technology, while Volume II covers papers on the theme TCT-B01: Traffic flow theory, operations and facilities; TCT-C01: Transport planning, policy, economics and project finance, and TCT-I01: Other transportation modes (including NMT) and pedestrian. Volume III presents a compilation of papers on the theme TCT-E01: Environment (including energy) and sustainability in transportation, TCT-F01: Transportation safety and security, and TCT-G01: Transport and mobility networks (including public transportation, freight and logistics). We acknowledge the support rendered in handling the review process by the co-chairs of the respective TCTs, namely Prof. Gowri Asaithambi, Prof. Debasis Basu, and Prof. Shriniwas Arkatkar.

Incidentally, this year also commemorates the successful completion of 10 years of TRG. TRG was an initiative to bring all transportation academicians and professionals to one platform to work on transportation-related problems in India and collectively propose remedial measures for better traffic and transportation in the country. We have been associated with the setting up of TRG from its inception, and as founding members, it gives us immense pleasure to see that this professional body has come up to an age where it can claim at the global level to be a representative body of transportation professionals in India. It is also a golden moment to share some of the good research works done by Indian transportation professionals in the areas of traffic flow, facilities, transport planning, economics, and other transport modes through this volume.

Volume II of the 5th Conference of Transportation Research Group of India consists of 35 papers. Out of these, 21 papers are related to traffic flow, operations, and facilities; nine papers present works done in the area of transport planning, policy, economics, and project finance; and five papers focus on other transportation modes which includes non-motorized transport and pedestrians. Research areas covered under theme TCT-B01 include saturation flow, queue discharge, and control delay estimation at signalized/unsignalized intersections, flows in merging sections, driver behaviour studies, simulation of traffic control management, and capacity of urban divided roads. Research papers on the theme TCT-C01 cover topics related to financing approaches and performance evaluation of transit projects or services, electric buses, LOS of urban parking systems, economic evaluation of highway projects, and effect of NPV, rental and land value on transit development. Research works on other varied topics related to electric two-wheelers, walk-access infrastructure, crowd density prediction, access environment of transit services, LOS approaches for sidewalks, and effect of street design, and operational factors on the use of green transport have been submitted under the theme TCT-I01. This book on conference proceedings is a compilation of good works that are screened through a double-blind review system and provides the readers with research thoughts that will take the future of transportation in the country forward.

It is our firm belief that this compilation of selected presentations made in the 5th Conference of Transportation Research Group of India held at Bhopal, India, will provoke research thoughts in the readers. This volume covers a wide spectrum of aspects related to traffic flows, operations, economics and finance of transit projects, simulation of flows at varied facilities, LOS, electric vehicles, pedestrian facilities, and access environment. We sincerely hope that readers will be further enlightened and enriched with the works included in the other two volumes.

Guwahati, India — Akhilesh Kumar Maurya
Kharagpur, India — Bhargab Maitra
Roorkee, India — Rajat Rastogi
Kanpur, India — Animesh Das

About TRG and CTRG

Transportation Research Group of India (TRG) is a not-for-profit registered society with the mission to aid India's overall growth through focused transportation research, education, and policies in the country. It was formally registered on 28th May 2011 and has completed 10 years of its journey this year. The following are the vision and objectives of TRG.

Vision

- To provide a unique forum within India for the interchange of ideas among transportation researchers, educators, managers, policymakers from India and all over the world, with the intention of covering all modes and sectors of transport (road, rail, air, and water; public and private; motorized and non-motorized) as well as all levels (urban, regional, inter-city, and rural transport) and for both passenger and

freight movement, in India, and at the same time to also address the transportation-related issues of safety, efficiency, economic and social development, local and global environmental impact, energy, land use, equity and access for the widest range of travellers with special needs, etc.
- To serve as a platform to guide and focus transportation research, education, and policies in India towards satisfying the country's needs and to assist in its overall growth.

Objectives

- To conduct a regular peer-reviewed conference in India so as to provide a dedicated platform for the exchange of ideas and knowledge among transportation researchers, educators, managers, and policymakers from India and all over the world from a perspective which is multi-modal, multi-disciplinary, multi-level, and multi-sectoral, but with an India-centric focus. Initially, this conference will be held every two years; however, the frequency may change as per the decision of the society from time to time.
- To publish a peer-reviewed journal of good international standard that considers and recognizes quality research work done for Indian conditions, but which also encourages quality research focused on other developing and developed countries that can potentially provide useful learning lessons to address Indian issues.
- To conduct other activities such as seminars, training and research programmes, meetings, discussions, etc., as decided by the society from time to time, towards fulfilling the mission and vision of the society.
- To identify pertinent issues of national importance, related to transportation research, education and policy through various activities of the society, and promote transportation researchers, educators, managers, and policymakers in an appropriate manner to address the same.
- To collaborate with other international societies and organizations like, WCTRS, ASCE, TRB, etc., in a manner that works towards fulfilling the mission and vision of the society.

The Conference of Transportation Research Group of India (CTRG) is the premier event of TRG. It is held every two years and traditionally moves around India. In the past, CTRG has been organized in Bangalore (December 2011), Agra (December 2013), Kolkata (December 2015), Mumbai (December 2017), Bhopal (December 2019), and Trichy (upcoming in December 2021 jointly with NIT Trichy, in association with IISc Bangalore, IIT Madras, IIT Palakkad and NATPAC). CTRG has been getting wide scale recognition from reputed Indian and international institutions/organizations, like IIT Kanpur, IIT Kharagpur, IIT Guwahati, IIT Bombay (Mumbai), SVNIT Surat, MANIT Bhopal, NIT Trichy, TRB, WCTRS, CSIR-CRRI, ATPIO, T&DI-ASCE, EASTS, to name a few. CTRG is a large conference typically attended by around 400–500 participants, usually from 12 to 15 countries, with about

200 double-blind peer-reviewed technical papers being presented. The conference provides a wide range of executive courses, tutorials, workshops, technical tours, keynote sessions, and special sessions.

Transportation in Developing Economies (TiDE) is the official journal of TRG and is published by Springer. TiDE was formally launched in 2014 and has so far published seven volumes.

Prof. Ashish Verma
Indian Institute of Science Bangalore
Founding and Immediate Past President, TRG

Prof. Akhilesh Kumar Maurya
Indian Institute of Technology Guwahati
Current President, TRG

List of Reviewers

A Ramesh, IIT-Ropar
A. Mohan Rao, CRRI
A. R. Ganesh, IIT-Madras
AbdulHassan Norhidayah, UTM Universiti Teknologi, Malaysia
Advani Mukti, CRRI
Agarwal Amit, IIT-Roorkee
Agarwal P. K., MANIT-Bhopal
Agarwala Ruchika, IIT-Bombay
Ahmad Juraidah, Universiti Teknologi MARA
Ali Notani Mohammad, Purdue University
Amarkumar D. S. N. V., Shell Bitumen, India
Andrii Galkin, O. M. Beketov National University of Urban Economy in Kharkiv, Kharkiv, Ukraine
Anjaneyulu M. V. L. R., NIT-Calicut
Anowar Sabreena, University of Missouri
Arepalli Uma, Maine Department of Transportation
Arkatkar Shriniwas, NIT-Surat
Arun Ashutosh, CSIR
Bachu AnilKumar, IIT-Patna
Bahari Rajneesh, Indian Army
Bandyopadhyaya Ranja, NIT-Patna
Banerjee Arunabha, IIT-Guwahati
Banerji Ipsita, IISc., Bangalore
Barman Manik, University of Minnesota, Duluth
Basha Munwar, IIT-Hyderabad
Basu Debasis, IIT Bhubaneswar
Behl Ambika, CRRI
Bellary Ashik, KLS, VDIT
Bera Reema, IIT-Kharagpur
Bhanumurthy P. R., JNTU, Andhra Pradesh
Bharath Gottumukkala, CRRI

Bhaskar Ashish, Queensland University of Technology
Bhat Prithvi, Queensland University of Technology
Bhattacharjee Sudip, Alabama A&M university
Bhowmik Tanmoy, University of Central Florida
Bhushan Sasanka, VIT University, Vellure
Bhuyan Prasanta, NIT-Rourkela
Biswas Mehek, IISc., Bangalore
Bolia Nomesh, IIT-Delhi
Bora Bhaswati, IIT-Kanpur
Budhkar Anuj, IIEST Shibpur, West Bengal
Chandrappa Anush, IIT-Bhubaneswar
Chaudhary Arti, CRRI
Chepuri Akhilesh, Sreenidhi Institute of Science and Technology, Ph.D. from SVNIT
Cheranchery Munavar Fairooz, TKM College of Engineering, Kerala
Chilukuri BhargavaRama, IIT-Madras
Chopra Tanuj, Thapar Institute of Engineering and Technology
Choudhary Pushpa, IIT Roorkee
Choudhary Rajan, IIT-Guwahati
Chowdary Venkaiah, NIT-Warangal
Dandapat Saurabh, IIT-Kharagpur
Das Atasi, GR Infratech
Das Bibhutibhusan, NIT-Surathkal
Das Pritikana, CSIR, CRRI
Das Sanhita, IIT-Guwahati
Das Subasish, Texas A&M university
Das Vivek R, Dayanand Sagar College of Engineering, Kerala
Delhi Venkata Santosh Kumar, IIT Bombay
Deshpande Ravi, Indian Army
Dey Partha Pratim, IIT Bhubaneswar
Divya P. V., IIT-Palakkad
Duddu Venkata Ramana, University of North Carolina, Charotte
Errampalli Madhu, CRRI
Gandhi Tejash, Kao-Corporation
Garikapati Venu, National Renewable Energy Laboratory, NREL
Ghataora G. S., University of Birmingham, UK
Ghosh Indrajit, IIT-Roorkee
Ghosh Kaniska, IIT-Kharagpur
Gogoi Ripunjoy, Amity University
Gopalaraju S. S. S. V., MVGR College of Engineering
Goswami Sutapa, University of Calcutta
Gottumukkala Bharath, CRRI
Govindaraju Shiva Kumar, University of Pisa UNIPI, Italy
GR Bivina, IIT-Roorkee
Gundaliya Pradip, L. D. College of Engineering, Gujarat
Gupta Ankit, IIT-BHU

Gupta Sanjay, School of Planning and Architecture, Delhi
Habal Ayyanna, IIT-Bombay
Hainin Mohd Rosli, UTM Universiti Teknologi, Malaysia
Haque Md Bashirul, University of Leeds, UK
Haque Shimul, Queensland University of Technology
Harihan Gayathri, IISc., Bangalore
Havangi Vasant, CRRI
Hesp Simon, Queen's University, Canada
Idara Praveen, University of Missouri, USA
Immanuel Suresh, University of Evansville
Ismail Norfarah Nadia, International Islamic University, Malaysia
Iyengar Abhijith, IIT-Madras
Izzi Nur, UKM National University of Malaysia
Jain Bhavesh, IIT-Roorkee
Jain Subham, IIT-Kanpur
Jain Udit, IIT-Roorkee
Jakkula Nataraju, CSIR
Jangpangi Lalita, CSIR
Julaganti Ashok, NIT-Warangal
Kadali Raghuram, Sri Vasavi Engineering College, Andhra Pradesh
Kamaruddin Nurul Hidayah, Universiti Tun Hussein Onn Malaysia, UTHM
Kanagaraj Venkatesan, IIT-Kanpur
Kar Siksha, CRRI
Karuppanan Gunasekaran, Anna University, Tamil Nadu
Kataware Aniket, IIT-Bombay
Kathuria Ankit, IIT-Jammu
Kayitha Ravinder, CRRI
Khan Ahmad, Queen's University, UK
Korlapati Durgarani, Andhra University, Andhra Pradesh
Koting Suhana, University of Malaysia
Krishnan Rajesh, Imperial College of London, UK
Kuity Ambika, NIT-Silchar
Kukkapalli VenuMadhav, University of North Carolina, Charlotte
Kumar Aditya, IIT-Bombay
Kumar Anjan, IIT-Guwahati
Kumar Anupam, Delft University
Kumar Binod, CRRI
Kumar Molugaram, UCE, Osmania University, Hyderabad
Kumar Patnala Phani, Echelon Intitute of Technology, Haryana
Kumar Prabin, IIT-Bombay
Kumar Rajiv, NIT-Jalandhar
Kumar Ravindra, CRRI
Kumar Vasantha, VIT, Vellore
Kuna Kranti Kumar, IIT-Kharagpur
KV Ganeshbabu, Amaravati Development Corporation, Andhra Pradesh

L Deepa, TKM College of Engineering, Kerala
Labi Sam, Purdue University, USA
Laxmanrao K. M., JNTUH College of Engineering, Hyderabad
M Harikrishna, NIT-Calicut
M Shashi, Indian School of Mines
M. Manoj, IIT-Delhi
M. K. Nivedya, Worcester Polytechnic Institute WPI, USA
M. N. Nagabhushana, CRRI
Mahapatra Geetimukta, IIT-Guwahati
Maheshwari Saurabh, University of California, Davis
Majhi Ramesh Chandra, CRRI
Maji Avijit, IIT-Bombay
Majumdar Bandhan, BITS-Pillani, Hyderabad
Malladi Haritha, Lafayette College, NYC
Mallikarjuna C, IIT-Guwahati
Mane Ajinkya, University of North Carolina, Charlotte
Manna Bappaditya, IIT-Delhi
Mathew Samson, NIT-Trichy
Mazzulla Gabriella, University of Calabria, Italy
Mehar Arpan, NIT-Warangal
Mehta Prakash, Tawata Technology, Gujarat
Mitra Swati, IIT Kharagpur
Mittal Abhishek, CRRI
Mohapatra Smruti Sourava, IIT-ISM Dhanbad
Mohan Mithun, NIT-Karnataka
Mohan Ranju, IISc., Bangalore
Mudgal Abhisek, IIT-BHU
Munigety Caleb, IISc., Bangalore
Murty Dasaka, IIT Bombay
Myakuntla SaiKiran, IISc., Bangalore
Nainegali Lohit kumar, IIT-Dhanbad
Nair Syam, IIT-Kanpur
Nanthagopalan Prakash, IIT-Bombay
Narzary Binanda, Tezpur University
Ningappa Akhandappagol, NIT-K, Surathkal
Nirmale Sangram, IISc., Bangalore
Ozkul Seckin, University of South Florida
Paleti Rajesh, Pennsylvania State University
Pan Pan, Queen's University, Canada
Panda Mahabir, NIT-Rourkela
Pande Anurag, California Polytechnic State University
Parida Manoranjan, IIT-Roorkee
Parida Purnima, CBRI
Paul Bhaskar, IIT-Kharagpur
Pawar Digvijay, IIT-Hyderabad

Penmetsa Praveena, University of Alabama
Perumal Vedagiri, IIT-Bombay
Prasad C. S. R. K., NIT-Warangal
Prasad Prashant, IIT-Kharagpur
Prasad M. Lakshmi Vara, NIT-Silchar
Prasadaraju G. V. R., JNTU Kakinada, Andhra Pradesh
Prusty Sushma, IIT-Bombay
Rahman Farzana, University of Asia-Pacific
Rajan Bharat, IIT-Bombay
Raju Sridhar, BIT-Pilani, Hyderabad
Ramadurai Gitakrishnan, IIT-Madras
Ramanathan Ayothiraman, IIT-Delhi
Ramanayya T. V., NIT-Warangal
Ransinchung G. D., IIT-Roorkee
Rao K. Ramachandra, IIT-Delhi
Reddy Amaranatha, IIT-Kharagpur
Reddy K. S., IIT-Kharagpur
Reddy SitaRami, VNIT, Nagpur
Rekha Padma, SRM Institute of Science and Technology
Rokade S., MANIT-Bhopal
Roy Sandeepan, IIT-Bombay
S Minal, CRRI
Sabavath Shankar, NIT-Warangal
Saboo Nikhil, IIT-BHU
Sagwal Romeil, CSIR
Saha Dulal, University of Waterloo
Sahoo Amarendra kumar, GMR Infrastructure Ltd.
Sahoo Umesh Chandra, IIT-Bhubaneswar
Sahu Prasant, BITS-Pilani, Hyderabad
Sahu Jagadish Prasad, IIT-Roorkee
Saladi Subbarao, Mahindra Ecole College of Engineering, Hyderabad
Santhakumar Moses, NIT-Trichy
Sara Binu, College of Engineering, Trivandam
Sarang Goutham, VIT-Vellore
Sasidharan Sajitha, IISc., Bangalore
Saxena Shobhit, IISc., Bangalore
Schneider Robert, School of Architecture and Urban Planning, University of Wisconsin, Milwaukee
Seetharaman Padma, CSIR
Sekhar Ravi, CSIR
Selvaraj Sathish kumar, University of Melbourne
Shah Yogesh, IITRM Ahmedabad
Shankar K. V. R. Ravi, NIT-Warangal
Shankar S, NIT-Warangal
Shankar S Ravi, CRRI

Sharma Niraj, CSIR
Shivashankar R, NIT-Surathkal
Showkat Burhan, IIT-Bombay
Shukla Manoj, CRRI
Sidharthan Raj, University of Nevada
Sil Gourab, BITS Pilani
Singh Bhupendra, NIT-Patna
Singh Dharamveer, IIT-Bombay
Singh Priyansh, BITS-Pilani
Singh Rina, CRRI
Singh Varun, MNNIT-Allahabad
Sinha Anil Kumar, CRRI
Sivagnanasundaram Kalaanidhi, Technion Isreal Institute of Technology
Sivakumar M., NIT-Calicut
Sivananadan R, IIT-Madras
Sreeram Anand, Hong Kong Polytechnic University; Visiting Scholar at University of Texas
Srinivasakumar R, Osmania University, Hyderabad
Srinivasan Karthik K, IIT-Madras
Sudheerbabu Y, Nimra College of Engineering and Technology, Andhra Pradesh
Suman Sanjeev Kr., NIT-Patna
Sunitha V, NIT-Trichy
Suresha S. N., NIT-Karnataka
Swami Mansha, IIT-Roorkee
Swamy Aravind Krishna, IIT-Delhi
Swarna Subba Rao, Survey of India, Dehradun
Syamsunur Deprizon, UCSI University, Malaysia
T. M. Rahul, IIT-Ropar
Tanvir Shams, North Carolina State University
Tipnis Manoj, L&T
Tiwari Devesh, CRRI
Vanajakshi Lelitha, IIT Madras
Vajjarapu Harsha, IISc., Bangalore
Varma Sudhir, IIT-Patna
Velaga Nagendra, IIT-Bombay
Venkatareddy K, NIT-Warangal
Venudharan Veena, IIT-Palakkad
Veraragavan A, IIT-Madras
Verma Ashish, IISc., Bangalore
Verma Meghna, Ramaiah Institute of Management, Bangalore
Vikram Durgesh, BITS-Pilani
Vinayaka Ram V., BITS-Pilani, Hyderabad
Vittal Guru, CRRI
Yaacob Haryati, Universiti Teknologi Malaysia
Yasmin Shamsunnahar, Queensland University of Technology, Australia

Contents

About the Editors

Dr. Akhilesh Kumar Maurya is currently Professor in Department of Civil Engineering at Indian Institute of Technology (IIT) Guwahati, India. He received his Ph.D. degree in Civil Engineering from Indian Institute of Technology (IIT) Kanpur and M. Tech. degree in Computer Aided Design from Indian Institute of Technology (IIT) Roorkee. He is active academically and professionally in the area of traffic flow modeling, driver behavior, road safety audit and accident analysis, traffic data collection and analysis studies since more than one decade. He has published more than 95 technical papers in international journals and conferences. He has received the DAAD fellowship for one month visited at Technische Universität Darmstadt. He is currently President of "Transportation Research Group (TRG)" of India, member of World Conference on Transport Research Society (WCTRS) and the life member of "Indian Roads Congress". He is also certified "Road safety Auditor" by International Road Federation (India) and Australian Road Research Board. Apart from several Indian Institutions, he has also delivered invited lectures at various International Institutes like Technische Universität Berlin, Technische Universität Darmstadt, University Duisburg-Essen and National University of Singapore.

Dr. Bhargab Maitra is currently a Professor in Department of Civil Engineering, and former Head of RCG School of Infrastructure Design and Management, Indian Institute of Technology (IIT) Kharagpur, India. His research interest includes travel behavior analysis, public transportation system, sustainable transportation, traffic management & control, road safety, and transport policy. He has published nearly 150 technical papers and case studies in various journals and proceedings of conferences, seminars and workshops. He has carried out several sponsored research and consultancy projects in the area of traffic and transportation system. He has been instrumental in leading the large interdisciplinary mega project "Future of Cities" involving more than 40 researchers across several departments and institutes in India and abroad. He is an Alexander von Humboldt Fellow and DAAD Fellow. He also received Pt. Jawaharlal Nehru Birth Centenary Award from the Indian Roads Congress.

Dr. Rajat Rastogi is presently working as Professor in the Department of Civil Engineering at Indian Institute of Technology (IIT) Roorkee. He has a research and teaching experience of 31 years in the area of Transportation Engineering. He pursued his Ph.D. from IIT Bombay and M.E. (Highways) and B.E. (Civil Engineering) from Punjab Engineering College, India. His research interests are in pedestrian and non-motorized traffic, travel behavior and its modelling, integration of transportation systems, policy analysis, traffic flow analysis and road safety. He has published over 125 papers in journals and conferences. He is a member of professional bodies like IRC, TRG of India, IUT, and IGS. He also worked as a core member in the development of Indian Highway Capacity Manual (Indo-HCM). Presently he is a member of IRC H-1 and H-8 committee and Co-convenor of G-6 committee. The present work on public transport, active transportation and pedestrian flows on escalators will be of value for relevant IRC documents or codes.

Dr. Animesh Das is a Professor in the Department of Civil Engineering, IIT Kanpur. Dr. Das's area of expertise is pavement material characterization, analysis, design, and evaluation. He is interested in both theoretical as well as experimental studies. He has worked on problems related to characterization of asphalt mix and aggregates, asphalt recycling, asphalt micromechanics, microstructure and image analysis of asphaltic material, pavement design optimization, pavement surface characterization etc. He has published several papers in various journals and conference proceedings of repute.

Estimation and Comparison of Saturation Flow at Signalized Intersection in Mixed Traffic Condition

Mangal Jyoti Mahapatra, Mukti Advani, and A. U. Ravi Shankar

1 Introduction

An intersection is the crucial point of conflicts and congestion in the road network. An intersection is also a place for the arrival and departure of traffic flows where the traffic flows from different directions mutually intersect and various vehicles run in mixing manner. To avoid vehicular conflict of space and time at intersection, traffic signals are mostly used worldwide. Signalized intersection refers to the area of intersection of two or more roads with controlled operation of the traffic movements. With the increase in traffic at intersections, it is required to install traffic control devices to regulate the movements through the intersection. To prevent accidents, conflicts, etc., traffic streams are separated in time by providing signalized intersection. Automatic traffic signals are the most commonly used traffic control devices installed at the road intersection. These devices allocate right of way to different approaches of the intersection as per the phase allocation in order to minimize the delay and also to avoid conflicts due to crossing traffic. The operation of signalized intersections is affected by the traffic flow, the composition of the traffic flow, the geometrical structure of the intersection, i.e., number of lanes, width of the approach, width of shoulder, and free movement (if any). While approaching an intersection, vehicles undergo the process of deceleration, braking, stop and then start, acceleration, which results in the waiting and start-up loss of vehicles. Signal timing is the technique to decide the signal cycle and effective green time for a particular intersection, phase, traffic flows, etc. Saturation flow is an important parameter in a signalized intersection. The saturation flow rate is defined as the maximum number of vehicles that can pass a stop line per hour during the green phase. The maximum flow rate occurs after

M. J. Mahapatra (✉) · A. U. Ravi Shankar
National Institute of Technology Karnataka, Surathkal, India

M. Advani
Central Road Research Institute, New Delhi, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_1

the start of the green phase and continues until that queue has moved completely. At the beginning of green signal phase on an approach to an intersection, vehicles react slowly before reaching a normal running speed; and the queue discharges at a constant rate after few seconds that is termed as saturation flow. Traffic demands vary by time of day, but this is controlled by applying the analysis method for the peak hour volume and utilizing the peak hour factor (PHF).

2 Objectives and Scope of Study

The objective of this study is to estimate saturation flow from the field observations. The objectives of this research work have been listed below, and the combination of all will help to achieve the ultimate goal for this research work:

i. To estimate saturation flow for approach roads of signalized intersections.
ii. To compare the saturation flow values with developed by different authors.

This study is based on data collected at two signalized intersections covering ten approaches in the city of Delhi. Width of these approaches ranges from 7.6 to 10.3 m (~2 lane to 3 lane roads).

3 Literature Review

Traffic stream is converted into passenger car unit (PCU) to simplify the mixed traffic condition into a single homogenous type. Many researchers have proposed various methods to estimate PCU values. INDO-HCM [4] deals with the methodology devised for determination of saturation flow in a signalized intersections. The equation developed for saturation flow is only valid for a specific range as given in Eq. (1).

$$\mathrm{USF}_0 = \left\{ \begin{array}{ll} 630; & \text{for } w < 7.0 \text{ m} \\ 1140 - 60w; & \text{for } 7.0 \leq w \leq 10.5 \text{ m} \\ 500; & \text{for } w \geq 10.5 \text{ m} \end{array} \right\} \tag{1}$$

where

USF_0 Unit-based saturation flow rate (in PCU/h/m);
w effective width of approach (m).

The saturation flow of the intersection approach for the movement group is as follows:

$$S = w * \mathrm{USF}_0 * f_{\mathrm{bb}} * f_{\mathrm{br}} * f_{\mathrm{is}} \tag{2}$$

where SF is the prevailing saturation flow rate (PCU/h); w is the effective width of the approach in 'm'; f_{bb} is the adjustment factor for bus blockage due to curbside bus stop; f_{br} is the adjustment factor for blockage of through vehicles by standing right turning vehicles waiting for their turn, and f_{is} is the adjustment factor for initial surge of vehicles due to approach flare and anticipation effect.

Webster and Cobbe [13] proposed that saturation flow of an intersection can be estimated directly using its approach width. In this study, saturation flow (S) was estimated in terms of PCU/h of green by considering the approach width (w) of the intersections in feet, and the authors did not pay any attention to the other affecting factors of saturation flow. Indian Road Congress (IRC) [5] used the model proposed by Webster and Cobbe [13]. Suitable adjustment factors are provided to account for the effect of left turns and right turns as shown in Eq. (3):

$$S = 525 * w \tag{3}$$

Sarna and Malhotra [10] conducted a study for the mixed traffic condition in India by collecting data from some of the signalized intersections in Delhi. The model is only applicable for the intersections having approach width between 16 and 8 feet which is a very narrow range and where right turning traffic is restricted to 10% only. Bhattarcharya and Bhattarcharya [3] proposed to estimate the saturation flow of an approach separately for through and right turning traffic in mixed traffic condition. This is valid only for the road width of 3.5–10.5 m. Susilo and Solihin [12] used regression model for the estimation of saturation flow according to the approach width. They have proposed two different equations for approach width of 3–8.9 m (Eq. 4) and 9–12 m (Eq. 5), respectively, as given below:

$$S = 600W_e \tag{4}$$

$$S = 500W_e + 400 \tag{5}$$

The main drawbacks for this model were lesser sample size and flows observed only during peak hour. In this analysis, width of left-turning lane was not considered and static PCU is taken.

Radhakrishnan and Mathew [7] proposed a saturation flow model based on the composition of traffic stream for Indian condition. They developed a saturation flow model based on dynamic PCUs by optimization using multiple linear regression. The proposed saturation flow model is:

$$S = \sum_{i=1}^{N} a_i P_i \tag{6}$$

$$S = 25.59P_{\mathrm{cr}} + 60.11P_{\mathrm{rw}} + 18.25P_{\mathrm{auto}} + 14.82P_{\mathrm{car}} \tag{7}$$

where a_i is the proportion of vehicle of 'i' type and P_i is the PCU of the vehicle type 'i'. Sample size is not statistically verified. Base saturation flow is considered as 1900 pcu/h/green which might not be true always. The proposed model resulted in lower error as compared to the conventional flow estimation techniques. Saha et al. [9] conducted a study on seventeen different intersections from various parts of India. In this, the saturation flow is dependent on approach width, and vehicle composition, expressed in vehicles per hour of green. Among these, regression model was proposed taking seven intersections together, and the data of last ten intersections were used to validate the applicability of the model in different cities. The model shows how saturation flow varies linearly with approach width as shown in Eq. (8).

$$S = 246.3w + 2020 \tag{8}$$

where S is the saturation flow rate in vehicles/h and w is the width in meters. This equation is valid only for the similar range of traffic compositions as observed in the field.

Rahman et al. [8] described the comparative analysis of saturation flow in Yokohama and Dhaka city by using conventional headway method. They have taken that the saturation flow starts after fourth vehicle in the queue by using ANOVA test for saturation flow region. Zhang and Chen [14] used the HCM model but did some modification to get the saturation flow. In their formulation, only heavy vehicle adjustment factor was introduced. Shao et al. [11] used method of least square. They have considered three category of vehicles and nine type of headways with all the possible combination with static analysis of time headway. Saturation flow rate was found out to be:

$$S_i = \frac{3600}{h_c} \tag{9}$$

$$h_c = \frac{\sum_{j=1}^{k} \sum_{i=1}^{m} h_{ij}}{m_k} \tag{10}$$

$$\mathrm{PCE} = \frac{h_{ci}}{h_b} \tag{11}$$

where h_c is the saturation headway and h_{ij} is the headway of vehicle type i standing at jth queue.

Alam et al. [1] used the HCM method to obtain the saturation flow rate in Mecca, Kingdom of Saudi Arabia, and introduced some adjustment factors to get accuracy in the result. They concluded that the capacity of a signalized intersection was much higher than the value prescribed in HCM. Anusha et al. [2] discussed on the adjustment factor for saturation flow measurement. They introduced a new adjustment factor for two-wheelers by incorporating the effect of two-wheelers on saturation flow rate. They concluded that the saturation flow measured using the modified

HCM equation is closer to the observed saturation flow values. The empirical equation proposed by them for the calculation of adjustment factor is given below in Eq. (12):

$$f_{\mathrm{tw}} = 0.378 - 0.8 * p_{\mathrm{tw}} + 0.004 * \frac{\mathrm{vol}}{w} \tag{12}$$

where P_{tw} is the percentage of two-wheeler, vol is the volume of traffic, and w is the width of road. In their study, saturation flow rate has been considered in veh/h, which is not true in case of Indian traffic.

4 Study Area, Data Collection, and Extraction

Two intersections have been selected in the city of Delhi among which one is a four-legged intersection and other is a T-intersection. These seven approaches have been studied in this paper. These intersections are present in a corridor of 1.1 km stretch. Data were collected in the month of October 2018. Four hours of data from 8 a.m. to 12 noon have been collected. The intersections are shown in Fig. 1 where one is a four-legged channelized intersection and other is a T-intersection.

The following data have been collected from the field to carry out the analysis with respect to estimation of saturation flow and PCU at signalized intersection:

- Classified traffic volume count per 5 s of green time.
- Geometry data, i.e., width of the approach, width of shoulder, number of lanes, median width, etc.
- Type of intersection, i.e., four-legged or T-intersection, channelized or non-channelized, etc.
- Signal phasing and timings.

Video camera has been installed at all the sites such that all approaches have clear visible of the approaching traffic. The camera has been installed at some elevation

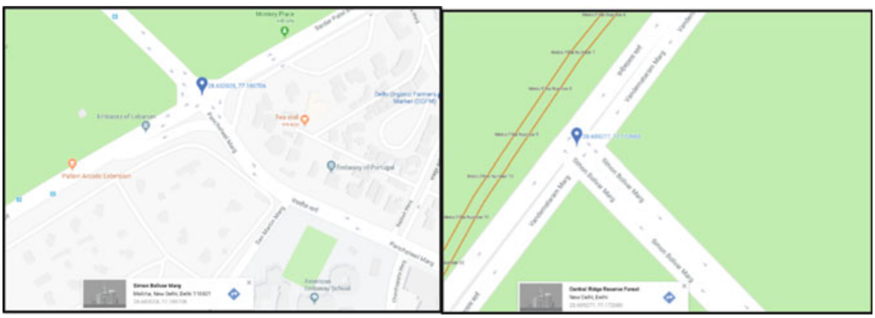


Fig. 1 Study area

early in the morning before traffic rushes into the location in such a way that it does not affect the driving behavior of drivers. Video data have been collected at all these approaches. The video camera covered the total approach of the study intersection by installing at some elevated place like the traffic signal pole or any nearby located tree. The required discharge data have been extracted from the video at the stop line. Classified discharge has been measured at the stop line by dividing the entire green interval into 5-s sampling interval for 30 cycles. The proportion of different category of vehicles for each cycle in all the approaches is calculated. The discharge profile has been analyzed by plotting graphs of discharge rate of vehicle versus the time taken by vehicle. The saturation flow is not directly related to the saturation headway, as practically the headway cannot be measured for Indian traffic condition. Hence, it is better to extract the volume count for whole width of the approach while discharging from the stop line, so that the field discharge rate can be plotted in a graph. Determination of saturation flow region and saturation flow is very difficult for mixed traffic condition, as vehicles do not follow lane discipline. So, it is recommended to find the saturation region by free hand drawing over the scattered plots in each graph to get a flat portion.

Procedure for data extraction has been stated below:

1. Note down the time and number of vehicles started moving before signal turns green.
2. Count the number of vehicles discharged in each 5-s interval when the green starts.
3. No. of intervals will be decided by dividing the total green time by 5-s interval.
4. Make a classified count of each category of vehicle crossing the stop line.
5. The total discharge in 5 s is determined per cycle of green.
6. The graphs are plotted having points with green interval in x-axis and discharge in y-axis.
7. The hard copy of the graphs is taken and then drawn freely by hand to get a flat portion of the curve for each cycle.
8. Then, the total no. of vehicles discharged in the flat portion is divided by the saturated green time in one hour which will give the saturation flow in vehicles per hour green at that intersection.
9. PCU value has been taken from the INDO-HCM.
10. The total discharge in each interval is multiplied by PCU for each category of vehicle, and the saturation flow is calculated in PCU/h of green.

5 Estimation of Saturation Flow

The collected data have been extracted using time-slice analysis which is the analysis by dividing the green interval into a number of intervals. An approach has been explained in details in this study. In every 5 s, discharge has been measured for different category of vehicles throughout the green time to see the discharge pattern.

Five categories of vehicles, viz. motorized two-wheelers (2W), three-wheelers (auto-rickshaws), cars, light commercial vehicles (LCV), heavy commercial vehicles (HCV), have been considered. Data of one cycle of this approach are given in Table 1. Similarly, thirty cycles of data have been extracted. The study of the behavior of vehicles has been considered only for that time of green where the vehicles move touching bumper to bumper without any disturbance in the uniformity of flow which can be considered as the ideal flow. Flow of different vehicle types has been extracted with 5-s interval for 30 cycles, and a graph has been drawn showing the discharge profile of all the vehicle type.

The behavior of each vehicle type of an approach is shown in Fig. 2. It has been observed from the graph that the proportion of cars is very high as compared to that of other vehicle types with LCV having the minimum proportion. The proportion of two-wheelers is highest at the start of green time as more number of vehicles sneak ahead at the start of green time or sometimes before green through the traffic because of their smaller size.

In case of homogeneous lane disciplined traffic, there occurs a situation where discharge of all the vehicle types reaches a constant peak flow during the green period, which is referred to as the saturation region, and the corresponding peak discharge

Table 1 Extracted classified 5-s data

Time	2W	3W	CAR	LCV	HCV	S.F/Veh
5	4	2	1	0	0	7
10	1	0	2	0	1	4
15	0	0	6	0	0	6
20	0	0	5	0	0	5
25	0.5	0	4.5	0	0	5
30	2.5	0	4.5	0	0	7
35	1	0	5	0	0	6
40	0	1	4.75	0	0	5.75
45	0	0.5	6.25	0	0	6.75
50	0	0.5	6	0	0	6.5
55	0	0	6	0	0	6
60	0	0	5	0	1	6
65	1	0	6	0	0	7
70	1	0	4	1	0	6
75	2	0	5	0	0	7
80	1	0	4	1	0	6
85	0	0	7	0	0	7
90	0	0	6	0	0	6
95	1	0	4	0	0	5

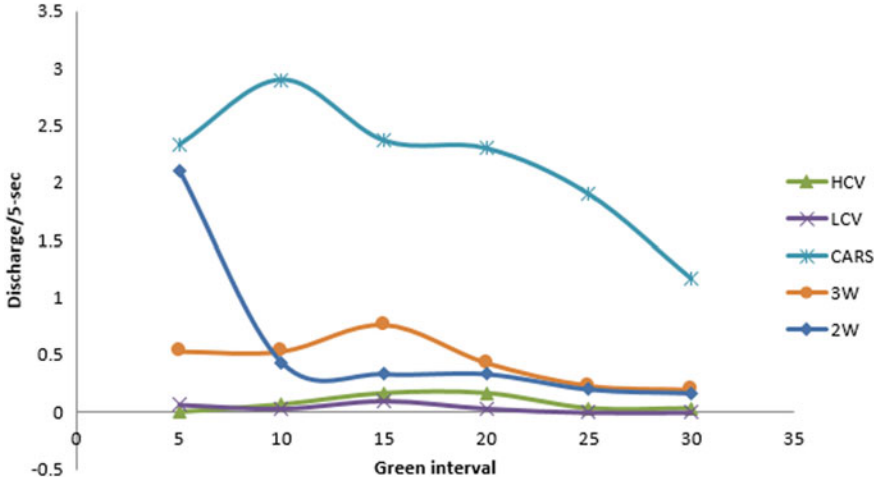


Fig. 2 Discharge pattern of different vehicle types

is the saturation flow. This is the condition of ideal saturation flow curve as shown in Fig. 3.

For the measurement of saturation flow at signalized intersection in case of heterogeneous traffic condition, there is a need of passenger car unit (PCU) to convert vehicles/h to PCU/h. As the discharge is extracted in vehicles per hour, so there is a necessity to convert the discharge to PCU/h to simplify the variations in static and dynamic characteristics of different vehicles into a single category (car). In this study, PCU of all the vehicle types has been taken from INDO-HCM [4].

The saturation flow of an approach depends mainly on the saturation region which shows that the vehicles are moving with their capacity in the green time. This saturation region is very difficult to get in case of low volume traffic and also in homogeneous traffic condition. The graphs are plotted with green interval in *x*-axis and

Fig. 3 Ideal saturation flow curve. *Source* Majhi [6]

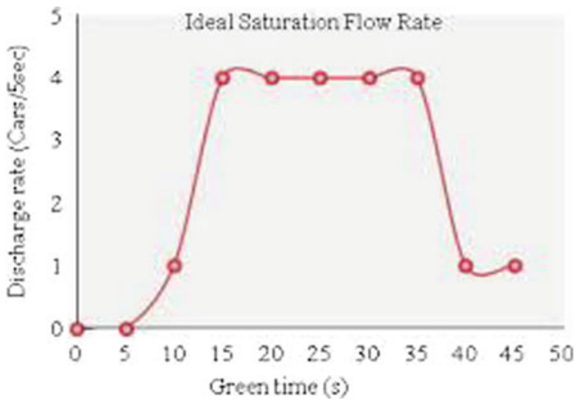


Table 2 Field saturation flows in PCU/5-s

PCU (INDO-HCM)	0.4	0.5	1	1.2	1.6	S.F/Veh	S.F/PCU
Time	2W	3W	CAR	LCV	HCV		
5	4	2	1	0	0	7	3.60
10	1	0	2	0	1	4	4.00
15	0	0	6	0	0	6	6.00
20	0	0	5	0	0	5	5.00
25	0.5	0	4.5	0	0	5	4.70
30	2.5	0	4.5	0	0	7	5.50
35	1	0	5	0	0	6	5.40
40	0	1	4.75	0	0	5.75	5.25
45	0	0.5	6.25	0	0	6.75	6.50
50	0	0.5	6	0	0	6.5	6.25
55	0	0	6	0	0	6	6.00
60	0	0	5	0	1	6	6.60
65	1	0	6	0	0	7	6.40
70	1	0	4	1	0	6	5.60
75	2	0	5	0	0	7	5.80
80	1	0	4	1	0	6	5.60
85	0	0	7	0	0	7	7.00
90	0	0	6	0	0	6	6.00
95	1	0	4	0	0	5	4.20

discharge (PCU/5-s) in *y*-axis. The plots are joined by drawing a curve manually in such a way that the discharge profile looks like an ideal saturation curve. A graph for Table 2 is shown in Fig. 4.

From the above figure, it can be observed that the saturation flow is approximately equal to 6.3 PCU/5-s. Similar graph has been drawn for all the thirty cycles of each approach.

Saturation flow for each approach has been decided in this methodology based on the optimization technique. The standard deviation of the saturation flows is optimized to obtain the PCU values for each category of vehicles mentioned in this study. Saturation flow of an approach is taken as the average of the saturation flow from thirty cycles. Similarly, saturation flow for seven approaches has been calculated. The saturation flow in PCU/5-s is converted into PCU/h.

PCU value of each category of vehicle obtained after optimization in this study is as follows:

Two-wheeler (2W)	0.34
Three-wheeler (3W)	0.63

(continued)

(continued)

Car	1
LCV	1.23
HCV	1.82

The saturation flow value obtained for different approach widths is given in Table 3.

The saturation flow of approaches has been plotted with the width of the respective approach as shown in Fig. 5.

It can be observed that the saturation flow rate increases as the width of the approach increases. The saturation flow model has been developed using these data, and an equation is proposed for saturation flow with width of approach as variable. The equation developed is as given below:

$$\mathrm{S.F} = 983.14 * W - 3863.9 \left(R^2 = 0.9\right) \tag{13}$$

where W is the width of the approach in meter and S.F is the saturation flow rate in PCU/h.

This equation is valid only for a small range of approach widths ranging from 7.6 to 10.3 m. This has been developed using the data collected from seven approaches. This equation gives the R^2 value of 0.9, which is significant.

6 Discussions and Conclusions

6.1 Comparison of the Estimated Saturation Flow with the Saturation Flow Proposed by Different Authors

The observed values of saturation flow vary with width of the approach. This means that the saturation flow depends on the width of the approach roads, and hence, various methods proposed by different authors that estimates the saturation flow based on width of the roads have been discussed in this section. Different methods proposed by different authors have been compared in Table 4 whose methods were discussed earlier.

In observed mixed traffic scenario coupled with non-lane disciplined behavior, with increased width, maneuverability of vehicles increases, which results in higher saturation flow.

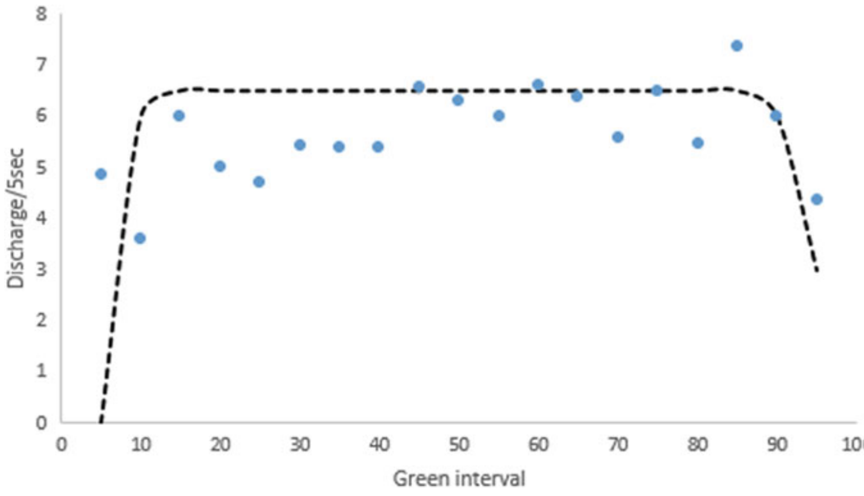


Fig. 4 Manually drawn saturation flow curve

Table 3 Estimated saturation flow

Intersection name	Approach	Width (m)	Saturation flow (PCU/h)	
			Per width	Per lane
SPM-PSL	Approach 1	7.6	3677.79	1693.72
	Approach 2	8.7	4702.99	1892.01
	Approach 3	8.6	4563.12	1857.08
	Approach 4	8.4	4309.82	1795.76
Ridge road	Approach 5	7.8	3747.63	1681.63
	Approach 6	10.2	6093.17	2090.79
	Approach 7	9.3	5453.26	2052.30

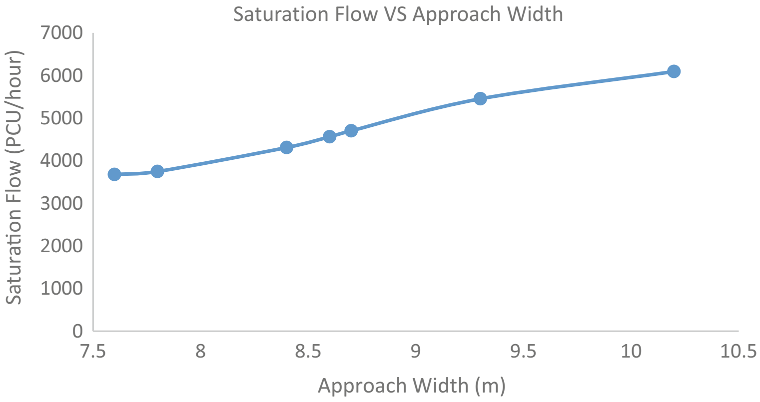


Fig. 5 Saturation flow versus approach width

Table 4 Comparison of saturation flow

Width (m)	Susilo and Solihin (PCU/h)	Saha (vehicles/h)	Webster's method (PCU/h)	INDO-HCM (PCU/h)	Proposed method (PCU/h)
7.6	4560	3892	3990	5199	3668
7.8	4680	3941	4095	5242	3748
8.4	5040	4089	4410	5342	4310
8.6	5160	4138	4515	5366	4563
8.7	5220	4163	4568	5377	4703
9.3	5050	4311	4883	5413	5453
10.2	5500	4532	5355	5386	6093

6.2 Conclusion

It can be observed from Table 4 that in case of mixed traffic condition for same width of approach, different methods yield different results. No adjustment factors have been taken in this comparison study. The saturation flow values by Susilo and Solihin [12] have been estimated by using the static PCU which is very difficult to get in case of mixed traffic condition and was also estimated with a small sample size. Hence, it can be observed from the table that as the width increases from 8.7 to 9.3 m, the saturation flow decreases which is not practically true. It can be observed from the above table that the saturation flow values estimated by INDO-HCM method also decrease as the width increases from 9.3 to 10.2 m. The saturation flows estimated by Saha method give the values in vehicles per hour which is easy to analyze and compute in case of mixed traffic condition, but the estimation is only possible by this method if the traffic has similar composition as that of the studied composition. Webster's and the estimated value in this study gives approximately the same value of saturation flow up to 8.6 m width road. After 8.3 m, the estimated value increases with high rate as compared to that of the Webster's method. Webster's method has not considered any other factors except width. As the saturation flow values are estimated directly from the field, hence, it can be taken as the accurate estimation of saturation flow.

The discharge in PCUs is found to be more significant in case of mixed traffic condition as there consists a wide variety of vehicle types than the discharge in terms of vehicle per hour.

It has been noticed that in most of the cycles, vehicles use to stop ahead of stop line or start moving before the start of green which is not permitted, but as this behavior is common, analysis has been done accordingly by starting the count only at the start of green time for vehicles standing at the stop line. In the present study, an attempt has been made to estimate for such type of traffic behavior, so that accurate estimation of saturation flow can be measured.

7 Future Scope

A small range of width has been taken for analysis in this study. Hence, the analysis could be extended to a wide range of width to obtain more accurate saturation flow model. These sites were so located that there was no effect from bus blockage, parking, etc., so it could be tried with sites having these disturbances, so that some adjustment factors could be suggested. The proportion of vehicles is considered in this study, so it could be considered for saturation flow model using regression model. A saturation flow model could be developed using the width of approach. Simulation modeling using VISSIM is to be done further, and the delay to be estimated to differentiate it is more from other models. Signal design and coordination could be done for intersections in corridor.

References

1. Alam Md., Osra KA, Albar HO, Zaran SZ (2011) Signalized intersection capacity adjustment factors for Makkah, Saudi Arabia. Can J Transp 4, Part 1:56–61
2. Anusha CS, Verma A, Kavitha G (2013) Effects of heterogeneity on saturation flow at signalized intersection in developing countries. J Transp Eng (ASCE) 448–557
3. Bhattarcharya PG, Bhattarcharya AK (1982) Observation and analysis of saturation flow through signalized intersection in Calcutta. Indian Highways 10(4):11–13
4. INDO-HCM (2017) Indian highway capacity manual. Council of Scientific and Industrial Research (CSIR), New Delhi, Dec 2017
5. Indian Roads Congress (IRC) (1994) Guidelines for design of at grade intersections in rural and urban areas. IRC special publication, No. 41, Indian Roads Congress, New Delhi, India
6. Majhi RC (2017) Field saturation flow measurement using dynamic passenger car unit under mixed traffic condition. Int J Traffic Transp Eng 7(4):475–486 (1994)
7. Radhakrishnan P, Mathew TV (2011) Passenger car units and saturation flow models for highly heterogeneous traffic at urban signalized intersections. Transportmetrica 7(2):141–162
8. Rahman MM, Ahmed SN, Hassan T (2005) Comparison of saturation flow rate at signalized intersection in Yokohama and Dhaka. In: Proceedings of the Eastern Asia Society for Transportation Studies, vol 5, pp 959–966
9. Saha AR, Chandra SA, Ghosh IN (2017) Saturation flow estimation at signalised intersections under mixed traffic conditions. In: 17th International conference on urban transport and the environment, Rome, Italy
10. Sarna AC, Malhotra SK (1967) Study of saturation flow at traffic light controlled intersection. Road research paper, 86
11. Shao CQ, Rong J, Liu XM (2011) Study on the saturation flow rate and its influence factors at signalized intersections in China. Procedia - Soc Behav Sci, 16504–16514.https://doi.org/10.1016/j.sbspro.2011.04.471
12. Susilo BH, Solihin Y (2011) Modification of saturation flow formula by width of road approach. Procedia - Soc Behav Sci, 16620–16629. https://doi.org/10.1016/j.sbspro.2011.04.482
13. Webster FV, Cobbe BM (1966) Traffic signals, technical paper 56. Road Research Laboratory, London
14. Zhang G, Chen J (2009) Study on saturation flow rates for signalized intersections. In: International conference on measuring technology and mechatronics automation, pp 598–601

Value Capture Finance: An Innovative Tool for Financing Urban Rail Transit Projects

Neeraj Sharma

1 Introduction

Most emerging cities of the twenty-first century in developing countries are poised for accelerated growth and have the kind of densities that are ideal for transit systems. Many are thus contemplating rail transit-driven sustainable urban mobility solutions to address a range of urbanization challenges. There is a growing recognition among cities across developed and developing nations that urban rail transit system is a key driver to maintain any city's economic competitiveness and helps catalyze livable and sustainable communities around station areas [1].

Indian cities, following the global trend, are now rapidly embracing urban metro rail transit systems in a big way. As these urban rail projects are highly capital intensive projects, most city governments face difficulty in funding for such transit systems and largely depend on grants from state or central governments. Innovative financing methods for implementation of such projects are being explored around the world. Value Capture Finance (VCF) has been emerged as an innovative and vital tool to finance the urban transit projects and raise the revenues to the implementing agencies.

Value capture as practiced widely in the world is based on the principle that private land and buildings benefit from public investments in infrastructure and policy decisions of governments (e.g., change of land use or FSI). Appropriate VCF tools can be deployed to capture a part of the increment in value of land and buildings. In turn, these can be used to fund projects being set up for the public by the Central/State Governments and ULBs.

In light of this, Ministry of Housing and Urban Affairs (MoHUA) has prepared a Value Capture Financing (VCF) policy framework for use by the Central Government Ministries and Departments/PSUs as well as the State Governments. MoHUA has

N. Sharma (✉)
RITES Ltd., 1, Sector 29, Gurgaon, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_2

issued instructions to include the VCF as an integral part of detailed project report (DPR) of all projects of the Central Government [2].

The present study is based on the case study of detailed project report for Nagpur Metro Phase II. The proposed metro corridors were passing through urban fringes with no major development activities resulting in low ridership along the corridors. The impact of this was the low fare box revenue which was not only sufficient for funding the project and improve the FIRR. There was an urgent need to identify sources to generate non-fare box revenue. In this regard, various VCF options were explored as a source to generate the non-fare box revenue for funding the project [3].

To start with, VCF sources which are in practice and used worldwide were studied and literature study of various international and national case studies on VCF implementation were reviewed. Based on these studies, the suitable sources of VCF that can be implemented in Nagpur were identified. The methodological framework was prepared for assessing the revenue from identified VCF sources, and the estimations were made based on the analysis from secondary data and primary surveys. The revenue figures were included in financial analysis to see the impact in improvement of FIRR of the project.

2 VCF Methods in Practice

Through capturing the increased value in urban land due to improved accessibility from building transport infrastructure, cities are now discovering a new way to improve their capacity to find the finance for building the infrastructure. A large number of diverse VCF tools are being used worldwide. The main types of VCF methods are given below [2].

2.1 Land Value Tax

Land value tax is a method of raising public revenue by means of an annual charge on the rental value of land. It is a more predictable way to tax based solely on the value of a parcel of land and not any associated buildings.

2.2 Fees for Change in Land Use

Land revenue codes provide for procedures to obtain permission for conversion of land use from agricultural to non-agricultural use.

2.3 Betterment Levy

This is a one-time upfront charge on the land value gain caused by public infrastructure investment. This occurs in two forms—revenue source for improvement schemes and for specific projects.

2.4 Development Charge (Impact Fee)

This is an area based and link the development charge to the market value of land by carrying out periodic revisions. By levying impact fee, upfront is collected while granting development permissions.

2.5 Transfer of Development Rights (TDR)

This is used for trading development rights and a way to compensate the property owners for loss in revenue on their properties.

2.6 Premium on Additional FSI/FAR

This is used to allow for additional development rights beyond the permissible limits in the State Town Planning Laws and Regulations.

2.7 Vacant Land Tax (VLT)

Under VLT, a tax is imposed on the registration value of the land if not used exclusively for agriculture purpose or is vacant without a building.

2.8 Tax Increment Financing (TIF)

In TIF, the incremental revenues from future increases in property tax or a surcharge on the existing property tax rate are ring-fenced for a defined period to finance some new investment in the designated area.

2.9 Land Pooling System

This is a form of land procurement where all land parcels in an area are pooled, converted into a layout, infrastructure developed, and a share of the land, in proportion to original ownership, returned as reconstituted parcels.

3 Literature Study for VCF Implementation

3.1 International Case Studies

CEPACS: Certificates of Additional Construction in Sao Paulo, Brazil
The project was aimed to employ innovative financing tools to raise funds for infrastructure projects and to make use of the sale of additional building rights to guide dense urban growth along transit corridors. A limited quantity of building rights was sold for large enough area—one CEPAC for each square meter of additional building right—through an electronic auction. The city holds periodic auctions for each area, gradually releasing additional FAR to maximize value capture. Those proposing to build over the basic FAR have to purchase CEPACs from the secondary market based on additional sq. m that the developer requires [2].

Tax Increment Financing in United States
Tax increment financing (TIF) has been acknowledged as a self-financing economic development tool and has been used for infrastructure and redevelopment projects. In general, TIF allows a government jurisdiction to take tax revenues derived from increase in property values within a prescribed development area and use those incremental tax revenues to fund the infrastructure and renewal projects [2].

Funding Opportunities for Urban Transport in Warsaw, Poland
Three value capture mechanisms have been explored to recover the capital costs incurred in the development of Warsaw metro system line extension and these were land tax, betterment levy, and public private partnership (PPP).

The implementation of land tax would capture the increase in property value but would require an advanced land registry system and new land evaluation system. Betterment levy to commercial properties has been used to raise the revenues. This has been a favored mechanism for many urban transport projects in the world [2].

3.2 Indian Case Studies

Introduction of Impact Fee as a Resource in Hyderabad

In Hyderabad, a 1 km stretch along both sides of 162 km eight lane expressway Outer Ring Road has been designated as a Growth Corridor (ORRGC). Hyderabad Municipal Corporation levies an impact fee in order to mitigate the impact of increased commercial construction activity along ORRGC and is collected at the time of granting building permissions. This impact fee is a one-time charge collected as a measure to pay for public infrastructure requirement [2].

Sale of Additional FSI and Transfer of Development Rights in Hyderabad

Government of Andhra Pradesh has permitted Municipal Corporation of Hyderabad (MCH) to grant additional FSI in lieu of land. Under this scheme, in case any land or building is affected in road widening activity, such an area is to be surrendered free of cost to sanctioning authority by land owner. Upon surrendering the affected area, the owner of the site is entitled to TDR or for purchasing additional FAR or is allowed to avail concessions in setbacks [2].

Value Capture Finance by Bangalore Metro

Government of Karnataka made amendments to Karnataka Town and Country Planning (KTCP) Act to enable capturing the land value through various methods like auctioning of sites, additional FAR, levy cess and surcharge, TDR, and additional property tax in the catchment area of their metro.

Under the KTCP act, levy of cess and surcharge at 5% of market value of land or building is charged to create a dedicated metro infrastructure fund. These revenues are shared among the metro transit agency BMRCL, BWSSB, and BDA. The floor area ratio (FAR) values were raised to 4 from 2.5 within 500 m distance along the metro rail corridor, and a cess of 10% of market value for residential and 20% market value for commercial on the additional FAR granted was levied. BMRCL also issues Transfer of Development Rights (TDR) to secure land for the metro rail alignment in lieu of compensation for the acquisition of land and private infrastructure bonds [2].

Value Capture Finance Practices in Gujarat

There are two successful examples of VCF practices in Gujarat. One example is land pooling through town planning scheme that enables the best redevelopment potential around stations. In such schemes, the government purchases agricultural plots on the city's periphery, constructs infrastructure, and then sells the now richer land back to the former owner. The farmer gives a portion of the new value, as a betterment fee, then keeps or sells the remainder.

The other example is the use of premium floor space index (FSI) tool in Ahmedabad. The Ahmedabad Urban Development Authority has proposed the premium FSI concept along the BRTS corridor also. Maximum FSI of 4 has been permitted till 200 m along BRTS corridors and proposed metro corridors [2].

4 Case Study of Nagpur Metro Phase II

Nagpur is an important urban center of Maharashtra, with the city being the administrative capital of the district. Nagpur Municipal Corporation (NMC) is spread over an area of 225.08 km^2 with a population of 24.48 lakh. The notified Nagpur Metropolitan Area (NMA) is house to 1.03 million population (excluding NMC area) with total area of 3567 km^2 and comprises of areas outside the Nagpur city [3].

Nagpur Metro Phase I consist of two corridors, i.e., North-South corridor from Automotive Square to Metro City and East-West corridor from Prajapati Nagar to Lokmanya Nagar with total length of about 42 km. At present, the North-South corridor is partially operational between Sitabuldi and Khapri. The ridership estimated for phase I corridors in the DPR prepared by DMRC in 2013 were 3.83 lakh, 4.59 lakh, and 5.63 lakh for 2021, 2031, and 2041, respectively.

The phase II of Nagpur Metro has been planned as extensions of the phase I corridors into the peri-urban areas and census towns situated along the periphery of the Nagpur city. Nagpur Metro Phase II consists of five corridors with total length of 48 km and 35 stations. The Ph II metro corridors are at the periphery of the city outside the municipal limits. The details of the corridors are given in Table 1, and map of the corridors is shown in Fig. 1.

4.1 *Ridership and Fare Box Revenue*

Various assumptions have been made for forecasting transport demand on proposed metro corridors for the years 2024, 2031, and 2041. The growth rate assumed for intercity passenger traffic was 3% per annum, while for inter and intra city goods traffic, 5% growth rate was taken. The special generator passenger traffic of bus terminals and railway stations was assumed to grow at 6%. It was estimated that the Phase II MRTS Corridors will cater to 2.89 lakh passenger trips per day by the year 2024 when the system gets operational. The ridership figures for key horizon years are given in Table 2.

Table 1 Details of Nagpur Metro Phase II corridors

S. No.	Corridor	Length (km)	Jurisdiction area
1	Automotive Square to Kanhan River	13	Kamptee Tehsil
2	Lokmanya Nagar to Hingna	6.7	Hingna Tehsil
3	Vasudev Nagar to Dattawadi	4.49	Nagpur Rural Tehsil
4	Prajapati Nagar to Transport Nagar	5.6	Kamptee Tehsil
5	Mihan to MIDC ESR	18.45	Nagpur Rural Tehsil
	Total	**48.24**	

Fig. 1 Map showing Nagpur Metro Phase I and Phase II corridors

Table 2 Expected metro ridership in horizon years

Year	Passenger trips per day (lakh)
2024	2.89
2031	3.39
2041	4.08

The estimated ridership was considerably low considering the long metro network length of 48 km. The low ridership was observed due to the fact that the proposed Ph II corridors were passing through urban fringes with no major development activities along them. With these ridership figures, the estimated fare box revenue in the operational year, i.e., 2024 came out to be Rs. 474 crore.

For the purpose of financial analysis of the project, fare box revenue was estimated for the overall period of 25 years as Rs. 41,925 crore, i.e., from 2024–25 to 2048–49. With these fare box revenue figures, the financial internal rate of return (FIRR) came out to be lower, i.e., 3.7%. The project was not financially viable with this low FIRR because 8% FIRR was desirable for the financial viability of the project.

Thus, it became necessary to explore the other sources of non-fare box revenue, so that the project revenues may be increased to improve the FIRR. With this context, it was decided that the various sources of Value Capture Finance (VCF) and other sources will be explored to generate the substantial non-fare box revenue of the project for improving the FIRR.

5 VCF Implementation in Nagpur

For identification of sources of VCF that may be implemented in Nagpur, the current practices of value capture financing in Maharashtra state were studied and extracted from Value Capture Finance Policy Framework issued by Ministry of Housing and Urban Affairs, Government of India. As per the policy framework, the Government of Maharashtra was following the VCF practices such as urban land tax, betterment levy, development charge, TDR, premium on additional FSI, and surcharge on property transactions [2].

Considering the applicability of VCF in Nagpur and the sources of VCF used in Nagpur Metro Ph I, suitable sources were identified for generation of non-fare box revenue for Ph II. Premium on sale of additional floor space index (FSI), additional surcharge on property transactions, increase in development charge, and property development were the different options considered as a part of VCF.

5.1 Study Methodology

The broad methodology followed for the assessment of VCF has been discussed in the subsequent paragraphs.

An assessment of the existing and proposed land use distribution as per development plan remarks and development control and promotion regulations (DCPR) was conducted as per Development Plan 2012–32 incorporated for Nagpur Metropolitan Region in January 2018. The said analysis was conducted for only the land parcels falling within the jurisdiction of transit-oriented development (TOD) influence zone (500 m on both sides of metro). The extents of the TOD influence area and special

DCPR provisions for promoting densification of urban development along the corridors were referred from the said DCPR, and the stipulations pertaining to FSI cap based on plot size and road frontage were considered to arrive at the total development potential from land component of the TOD Influence area [3].

For consideration of understanding the supply–demand dynamics pertaining to the TOD influence area, two scenarios were developed—one for estimating the total development potential from the supply side, i.e., assessment of net built-up potential as per the maximum FSI cap, applicable as per the sanctioned TOD Policy. The second approach was a demand-side assessment which was based on projecting achievable utilization of land for real estate development along the Metro Phase II Corridors. The same was enumerated on the basis of Y-o-Y population projection method, which was extrapolated to derive the net housing and relative commercial demand. This two-way analysis approach provided a range of comparative analysis between a net development potential within the metro corridors viz-a-viz a realistic absorption scenario on the basis of Y-o-Y absorption projection.

In addition, a detailed corridor-wise market assessment was conducted to encapsulate prevalent real estate dynamics along the proposed phase II metro corridors. A wide range of relevant asset classes were studied, such as residential, commercial (including retail, commercial office, hospitality) which are potential avenues of development that is touted to consume additional FSI. The market assessment exercise ascertained a quantum (in terms of pricing and absorption trends) to the potential demand that shall catalyze development/redevelopment along the corridors, and the inputs received from the same were incorporated to calculate the revenue contribution from the stipulated TOD influence zone.

The assessment of development potential was undertaken across the following modules.

Real Estate Market Assessment Exercise

A detailed real estate market study has been undertaken for the relevant micro-markets, to identify key growth vectors and prevalent market trends with respect to residential and commercial (IT/ITeS, non-IT office, retail, hospitality, logistics) segments. This market study formulates a basis for further analysis.

Assessment of Supply Potential and Estimation of Demand

Review of the TOD Policy and Subsequent DCPR to be Followed for the TOD Influence Area of Nagpur Metro Phase II

A detailed review of current Nagpur Land Use and Development Control Regulations for the proposed metro corridor was undertaken considering the following parameters:

- On ground survey along the phase II corridors
- Desktop review of the latest sanctioned DP 2012–32 of Nagpur Metropolitan Area
- Review and incorporation of modifications in DP pertaining to change in land use

- Review of the latest sanctioned DCPR for assessment of guidelines pertaining to delineation of TOD buffer zone for Nagpur Metro Phase II Corridors and eligibility criteria of plots (complete or part thereof) which fall within the stipulated TOD influence area for additional FSI
- Understanding the proposed land use across the phase II TOD influence and scrutiny of other relevant development bye laws.

Estimation of Consumed FSI by Existing Buildings and Built Versus Vacant/Under-Developed Analysis

A rough estimate of built versus vacant land parcels along the phase II of Nagpur Metro was conducted to summarize the existing land utilization and to ascertain the proportion of developed vis-à-vis vacant or under-developed properties.

- As a start point, a broad mapping exercise incorporating visual inspection of existing buildings and structures falling within the TOD Corridor was conducted for a broad estimation of typical number of floors, land area covered, and typical height of buildings. This provides a rough estimation of existing FSI utilization along the stipulated TOD corridors.
- The same was represented with an AutoCAD reconstruction of cadastre maps of plots falling within the TOD influence area. Additionally, with an overlay of satellite imagery on the reconstructed vectors, a rough estimation of vacant land parcels was also accumulated. As an outcome, area statements for all the five studied corridors were prepared enumerating the total land area under various designated land uses and total area of vacant land parcels.
- Further, eligibility of the respective plots for applicability of maximum permissible FSI (as per the TOD Policy) was derived through a sample-size analysis considering blocks of 40–80 plots at a distance of every 100–200 m along each corridor. The same also involved due consideration given to plots which were land-locked at the moment and therefore shall be subjected to a cutting of 30–35% land area toward development of roads.

Projecting Potential for FSI Utilization

In continuation to above sections, a review of all the previous version and the latest sanctioned TOD Notification for Nagpur Metro (Phase II) corridors was conducted. Following were the key highlights:

- Review of the TOD Notification enlists the FSI incentives that are extended toward potential developments in the earmarked TOD Influence area. The stipulated eligibility criterions for applicability of the incentives, such as size of a plot, width of the main access road, and tenement and population density, were also referred from the latest notified document.
- The evaluation of existing FSI utilization was based upon the on-ground survey, encompassing trends pertaining to existing demand and supply dynamics of Nagpur city's peri-urban fringes.
- Further, the said estimation of demand–supply gap, combined with summarization of upcoming developments across various asset classes, verifies the projected FSI utilization.

Assessment of Revenue Contribution from TOD

Based on findings of the market assessment exercise, thorough review of the existing and proposed land use plan and scrutiny of the Nagpur Metro TOD Notification, an estimation of maximum realizable revenue was made considering the following non-fare box avenues:

- **Supply-Side Assessment**: Total development potential on land parcels falling within TOD Influence area, considering various revenue yielding asset classes such as residential and commercial.
- **Demand-Side Assessment**: Projection of probable year-on-year demand trends (across asset classes) considering a regression analysis of organic population increment to be projected till 2048–49.

6 Nagpur TOD Policy

Nagpur TOD Policy was reviewed in the study for the assessment of VCF. The TOD Policy was notified in March 2018 by Urban Development Department, Government of Maharashtra. As per the notification, the definition of NMRC is stated as "It is the area falling within 500 m distance on either side of the Nagpur Metro Rail measured from its center line and also includes the area falling within 500 m distance from the longitudinal end of the last Metro Railway Station as shown on development plan [4]."

The salient features of TOD Policy has been discussed in subsequent paragraphs.

6.1 Maximum Permissible FSI

The maximum permissible total FSI in NMRC shall be 4.0 including the base permissible FSI, subject to condition that the additional FSI over and above the base permissible FSI shall be allowed within the overall limit of maximum permissible FSI.

The maximum permissible FSI shall be determined by satisfaction of both the criteria, viz minimum road width and plot area, simultaneously. The maximum permissible FSI shall be calculated on the gross plot area.

6.2 Payable Premium

The premium toward availing additional FSI, in case of development/redevelopment proposed in the NMRC, shall be decided on the basis of tenement density per hectare of the gross plot area, which may be calculated as

$$\text{Minimum number of tenements} = (\text{Gross plot area}) \times (\text{maximum proposed FSI for residential use}) \times (200 \text{ tenements per hectare})$$

The sale of additional FSI (within the NMRC Area) shall be permitted at a premium chargeable as 30% of annual statement of rates (ASR) values for developments which meets the above stipulated tenement density. The same is chargeable at 40% of ASR values for development which do not meet the stipulated tenement density.

6.3 Permissibility of Mixed Use in NMRC

Mixed use in the form of residential and commercial may be permissible on the residential plot in NMRC fronting on the road width of 12 m and above, and mixed use on plot (s) in commercial zone in NMC area shall be permissible as per principal DCR and the maximum permissible FSI under these regulations shall be allowed in payment of premium.

7 Real Estate Market Study

A detailed real estate market study was undertaken for the relevant micro-markets, to identify key growth vectors and prevalent market trends with respect to residential and commercial (IT/ITeS, non-IT office, retail, hospitality, logistics) segments. This market study formulated a basis for further analysis. Table 3 summarizes the real estate scenario pertaining to all the corridors [3].

8 Revenue Assessment from VCF Sources

8.1 Premium on Sale of Additional FSI

Regulations and guidelines pertinent to the latest sanctioned TOD Policy and other general by-laws, enumerated in the sanctioned DCPR (January 2018) of the Nagpur Metropolitan Area were referred to for computation of premium FSI area, pertinent to the NMRC/TOD Influence area of Metro Phase II corridors [5].

As per TOD Policy, the premium to be paid toward additional FSI was computed on the basis of satisfaction of a minimum tenement density per hectare of gross plot area.

Table 3 Real estate scenario along Nagpur Metro Phase II corridors

Corridor	Use	Typical unit sizes (in sq. ft)	Total supply (in sq. ft)/no. of units	Avg. capital price (in INR/sq. ft)	Avg. rental price (in INR/sq. ft/month)
Automotive Square–Kanhan River	Residential	2 BHK: 883–1130	120 (approx.)	3200–4395 (resale)	–
		3 BHK: 1062–1460			
	Commercial	Carpet area of 8000	28,000 sq. ft	5400–6050 (resale value)	58–72
	Residential	2 BHK: 800–1000	98	3250–3400	–
	Commercial	650–2250	20	5000	30
	Residential	2 BHK: 850–900	300	2200–2285	–
		3 BHK: 1350–1610 (duplex)			
		4 BHK: 2200 (duplex)			
Lokmanya Nagar–Hingna	Residential	2 BHK: 760–1092	600 (approx.)	3050–3650	–
		3 BHK: 1143–1200			
		4 BHK: 1375–1500			
	Residential	2 BHK: 850–1200	550	3400–4200 (resale value)	–
		3 BHK: 980–1290			
Vasudev Nagar–Dattawadi	Industrial	4000	Approximate supply of 1.2 Mn sq. ft of warehousing space	800–1050	12–14
		8000			
		12,000			
	Residential	2 BHK: 850–1175	800	5400–6800	–
		3 BHK: 1150–1375			
Prajapati Nagar–Transport Nagar	Industrial	8000	140,000 sq. ft	1250–1500	8–12
		12,000			
		20,000			
	Industrial	8000	0.8 Mn sq. ft	900–1250	9–10

(continued)

Table 3 (continued)

Corridor	Use	Typical unit sizes (in sq. ft)	Total supply (in sq. ft)/no. of units	Avg. capital price (in INR/sq. ft)	Avg. rental price (in INR/sq. ft/month)
		12,000			
		20,000			
MIHAN to MIDC ESR	Residential	1 BHK: 650–725 2 BHK: 950–1150 3 BHK: 1360–1700	Approx. 3000	3800–4180	–
	Residential	1 BHK: 650–700 2 BHK: 850–1150 3 BHK: 1260–1650	Approx. 5000	2400–2800	
	Non-agricultural	–	–	~18.5–25 Mn per acre	–
	commercial	1250–4800	200	NA	45–60
	Land	1–10 acres	NA	950–1250	10
	Industrial land	2000–40,000	NA	850–975	8
	Industrial land	1–10 acres	NA	~6.5–14.5 Mn per acre	NA

Developments/Redevelopment projects that fulfill the above parameter are liable to pay the premium at 30% of ASR value of the land. For projects which do not meet the minimum density criterion, the premium is charged at 40% of ASR value of land. For ease of computation, an average of the two figures, i.e., 35%, was assumed for consideration in the study.

As per Government of Maharashtra G.R. dated January 30, 2014, only 50% of the total revenue was accounted as revenue toward Phase II Metro Project.

8.2 *Additional Surcharge on Property Transactions*

The levy on property transactions is a standard share of the transaction/guideline value, whichever is more. The same apply to the transactions arising out of sale/purchase of transfer of land/property. The surcharge upon transactions of developed properties are charged at varying rates for different jurisdiction areas. In NMC jurisdiction, a 7.5% surcharge is levied which includes a 1% additional surcharge

Table 4 Collection of surcharge on property transactions in NMA

S. No.	Year	Surcharge received by NIT @0.50% of transaction value in NMA region (Rs. in crore)
1	2012–13	5.42
2	2013–14	5.97
3	2014–15	7.24
4	2015–16	6.66
5	2016–17	6.88

directed toward Nagpur Metro. This 1% additional surcharge has been notified to be levied in NMC area for phase I metro.

Nagpur Metro Phase II was touted for peripheral locations falling under the jurisdiction of Nagpur Metropolitan Region Development Authority (NMRDA) and parts of NMC/NIT. There was no notification for levying of additional 1% surcharge on the properties beyond NMC area. In similar lines with phase I, an additional 1% surcharge was proposed to be levied for Nagpur Metro Phase II also.

For the purpose of revenue assessment, the data of surcharge on property transactions was collected from Nagpur Improvement Trust (NIT) for the past 5 years, i.e., 2012–13 to 2016–17 (Table 4). To minimize the collection hurdles, it was proposed that the surcharge should be applied on whole of Nagpur Metropolitan Area (NMA) as done in Nagpur Municipal Area for Metro Phase I. It was also suggested that the present rate of 0.5% surcharge on property transactions in NMA as detailed under section 77 of NIT Act, 1936, may be increased to 1% as adopted in the case of Nagpur Metro Ph I [3].

It was observed that surcharge collection has grown at an average growth rate of 7% in past five years. To project the revenue collection, the growth rate has been moderated to 5%. The estimated revenue from surcharge on property transactions @1% is given in Table 5.

8.3 Increase in Development Charge

As per section 124B of Maharashtra Regional and Town Planning Act, 1966, projects involving development of land for residential or institutional use and also involving building or construction operations shall imply development charge fee as follows [6]:

- For land component—0.5% of rates of developed land as mentioned in the Stamp Duty Reckoner.
- For the built-up component—2.0% of the rates of developed land mentioned in the Stamp Duty Reckoner.

Table 5 Revenue from surcharge on property transactions in NMA area

Year	Revenue from 1% surcharge on property transactions in NMA area (Rs. in crore)
2024–25	20.33
2025–26	21.35
2026–27	22.41
2027–28	23.53
2028–29	24.71
2029–30	25.95
2030–31	27.24
2031–32	28.61
2032–33	30.04
2033–34	31.54
2034–35	33.11
2035–36	34.77
2036–37	36.51
2037–38	38.33
2038–39	40.25
2039–40	42.26
2040–41	44.38
2041–42	46.60
2042–43	48.93
2043–44	51.37
2044–45	53.94
2045–46	56.64
2046–47	59.47
2047–48	62.44
2048–49	65.57
Total	**970.28**

For Nagpur Metro Phase I, it was notified that 100% increase in development charge shall be levied and collected toward Nagpur Metro. In similar lines, it was proposed to levy 100% increase in development charge for Nagpur Metro Phase II, and the revenue so collected was accrued toward phase II metro.

9 VCF from Supply Side

The study of the supply-side dynamics included a comprehensive area assessment of the total land area encompassed within the earmarked 500 m buffer of the phase II metro corridors. To initiate the study, desktop analysis of the proposed land use plan of Nagpur Metropolitan Area and reconstruction of the proposed metro corridor on AutoCAD was conducted. This enabled an estimation of the approximate land area under the various land uses, which was further used to estimate the total built-up potential of the corridor as per the maximum FSI cap established as per the TOD Policy. A desktop analysis, combined with findings of the primary survey, was also incorporated to determine a built vs. vacant area matrix along the corridors to envisage a green-field development potential on vacant plots in the stipulated area [3].

9.1 Built-Up Area Estimation and Land Supply

For the estimation of built-up area, base FSI as per DCR 2018 was taken as 1.1 for residential and 1.25 for commercial. The estimation of built-up area along all the corridors for the period up to 2048–49 is given in Table 6.

9.2 Revenue Assessment

To calculate revenues from additional surcharge on property transactions, the Annual Statement of Rates (ASR) 2018–19 were referred pertaining to different locations (Talukas, Villages) falling within the five respective TOD influence corridors to assimilate a weighted average Ready Reckoner (RR) value. The weights were assigned as per the percentage of each land use zoning within the corridor. For example, if in a stretch of 1 ha, 50% area is residential, 35% is industrial and 15% commercial, the respective weights for the RR values for residential properties were adjusted within a sum of 50%. Similarly, for all the RR values for land with industrial property, the respective weights were adjusted within a sum of 35%.

However, for consideration of ASR or RR values for computation of premium toward additional purchasable FSI, only the ASR land values (for residential, commercial, and industrial properties) were considered. Hence, for each stretch, two sets of ASR weighted average values were deduced, one for computation of additional surcharge on property transactions (land + property) and other for computation of premium on additional FSI (land only—converted to non-agricultural use (residential/commercial)).

Annual escalation of 2% on Ready Reckoner (ASR) values was assumed. The Ready Reckoner values for each corridor are given in Table 7.

For estimation of revenue from sale of additional FSI, total built-up area potential of residential and commercial for each corridor as estimated in Table 7 has been taken. Then saleable built-up area has been assessed after deducting the base FSI from total built-up area potential. The estimated saleable area has been distributed in 25 year period. For calculating the premium, 35% of Ready Reckoner values was taken as described in section 8.1 above. The year-wise revenue has been estimated by

Table 6 Land and built-up area supply for residential and commercial development

S. No.	Corridor	Average premium FSI available[a]	Residential		Commercial	
			Land supply up to 2048–49 (sq. m)	Built-up area supply up to 2048–49 (sq. m)	Land supply up to 2048–49 (sq. m)	Built-up area supply up to 2048–49 (sq. m)
1	Automotive Square–Kanhan River	3.04	419,886	1,278,372	288,158	877,317
2	Lokmanya Nagar–Hingna	2.69	431,035	1,158,610	39,944	107,367
3	Vasudev Nagar–Dattawadi	3.20	27,597	88,327	–	–
4	Prajapati Nagar–Transport Nagar	3.32	273,806	909,093	157,258	522,129
5	MIHAN–MIDC ESR	3.20	3,128,610	10,387,655	689,301	2,202,371

[a] Average premium FSI is calculated by taking weighted average of plot sizes falling within the TOD influence areas and respective road frontages to the plots, thus providing an average value of eligible FSI

Table 7 Ready Reckoner values for each corridor

S. No.	Corridor name	RR value for additional surcharge on property transactions (INR/sq. m) (2018)	RR value for premium on additional FSI (INR/sq. m) (2018)
1	Automotive Square–Kanhan River	19,678	5010
2	Lokmanya Nagar–Hingna	16,072	8016
3	Vasudev Nagar–Dattawadi	11,261	7368
4	Prajapati Nagar–Transport Nagar	12,354	7830
5	MIHAN–MIDC ESR	18,275	7025

multiplying the saleable area with premium. This calculation has been done for each corridor separately to estimate the revenue from sale of additional FSI. The sample calculation for one of the corridors is explained in Table 8. Similar calculations have been done for other corridors and also for estimation of revenue from development charge. The estimated revenue from additional surcharge on property transactions is already presented in Table 5.

The total estimated revenues from sale of additional FSI, additional surcharge on property transactions, and development charges for supply side during 25 year period from 2024–25 to 2048–49 are given in Table 9.

10 VCF from Demand Side

10.1 Methodology for Estimation of Residential Demand

The estimation of real estate demand was established upon the population growth trends of the city. The Y-o-Y incremental trend in population figures was considered to calculate the housing demand for the increased population. To calculate the number of dwelling units required to accommodate the increased population, the Nagpur district census data was referred to observe the number of households in 2011 viz-a-viz the population of district at the time. An average household size of 4.47 has been assumed to calculate the number of household units required to accommodate the Y-O-Y increment in the population [3].

It was assumed that only a portion of the projected population in the analysis zones shall be residing in the areas falling within 500 m buffer of the metro alignment. Hence, a coefficient was assigned to total population along each corridor to calculate a tentative requirement of housing within the 500 m buffer zone.

Table 8 Revenue estimation from sale of additional FSI (Automotive Square–Kanhan River Corridor)

Year	Saleable area (sq. m)		RR value with 2% escalation (INR)	Premium @35% of RR	Revenue (area × premium) in Cr		
	Residential	Commercial			Residential	Commercial	Total
2024–25	39,437	24,977	5642	1975	7.79	4.93	12.72
2025–26	39,437	24,977	5755	2014	7.94	5.03	12.97
2026–27	39,437	24,977	5870	2055	8.10	5.13	13.23
2027–28	39,437	24,977	5987	2096	8.26	5.23	13.50
2028–29	39,437	24,977	6107	2138	8.43	5.34	13.77
2029–30	28,169	17,841	6229	2180	6.14	3.89	10.03
2030–31	28,169	17,841	6354	2224	6.26	3.97	10.23
2031–32	28,169	17,841	6481	2268	6.39	4.05	10.44
2032–33	28,169	17,841	6611	2314	6.52	4.13	10.65
2033–34	28,169	17,841	6743	2360	6.65	4.21	10.86
2034–35	16,901	10,704	6878	2407	4.07	2.58	6.65
2035–36	16,901	10,704	7015	2455	4.15	2.63	6.78
2036–37	16,901	10,704	7156	2504	4.23	2.68	6.91
2037–38	16,901	10,704	7299	2555	4.32	2.73	7.05
2038–39	16,901	10,704	7445	2606	4.40	2.79	7.19
2039–40	14,085	8920	7593	2658	3.74	2.37	6.11
2040–41	14,085	8920	7745	2711	3.82	2.42	6.24
2041–42	14,085	8920	7900	2765	3.89	2.47	6.36
2042–43	14,085	8920	8058	2820	3.97	2.52	6.49
2043–44	14,085	8920	8219	2877	4.05	2.57	6.62
2044–45	14,085	8920	8384	2934	4.13	2.62	6.75
2045–46	14,085	8920	8552	2993	4.22	2.67	6.89
2046–47	14,085	8920	8723	3053	4.30	2.72	7.02
2047–48	14,085	8920	8897	3114	4.39	2.78	7.16
2048–49	14,085	8920	9075	3176	4.47	2.83	7.31
Total	**563,383**	**356,812**			**134.50**	**85.08**	**219.58**

As per DPR of Nagpur Metro Ph II, the population was projected in the study area for 2021, 2031 and 2041 with growth rates of 1.4%, 1.2%, and 1.1%, respectively. The study area was divided in traffic analysis zones, and population projection had been done for each zone. Using the same growth rate for all influence areas in different horizon years, population has been projected for TOD influence areas of all five corridors in 2021, 2031, and 2041. From the decadal growth rates, compounded annual growth rate (CAGR) has been derived to project the Y-o-Y population from

Table 9 Supply-side revenue estimation (2024–2048) (INR in crore)

S. No.	Corridor	Sale of additional FSI	1% surcharge on property transactions in NMA	Increase in development charge
1	Automotive Square– Kanhan River	219.58	970.28	21.95
2	Lokmanya Nagar–Hingna	197.0		21.00
3	Vasudev Nagar–Dattawadi	14.40		1.35
4	Prajapati Nagar–Transport Nagar	246.38		26.03
5	MIHAN–MIDC ESR	474.05		60.72
	Total revenue	**1151.41**	**970.28**	**131.05**
	Revenue share to Maha Metro	**575.70[a]**	**970.28**	**131.05**

[a]50% of revenue from sale of additional FSI and 100% of revenue from 1% surcharge on property transactions and development charges will accrue to Maha Metro

base year of 2018 to horizon year of 2048–49. Based on CAGR, the net population for influence area of five corridors are projected in Table 10.

The dwelling units' requirement was estimated by dividing the population with the assumed average household size. To assume an average size of dwelling unit, a weighted average of typical DU size (based upon primary market research) is established at 84.10 m^2.

Table 10 Influence area-wise population projection

Corridor	2021	2031	2041
Automotive Square–Kanhan River	45,581	56,546	69,552
Lokmanya Nagar–Hingna	11,331	14,385	18,045
Vasudev Nagar–Dattawadi	9954	12,639	15,526
Prajapati Nagar–Transport Nagar	16,629	20,868	24,568
MIHAN–MIDC ESR	8234	12,598	16,836

10.2 Methodology for Estimation of Commercial Demand

For calculation of commercial demand, a model based on per capita commercial absorption coefficient of six major tier I cities of India, namely Mumbai, Delhi, Kolkata, Chennai, Bengaluru, and Hyderabad, was developed. A coefficient of 0.7 was achieved by assigning due weights to the coefficient of these cities depending upon their respective size and scale in comparison with Nagpur. Assuming that Nagpur is a tier II city, only 20% of the achieved per capita commercial demand coefficient was considered, i.e., 0.14 m^2 per person.

10.3 Corridor-Wise Estimation of Residential and Commercial Demand

Based on the typical dwelling size assumed, the residential built-up area requirement was calculated by direct multiplication with the annual additional housing requirement (no. of dwelling units). Similarly, commercial built-up area requirement was also calculated. The net weighted average eligible FSI for the corridor as assumed while assessing the supply-side dynamics was retained for the demand estimation.

The total land area demand and net built-up area requirement till 2048–49 for residential and commercial development on all corridors are presented in Tables 11 and 12, respectively.

Table 11 Land and built-up area requirement for residential development

S. No.	Corridor	Average weighted premium FSI	Land area demand in 2048–49 (sq. m)	Built-up area required in 2048–49 (sq. m)
1	Automotive Square–Kanhan River	3.04	209,185	636,876
2	Lokmanya Nagar–Hingna	2.69	66,188	177,911
3	Vasudev Nagar–Dattawadi	3.2	44,916	143,758
4	Prajapati Nagar–Transport Nagar	3.05	63,692	194,261
5	MIHAN–MIDC ESR	3.20	69,068	220,676

Table 12 Land and built-up area requirement for commercial development

S. No.	Corridor	Average weighted premium FSI	Land area demand in 2048–49 (sq. m)	Built-up area required in 2048–49 (sq. m)
1	Automotive Square–Kanhan River	3.04	72,108	219,537
2	Lokmanya Nagar–Hingna	2.69	20,526	55,174
3	Vasudev Nagar–Dattawadi	3.2	14,944	47,829
4	Prajapati Nagar–Transport Nagar	3.05	16,153	49,266
5	MIHAN–MIDC ESR	3.20	68,711	219,537

10.4 Revenue Assessment

For computation of revenue from demand side, the assumptions relating to the weighted average value of Annual Statement of Rates/Ready Reckoner were same as those adopted while computing the revenues from supply side. The total estimated revenues from sale of additional FSI, additional surcharge on property transactions, and development charges for demand side during 25 year period from 2024–25 to 2048–49 are given in Table 13.

It was observed that demand for absorbing the market is only 35% of the supply. Since demand-side estimation was a more realistic approach as compared to supply side, the revenues estimated from demand side were considered in the study. The total estimated revenue from all the three identified VCF sources worked out to be Rs. 1215 crore. The breakup of year-wise revenue is given in Table 14.

11 Estimation of Revenue from Property Development

Property development was also added as a part of VCF to further enhance the share of non-fare box revenue. Total 17 land parcels were identified by Maha Metro along Phase I corridors for property development. The expected revenue from the property development of Phase I will accrue to Maha Metro by the time Phase II becomes operational. Accordingly, expected revenue from property development of Phase I was taken as a part of non-fare box revenue of Phase II. Overall revenue potential from property development was estimated with a cumulative area of 428,201 m^2 [3].

The overall development suitability of each site was examined with respect to its location setting, land use pattern, and catchment potential. The revenue estimation

Table 13 Demand-side revenue estimation (2024–2048) (INR in crore)

S. No.	Corridor	Sale of additional FSI	1% surcharge on property transactions in NMA	Increase in development charge
1	Automotive Square–Kanhan River	140.51	970.28	13.88
2	Lokmanya Nagar–Hingna	56.77		6.14
3	Vasudev Nagar–Dattawadi	47.60		4.55
4	Prajapati Nagar–Transport Nagar	62.16		6.11
5	MIHAN–MIDC ESR	101.35		9.88
	Total revenue	**408.39**	**970.28**	**40.53**
	Revenue share to Maha Metro	**204.26**[a]	**970.28**	**40.53**

[a]50% of revenue from sale of additional FSI and 100% of revenue from 1% surcharge on property transactions and development charges will accrue to Maha Metro

was based on the prevailing annual statement of rates applicable to NIT to arrive at a base revenue expectation.

The available and potential built-up area (BUA) was computed at each site on the basis of applicable DCR norms. An indicative site development suitability was worked out, in order to comprehend the (1) development timeframe; (2) development phasing; and (3) phasing pattern, in terms of number of phases. The areas and development phasing of all identified land parcels are presented in Table 15. The likely revenue estimation from the identified land parcels over a horizon period of next 25–30 years is presented in Table 16.

12 Financial Analysis

The estimated revenue from VCF sources including premium on additional FSI, surcharge on property transactions, and development charge for 25 year period came out to be Rs. 1215 crore as given in Table 14, and revenue from property development worked out to be Rs. 5880 crore as per Table 16. The total revenue from VCF sources including property development for the period of 25 years worked out to be Rs. 7096 crore. In addition, revenue from advertisement had also been estimated in the DPR study [3].

Table 14 Year-wise breakup of revenue from VCF sources

Year	Revenue (Rs. in crores)			
	Premium on additional FSI	1% surcharge on property transactions in NMA	Increase in development charge	Total
2024–25	4.20	20.33	0.83	25.36
2025–26	5.48	21.35	1.08	27.91
2026–27	5.76	22.41	1.14	29.31
2027–28	6.05	23.53	1.20	30.78
2028–29	6.36	24.71	1.26	32.32
2029–30	6.69	25.95	1.32	33.95
2030–31	7.03	27.24	1.39	35.66
2031–32	7.40	28.61	1.46	37.46
2032–33	6.34	30.04	1.26	37.64
2033–34	6.61	31.54	1.31	39.46
2034–35	6.90	33.11	1.37	41.39
2035–36	7.20	34.77	1.43	43.4
2036–37	7.51	36.51	1.49	45.51
2037–38	7.84	38.33	1.56	47.73
2038–39	8.18	40.25	1.62	50.05
2039–40	8.53	42.26	1.70	52.49
2040–41	8.90	44.38	1.77	55.05
2041–42	9.29	46.60	1.85	57.73
2042–43	9.75	48.93	1.94	60.61
2043–44	10.18	51.37	2.02	63.58
2044–45	10.63	53.94	2.11	66.68
2045–46	11.10	56.64	2.21	69.94
2046–47	11.59	59.47	2.30	73.36
2047–48	12.10	62.44	2.40	76.95
2048–49	12.64	65.57	2.51	80.71
Total	**204.26**	**970.28**	**40.53**	**1215.03**

Financial analysis was done to see the improvement in FIRR due to revenue from VCF. For the purpose of financial analysis, apart from VCF project completion cost estimated in the DPR study was taken. Also operation and maintenance cost and fare box revenue estimated for 25 year period were used. The FIRR calculated without using revenue from VCF was 3.78% but after adding VCF sources, FIRR came out to be 8.23%. The calculation of FIRR for both the scenarios is presented in Tables 17 and 18.

Table 15 Identified land parcels for property development

S. No.	Land parcel location	Area (sq. m)	FSI	Built-up area (sq. m)	Phasing pattern	Phase 1	Phase 2	Phase 3	Phase 4
1	Zero Mile	12,829	4	51,312	2	2020	2026		
2	NMC Land near Sitabuldi	6189	4	24,756	1	2029			
3	Sitabuldi, TTMC	19,890	4	79,560	2	2020	2026		
4	Kasturchand Park	7220	4	28,880	1	2026			
5	Cotton Market A	70,425	4	281,699	4	2035	2041	2047	2050
6	Cotton Market B	5177	4	20,709	1	2029			
7	Prajapati Nagar	1875	1	1875	1	2035			
8	Hingna	44,601	1	44,601	3	2029	2033	2036	2039
9	Krazy Castle	26,193	NA						
10	Gaddi Godam	3644	4	14,576	1	2032			
11	Ajni (Neeri)	70,425	4	281,699	4	2021	2031	2041	
12	Ujjwal Nagar	6570	1	6570	1	2035			
13	Jaiprakash Nagar	68,085	1	68,085	4	2031	2034	2037	
14	Old Airport	2899	1	2899	1	2030			
15	New Airport	305	1	305	1	2030			
16	Khapri (MIHAN City)	937	1	937	1	2030			
17	Metro Depot (MIHAN Depot)	80,937	4	299,467	4	2023	2027	2030	
	Total	**428,201**	**1**	**674,538**					

Table 16 Estimated revenue from property development

Year	Potential revenue (INR crore)
2020	372.88
2021	0
2022	0
2023	129.12
2024	0
2025	0
2026	601.03
2027	156.95
2028	0
2029	265.05
2030	232.94
2031	342.36
2032	75.76
2033	86.93
2034	396.32
2035	559.23
2036	100.63
2037	458.79
2038	0
2039	116.49
2040	0
2041	567.82
2042	0
2043	0
2044	657.32
2045	0
2046	0
2047	760.93
Total	**5880.55**

13 Conclusion

The present study showcased that how the non-fare box revenue generated from VCF sources could help in the improvement of FIRR. The significant improvement in FIRR (from 3.78 to 8.23%) has been observed after using the VCF revenue in financial analysis. The project proved to be financially viable with this improved FIRR. It can be concluded from the present study that Value Capture Financing may be used to a great extent for generating the non-fare revenue which can be

Table 17 FIRR calculation without VCF (INR in crore)

Year	Completion cost	Land cost	Taxes	Total project completion cost	Fare box revenue	Revenue from advt.	Expected property development revenue	Revenue from VCF	Gross revenue	O&M cost	Addl. capital	Operational surplus
2019–2020	413	120	35	568	0							−568
2020–2021	1301	126	109	1536	0							−1536
2021–2022	1821		153	1974	0							−1974
2022–2023	2390		201	2591	0							−2591
2023–2024	2510		211	2721	0							−2721
2024–2025	1054		89	1143	474				474	267	0	−936
2025–2026					511				511	287	0	224
2026–2027					558				558	307	0	251
2027–2028					619				619	330	0	289
2028–2029					661				661	354	0	307
2029–2030					704				704	380	0	324
2030–2031					777				777	408	0	369
2031–2032					895				895	438	126	331
2032–2033					966				966	471	0	495
2033–2034					1085				1085	507	0	578
2034–2035					1168				1168	546	0	622
2035–2036					1274				1274	588	0	686
2036–2037					1386				1386	633	0	753
2037–2038					1519				1519	682	0	837

(continued)

Table 17 (continued)

Year	Completion cost	Land cost	Taxes	Total project completion cost	Fare box revenue	Revenue from advt.	Expected property development revenue	Revenue from VCF	Gross revenue	O&M cost	Addl. capital	Operational surplus
2038–2039					1658				1658	735	0	923
2039–2040					1801				1801	792	0	1009
2040–2041					1951				1951	853	0	1098
2041–2042					2159				2159	920	462	777
2042–2043					2355				2355	992	0	1363
2043–2044					2561				2561	1070	0	1491
2044–2045					2813				2813	1154	2639	−980
2045–2046					3052				3052	1245	0	1807
2046–2047					3348				3348	1344	0	2004
2047–2048					3656				3656	1450	0	2206
2048–2049					3974				3974	1565	0	2409
Total	**9489**	**246**	**798**	**10,533**	**41,925**				**41,925**	**18,318**	**3227**	**9847**
											IRR	**3.78%**

Table 18 FIRR calculation with VCF (INR in crore)

Year	Completion cost	Land cost	Taxes	Total project completion cost	Fare box revenue	Revenue from adv.	Expected property development revenue	Revenue from VCF	Gross revenue	O&M cost	Addl. capital	Operational surplus
2019–2020	413	120	35	568	0	0			0			−568
2020–2021	1301	126	109	1536	0	0	373		373			−1163
2021–2022	1821		153	1974	0	0			0			−1974
2022–2023	2390		201	2591	0	0			0			−2591
2023–2024	2510		211	2721	0	0	129		129			−2592
2024–2025	1054		89	1143	474	133	0	25	632	267	0	−778
2025–2026					511	141	0	28	680	287	0	393
2026–2027					558	151	601	29	1339	307	0	1032
2027–2028					619	160	157	31	967	330	0	637
2028–2029					661	170	0	32	863	354	0	509
2029–2030					704	181	265	34	1184	380	0	804
2030–2031					777	192	233	36	1238	408	0	830
2031–2032					895	213	342	37	1487	438	126	923
2032–2033					966	225	76	38	1305	471	0	834
2033–2034					1085	238	87	39	1449	507	0	942
2034–2035					1168	252	396	41	1857	546	0	1311
2035–2036					1274	261	559	43	2137	588	0	1549
2036–2037					1386	276	101	46	1809	633	0	1176
2037–2038					1519	292	459	48	2318	682	0	1636

(continued)

Table 18 (continued)

Year	Completion cost	Land cost	Taxes	Total project completion cost	Fare box revenue	Revenue from adv.	Expected property development revenue	Revenue from VCF	Gross revenue	O&M cost	Addl. capital	Operational surplus
2038–2039					1658	309	0	50	2017	735	0	1282
2039–2040					1801	326	116	52	2295	792	0	1503
2040–2041					1951	344	0	55	2350	853	0	1497
2041–2042					2159	400	568	58	3185	920	462	1803
2042–2043					2355	422	0	61	2838	992	0	1846
2043–2044					2561	446	0	64	3071	1070	0	2001
2044–2045					2813	470	657	67	4007	1154	2639	214
2045–2046					3052	493	0	70	3615	1245	0	2370
2046–2047					3348	520	0	73	3941	1344	0	2597
2047–2048					3656	548	761	77	5042	1450	0	3592
2048–2049					3974	577	0	81	4632	1565	0	3067
Total	**9489**	**246**	**798**	**10,533**	**41,925**	**7740**	**5880**	**1215**	**56,760**	**18,318**	**3227**	**24,682**
											IRR	**8.23%**

utilized for financing the project and becoming the project financially viable. VCF proved to be an innovative financing tool for urban rail transit projects. There is a huge potential that exists for a VCF based funding opportunity in India and other emerging nations and cities. If it can be tapped with a strategic approach from the planning to implementation, the cities involved will have significant development benefits. It is also equally important to select the appropriate VCF sources that can be implemented easily to raise the finances of the project.

References

1. Jillella SSK, Newman P (2016) Innovative value capture based rail transit financing: an opportunity for emerging transit cities of India. J Sustain Urbanization Plann Prog 1(1):56–65
2. Ministry of Housing and Urban Affairs, Government of India: Value Capture Finance Policy Framework, New Delhi (2017)
3. Detailed project report for extension of Nagpur metro rail phase-II. By RITES Ltd. on behalf of Maha Metro, Nagpur (2018)
4. Urban Development Department, Government of Maharashtra, Nagpur TOD Policy Notification No.TPS-2414/477/CR-248 (Part 1)/2014/UD-9, Mumbai (2018)
5. Urban Development Department, Government of Maharashtra, Development Control and Promotion Regulations for Nagpur Metropolitan Regional Development Authority (Nagpur Metropolitan Area Development Plan), Notification No.TPS-2416/CR-122(A)/2016/DCPR-NMA/UD-9, Mumbai (2018)
6. Law and Judiciary Department, Government of Maharashtra, The Maharashtra Regional and Town Planning Act, 1966 (2015)

Analysis of the Key Determinants of Electric Two-Wheeler Use in the Indian Context

Mallikarjun Patil and Bandhan Bandhu Mujumdar

1 Introduction

India has observed rapid urbanization in recent decades, and the urban population in India has increased from 28% in 2001 to 31.7% in 2011. The number of urban areas in India increased by 44.5% between 2001 (6498) and 2011 (9391) and expected to reach 60% by 2050 [1]. Having more than 50 cities with a population greater than 10 lakhs and 40 crores urban residents, these numbers are anticipated to double by the year 2050 [2]. A surge in urban population is due to the intense economic activities in and around the urban centers, and subsequently, these activities are associated with the considerable travel demand. As a result of the increased travel demand, the ownership of motorized vehicles in India has increased from 0.3 million in 1951 to 142 million in 2011 [3]; most of these vehicles are observed to be operating on the urban streets. The continuous operation of this significant number of vehicles is responsible for the increase in harmful Green House Gas (GHG) emissions. The increase in GHG emission levels leads to the deterioration of air quality, and the ill effects of the deteriorated air quality are mainly experienced among the urban residents in the form of various health disorders. Added to the poor air quality, these huge numbers of vehicles operating on a limited road space available affect the quality of travel with frequent congestion, leading to lower commuter productivity and the increase in the accident rates.

The restrictions to the use of motorized vehicles would be a possible solution to the problem of negative externalities of motorised vehicle use on the environment. However, the sudden prohibition of motorised vehicles, on the other hand, may have the adverse impact on the economy. Therefore, a well phased policy directives are

M. Patil · B. B. Mujumdar (✉)
Department of Civil Engineering, BITS Pilani, Hyderabad, India
e-mail: majumdar@hyderabad.bits-pilani.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_3

essential to gradually phase out the replacement of motorized vehicles with the environment friendly and sustainable vehicles. The replacement has to be planned in a phased manner. Initially, the vehicles with higher emission rates should be replaced on priority, as different vehicles emit a different amount of harmful compounds responsible for the degradation of air quality. For example, trucks and lorries contribute about 28.8% CO_2, 39% NO_x, 27.3% SO_2, and 25% PM, which constitute 25% of the total vehicular emission of India. Similarly, two-wheelers constitute a significant source of CO (23.7%), CH_4 (46.4%), and HC (64.2%), and buses are emitting NO_x (30.7%) and PM (20.5%) [4]. Added to the pollution effect, the usage and availability of crude oils are critical issues that need immediate attention from the global community, as per the statistics available on crude oil, under the current consumption rate the crude oil is available for the next 53.3 years only [5].

Considering the above-discussed negative externalities, shifting the transportation sector gradually from crude oil to alternate energy sources is essential. Among the various alternative energy sources, electrical energy seems promising due to its easier availability than remaining alternative energy sources. Hence, shifting to electric mode from motorized mode can be considered one of the critical solutions for the energy crisis in the coming decades.

If we consider the mode-specific distribution of the vehicle users among all types of vehicles, two-wheelers are very popular among low- and middle-income Indian user groups. The popularity is due to the lower purchase cost compared to the other personal modes. The two-wheelers are broadly classified into two types—namely, motorcycle and bicycle. The motorcycle is the motorized mode, which uses petrol as a fuel. In contrast, the bicycle is a non-motorized mode, assisted by the pedaling mechanism. As per the general observations, it is observed that the proportions of bicycle users are lower than compared to motorcycle users, indicating the user's lower willingness for pedaling. Hence, it can be anticipated that if the cycling mechanism is made easier by providing additional assistance, then the share of bicycle users might increase. Because, as discussed earlier, the replacement of motorized modes with non-motorized modes can be considered a sustainable solution. Conversely, the reduction in bicycle usage is mainly associated with the increase in the usage of motorcycles. Statistics indicate that the total number of registered vehicles in India increased at a compound annual growth rate (CAGR) of 9.9% between 2001 and 2011. In particular, the number of motorcycles has increased at a rate of 12% per year during the last two decades [6]. Hence, the concern on the degradation of the air quality can be minimized to some extent, at least if the motorcycles are replaced with electric motorcycles. Apart from the electric motorcycles, the promotion of electric bicycles is also necessary to improve the share of bicycle mode by capturing the potential users who are willing to bicycle with reduced efforts. Thus increasing the overall share of non-motorized two-wheelers. Hence, both the objectives of reducing the pollution from motorcycles and reducing the effort for bicycling can be achieved by shifting to electric motorcycles and electric bicycles.

Suitable investigations on intentions to use electric two-wheelers could improve the overall quality of urban life. In this regard, the Government of India is aggressively pursuing the strategy to introduce electric vehicles, including electric two-wheelers.

Faster Adoption and Manufacture of Hybrid and Electric vehicles (FAME) is introduced to promote the manufacture and use of electric vehicles. In FAME policy, incentives are provided for both manufacturer and user. For manufacturers, it is intended to manufacture electric vehicles with desired performance characteristics, and for the users, the incentives are provided to encourage shifting from motorized to electric mode. According to the phase-II of FAME policy, which is effective from 1st April 2019 to a period of next three years, it is intended to provide an incentive of 20,000 INR for 10 lakh registered electric two-wheelers, with a battery capacity of 2KWH which accounts for nearly 2000 crores INR for a span of 3 years [7].

Apart from the policies and decisions of the government, planning for the promotion of electric vehicles as competitive modes of motorized vehicles may have a larger social and environmental impact, especially in urban centers. Hence, there is a necessity to concentrate on the studies related to assessing the consensus on how and under what conditions one may shift from motorized mode to electric mode. Several studies have investigated the modal shift characteristics of motorized car users, but studies specific to electric two-wheelers are limited. Hence, this research was aimed to identify the important attributes that would influence two-wheeler users to shift from motorized to electric mode. In this study, electric two-wheelers were classified into Electric Bicycles (EB) and Electric Motorcycles (EM). The classification considers the differences in operational characteristics of EB and EM. In this study, the following broad research objectives were set. Firstly, identify key attributes influencing electric two-wheelers in the Indian context based on a sound literature review. Secondly, prioritize the identified determinants using AHP methodology, an advanced Multi-Criteria Decision-Making (MCDM) approach. In order to identify the various attributes associated with the electric two-wheelers use, a detailed literature review is taken up and is discussed in the following section.

2 Literature Review

The existing literature was revived to find the attributes related to with the use of electric two-wheelers. The list of the attributes identified from the literature has been presented in Table 1. The attributes were further examined, and only those determined to be appropriate for this study were selected for further analysis. In addition to the attributes, a set of significant motivators and deterrents for the use of electric two-wheelers were identified and are presented in Table 2.

From the literature review, it was observed that there are various aspects of studies available on electric two-wheelers. However, there are no studies available that have specifically concentrated on identifying the key attributes of electric two-wheeler use. The key attributes will be helpful for identification of important attributes that have maximum influence on the electric two-wheeler use. As there are no studies available to identify the key attributes, this study has been conducted to identify the key attributes of electric two-wheeler users, specifically in the Indian context.

Table 1 Attributes identified from the literature review

Authors	The attributes considered in the study
Simsekoglu and Klöckner [8]	**Perceived benefits**: Mobility benefits**,** symbolic benefits**,** health, and other benefits
Zhu et al. [9]	**Socioeconomic information**: Age, gender, living time in Macau, monthly income, educational level, number of family persons, motorcycle number **Environmental issues**: Air pollution, noise issue, potential pollution from motorcycle **Knowledge of EM**: Environment benefits, low charges cost, high price, poor battery lifetime, immature technology, high safety, low maintenance cost **Buying EM**: Sale price, charging fee, repair fee, battery lifetime and cost, battery endurance, tax incentives
Elliot et al. [10]	**E two-wheeler use**: Age, gender, income, bike use, car use, fitness
Lin et al. [11]	**Trip purpose**: Commute, pick up children, leisure, visit friends, go to school, go to metro, go shopping, business **Previous modes**: Bicycle, bus, walking, motorcycle, metro, private car, taxi, coach, tricycle **Reason for E-bike use**: Effort saving, timesaving, low-cost operation, high accessibility, timesaving, environment-friendly, low purchase cost, health **Attitudes for E-bike adoption**: Sense of freedom, practical usage, relaxing, greener, fashion, part of your community
Fyhri et al. [12]	**Psychological**: Intentions, attitudes, personal and social norms, and habit **Barriers**: Not good enough cycling infrastructure; it feels unsafe; (the possibility of) bad weather; too physically demanding; steep hills; need to bring children or goods; need to use the car for work; does not want to sweat/there is no shower at work; no safe parking options; poor health **Behavioral**: Timesaving, comfort, low risk of accidents, gives the right image, gives the freedom, mentally relaxing, monetary savings, and improves fitness
Kroesen [13]	**Sociodemographic and household characteristics**: Gender, age, level of education, primary occupation, household income, license ownership, residential density, number of household members **Vehicle ownership**: E-bike ownership, bicycle ownership, car ownership. **Travel behavior**: E-bike use, bicycle use, car use (driver), car use (passenger), public transport use **Exogenous**: Gender, age, level of education, primary education, net household income, license ownership, residential density, number of household members **Endogenous**: Distance traveled by, vehicle ownership

Table 2 Motivators and deterrents of electric two-wheeler use

E-Bicycle		E-Motorcycle	
Motivators	Deterrents	Motivators	Deterrents
Health benefits	Higher purchase cost	Health benefits	Higher purchase cost
Ease of operation	Attitude toward bicycle users	Ease of operation	Attitude toward bicycle users
Higher speed	Charging infrastructure	Higher speed	Charging infrastructure
Long range	Safety of bicycles	Long range	Safety of bicycles

The attributes related to electric two-wheelers identified from the literature review were finalized in relevance to the Indian context and were divided into different categories. Because of the difference in operational and functional characteristics, there is a slight difference among the classification of attributes considered under electric motorcycles and electric bicycles. Therefore, the attributes related to electric bicycles have been classified under six categories. At the same time, attributes related to electric motorcycles have been classified under seven different categories. The only difference between classifications of attributes is the inclusion of the environment category under electric motorcycle. Because it is a well-known thing that as the bicycle is a nonpolluting vehicle and doesn't have the least effect on the environment. Hence, the attributes under the environment category are considered only for electric motorcycles. The attributes and classification under electric bicycle and electric motorcycle are presented in Tables 3 and 4.

Table 3 List of attributes

Category	Attributes
Economic related	(i) Purchase cost (ii) Operation cost (iii) Maintenance cost (iv) Resale value (v) Monetary savings
Vehicular related	(i) Top speed (ii) Range (iii) Acceleration (iv) Weight (v) Passenger and the cargo carrying (vi) Ease of pedaling
Environmental related	(i) Tailpipe emission (ii) Total production emission (iii) Noise pollution
Battery related	(i) Battery life (ii) Battery endurance rating (iii) Battery recycling (iv) Charging duration
User related	(i) Safety (ii) Health benefits (iii) Ride quality (iv) Mentally relaxing (v) Operating environment
Transport infrastructure related	(i) Width of the carriageway (ii) Gradient of the carriageway (iii) Carriageway condition (iv) Intersection type (v) Charging infrastructure
Commute quality related	(i) Flexibility (ii) Travel time reliability (iii) Social aspects (iv) Cost compared to other alternatives (v) Style

Table 4 Description of the scale

Scale	1	3	5	7	9
Description	Equally important	Moderately important	Strongly important	Very strongly important	Extremely important

3 Prioritization of Attributes

The attributes identified from the literature review were evaluated for their relevance to study. In this regard, a total of 32 attributes identified from the literature review were divided into different categories, and the Multi Criteria Decision Making (MCDM) techniques was applied to prioritize the attributes within each category. It was considered that these attributes would influence the perception of users toward using electric two-wheelers. The next step in the study was to prioritize the attributes among their respective categories to identify the most influential attribute that would motivate the users to shift toward electric mode. The methodology adopted in the prioritization of attributes was classified into three stages. Namely, attributes identification, data collection, and analysis using appropriate Multi-Criteria Decision-Making (MCDM) technique for prioritizing the attributes. Analytical Hierarchy Process (AHP), an extensively used MCDM technique, was adopted for prioritizing the attributes. AHP is a pairwise comparison method, which can be employed for deriving the relative weights of a set of attributes. The following section briefly presents the data collection process followed by AHP and interpretation of results.

3.1 Data Collection

In this study, experts' perception survey was conducted with a questionnaire designed on a pairwise comparison framework. The perceptions were then evaluated with the Analytical Hierarchy Process (AHP). For effective policymaking, the inputs from the experts could be instrumental, as they would be better equipped to understand the needs of electric mobility, which is still a relatively new mode in India. Experts were identified from different fields of transportation; experts were identified as academicians, researchers, and Industry personals. An AHP questionnaire comprising different sets of pairwise comparisons of attributes presented to the experts, Saaty's scale, was used to design the questionnaires. A standard Saaty's scale of {1, 3, 5, 7, and 9} was used for the study. Table 4 describes the values adopted from Saaty's scale.

The questionnaire was sent through E-mail to 20 identified experts from the transportation field; the sample size limited numbers for avoiding the complexity in the analysis. As per Cheng and Li [14], the AHP method might be impractical for a survey with a large sample. AHP analysis with a large sample size may lead to high degrees of inconsistency [15]. Hence, in this regard, the sample size in this study was

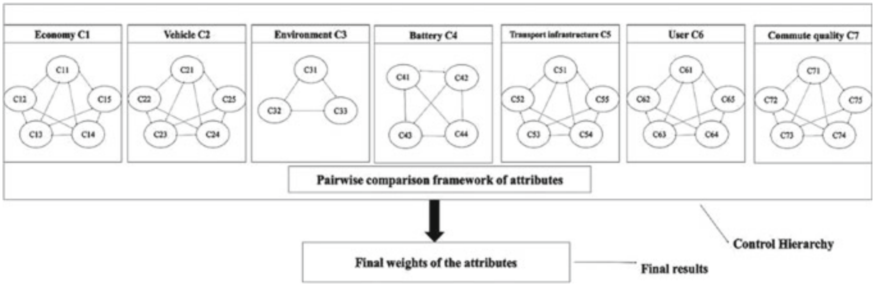


Fig. 1 Control hierarchy: pairwise comparison framework

limited to 20 numbers. Among those 20 experts, only 15 experts responded, some of the responses were received back through E-mails, and the remaining responses were collected through personal discussions. The responses from those 15 experts were used for AHP analysis. A detailed explanation regarding AHP analytical steps is discussed in the subsequent section. As per the standard procedure of the AHP analysis, the collected responses were checked for consistency. It was found that among those 15 responses, only 11 responses were found to be consistent. The consistent responses were identified from the CR value. As per AHP analysis, if the CR ≤ 0.1, then the responses are considered consistent. As a further step, only those consistent responses were used to calculate the final relative weights of the attributes. The total responses were reduced to 3 from Academicians, 4 from researchers, and 4 from Industry personals. Figure 1 represents the distribution of the final accepted responses from different experts.

3.2 Analytical Hierarchy Process (AHP) Analysis

The data are analyzed based on the opinions collected in a pairwise comparison framework, using Analytical Hierarchy Process (AHP), a widely used MCMD technique. AHP can accommodate both qualitative and quantitative measures on the same platform. The other main advantage of AHP is that it can check the consistency of the responder. AHP is used in most studies, which involves decision-making to decide the best among the various alternatives available. AHP is one of the widely MCDM techniques in a wide range of the fields. For example, Patil et al. [16] applied AHP to analyze the experts' perceptions. Similarly, AHP was applied to find the best alternate route in terms of the least-cost path in Thailand [17]. Similarly, AHP was used in finding the best alternative fuel available for land transport vehicles in Singapore [18]. Hence, by referring to those previous studies and assessing the suitability for this study, the AHP method was finalized. For details on AHP analysis, one may refer to Saaty [19]. The stepwise analysis adopted for calculating the relative weights of attributes is discussed in the subsequent section. The consolidated response matrix was created by calculating the geometric mean of all individual responses. As per, the geometric mean of individual opinions is calculated and entered in the final judgmental matrix to determine the relative weights of the attributes. A brief description of the analytical steps followed to calculate the relative weights of the attributes is as follow:

The normalized matrix for the consolidated matrix was created to calculate the Eigenvectors; it is shown as a control hierarchy in Fig. 1, representing the pairwise comparison of attributes.

1. The pairwise comparison framework is presented in the form of a matrix. The matrix was normalized by dividing the cell value by the column total.
2. The normalized matrix was multiplied with it to get an almost equal Eigenvector in two consecutive iterations.
3. Principal Eigenvalue (λ_{max}), Consistent ratio (CR), and Consistency index (CI) were calculated.
4. CR measures the consistency of the response. As per Saaty [19], the CR value should always be less than 0.1 ($0 \leq CR \leq 0.1$). Otherwise, the analysis should be repeated by excluding the inconsistent individual responses.
5. The final Eigenvectors of each criterion are multiplied with the respective Eigenvectors to get the final weights.
6. After getting the final weights, the analysis is continued for the remaining clusters.
7. The final priority order of attributes in each cluster is obtained by arranging the attributes in a descending order based on the relative weights.

3.3 Results

Prioritization is a method of identifying the most important attribute; in this study, the attributes related to Electric two-wheeler use are prioritized based on the relative weights obtained from the AHP analysis. Thus, the most important attribute is the one with the highest relative weight under their respective categories. However, as AHP is a survey-based technique, the consistency of the ranks provided by respondents needs to be thoroughly examined, as the responses might vary over time. Hence, in the following section, a consistency check of the ranks is presented. The priority order of attributes, along with the relative weights obtained, are presented in Table 5.

4 Consistency Test

In order to validate the results, a consistency test was carried out. Checking the consistency of the weights of attributes obtained from 11 experts is important to verify if the attributes were ranked consistently. The nonparametric repeated measures ANOVA

Table 5 Relative weights of attributes

Category	E-Bicycle			E-Motorcycle		
	Attribute	Weights	Order	Attribute	Weights	Order
Economy				Purchase cost	32	1
	Purchase cost	53	1	Monetary savings	22	2
	Maintenance cost	23	2	Operation cost	18	3
	Resale value	24	3	Maintenance cost	15	4
				Resale value	13	5
Vehicle	Top speed	20	1	Range	28	1
	Range	23	2	Cargo carrying	25	2
	Weight	13	3	Weight	18	3
	Cargo carrying	9	4	Top speed	16	4
	Ease of pedaling	35	5	Acceleration	13	5
Environment				Tailpipe emission	60	1
				Production emission	25	2
				Noise pollution	15	3
Battery	Battery life	36	1	Battery life	36	1
	Endurance rating	20	2	Charging duration	23	2
	Battery recycling	20	3	Endurance rating	20	3
	Charging duration	23	4	Battery recycling	20	4
User	Safety	27	1	Safety	27	1
	Health benefits	26	2	Health benefits	26	2
	Ride quality	20	3	Ride quality	20	3
	Mentally relaxing	15	4	Mentally relaxing	15	4
	Operating environment	12	5	Operating environment	12	5
Infrastructure	Width of carriageway	18	1	Charging infrastructure	31	1
	Gradient of carriageway	17	2	Intersection type	21	2

(continued)

Table 5 (continued)

Category	E-Bicycle			E-Motorcycle		
	Attribute	Weights	Order	Attribute	Weights	Order
	Carriageway condition	17	3	Width of carriageway	18	3
	Intersection type	20	4	Carriageway condition	17	4
	Charging infrastructure	29	5	Gradient of carriageway	13	5
Commute quality	Flexibility	16	1	Travel time reliability	33	1
	Travel time reliability	30	2	Social aspects	19	2
	Social aspects	22	3	Flexibility	17	3
	Cost compared to other	14	4	Style	17	4
	Style	16	5	Cost compared to other	14	5

test called the Friedman test was used to verify the consistency. Friedman test is based on the method of rank-transformation, in which the inputs are substituted in terms of ranks instead of the absolute values. These Friedman tests do not require that the distributions are normal, but these tests assume that the data points are not dependent on each other and that each group has approximately the same variance. Therefore, instead of assessing the difference of means across groups, this test will calculate the differences in median values across the groups. They have been provided with tables for finding out the distribution [20]. Based on the above-explained points, the Friedman test was carried out to check the consistency of the weights of the attributes. The results obtained from the Friedman test are discussed in the subsequent section.

4.1 Friedman Test

In this stage of the study, the ranks as provided by the experts are compared for assessment of consistency across ranks. If the ranks across experts are consistent, that would indicate that these derived ranks can be generalized and can be used for further policymaking. As both AHP is a survey-based technique, the consistency of the expert responses needs to be assessed as the responses might vary temporally. Hence, a comparison of rankings across experts is conducted through a consistency check among the responses. Friedman test was used to compare the derived ranks across the expert responses performed on rank-transformed data. The null hypothesis in this test states that "There are no statistically significant differences between the ranks for different attributes as perceived by the experts" was tested at a 5%

Table 6 Friedman test results

Category	χ^2 value	Q-value	P-value	Remarks
Economy	3.9	2.6	0.01	The null hypothesis cannot be rejected
Vehicle	3.9	1.4	0.01	The null hypothesis cannot be rejected
Environment	3.9	0.8	0.01	The null hypothesis cannot be rejected
Battery	3.9	1.7	0.01	The null hypothesis cannot be rejected
User	3.9	3.5	0.05	The null hypothesis cannot be rejected
Infrastructure	3.9	1.8	0.01	The null hypothesis cannot be rejected
Commute quality	3.9	1.5	0.01	The null hypothesis cannot be rejected

χ^2: Chi-square value for $(n - k)$ degrees of freedom (n: number of attributes, k number of respondents); Q-value: Friedman test statistic; P-value: level of significance

significance level. If the groups have a difference of opinions, then it is understood that the obtained results would not reflect the true priority order of the attributes derived from the AHP and ANP techniques. Table 6 presents the Friedman test result.

Results in Table 6 indicate that all estimated Friedman test statistic for every attribute cluster is associated with a p-value less than 0.05. This result indicates that there is no statistically significant difference among ranks as perceived by experts, or the ranking is consistent among transportation experts. This result strengthens the future application of the derived rankings for policy development and future analysis purpose.

5 Discussions

The aim of the study was to identify the most important attributes related to the user perception of electric two-wheeler use; the importance of attributes was decided based on the relative weights obtained from the AHP analysis. Based on the results obtained, the following conclusions can be drawn.

The results from this study suggest that purchase cost is found to be an important economic-related attribute, indicating the importance of minimizing the purchase cost associated with the purchase cost of electric two-wheelers, from the market analysis, it is found that the cost of electric two-wheelers is higher than compared to that of the conventional two-wheelers. Therefore, this factor might be one of the important issues that the manufacturers must address immediately to reduce the purchase cost of electric two-wheelers to make it a viable alternative for the users.

The findings of this study suggest that the Ease of pedaling for electric bicycles and cargo carrying capacity for electric motorcycles is found to be the important vehicular-related attributes. However, the importance of cargo carrying capacity of an electric motorcycle seems to be surprising, as it was not expected among the vehicular-related category, it is likely that the reason might be the lack of confidence among the user toward the performance of electric motorcycles, the users might get clarity only when they start to use the electric motorcycle. If the cargo carrying capacity is acceptable for the user, it might increase electric motorcycle ownership among the users.

As anticipated, the absence of Tailpipe emission for electric motorcycles is a positive influencing factor for the users to shift toward the electric motorcycle to mitigate the environmental concerns regarding the deteriorating air quality in major cities and towns with high-motorized traffic. Among battery related, it is not surprising that battery life is found to be the most important attribute, in a positive note, even the manufacturers of electric vehicles are seriously working toward the enhancement of the performance of the battery in terms of increasing the life of a battery that could encourage the users to believe in the electric two-wheelers. As the battery is an important component of an electric two-wheeler, an increase in battery life would minimize the variable cost associated with electric wheelers and positively influence the increase in electric two-wheelers. Similarly, the findings from this study highlight the importance of the Safety factor, the concerns regarding the safety of the user during the operation of the electric two-wheeler. Hence, the traffic controlling authorities must primarily concentrate on framing effective regulations regarding the safety of the slow-moving electric two-wheeler users.

The government and the local administration authorities must concentrate on improving the charging infrastructures that could facilitate the users to charge the battery of the vehicles at public places such as shopping areas and busy commercial localities. Even we firmly believe that the success of the policies framed to focus on increasing the electric two-wheeler ownership will depend upon the availability of charging infrastructures at the required locations. Proper investigations will be helpful to locate the critical locations required for the improvement in charging infrastructures. Finally, Travel time reliability is found to be the most important attribute among the commute quality-related attributes.

6 Conclusions

This study has contributed significantly toward the identification of key attributes related to the use and purchase intention of the users related to the electric two-wheelers, it can be suggested that to frame the policies effectively, the policymakers must consider the issues related to the above-identified key attributes. If the policies are framed effectively, then it can be anticipated that at least some portion of the conventional two-wheeler users might be shifted to electric two-wheelers. As mentioned earlier, this study was limited to the identification and prioritization of the attributes. Based on these prioritized attributes, further studies related to the mode choice analysis of conventional users will be carried out by collecting the perceptions of the actual two-wheeler users. A full-scale user-related study will be carried out across different cities in India to study the extent of influence of key attributes toward the users to shift their mode of travel to the electric two-wheeler. Based on the user perception, relevant choice models will be developed, which would help to estimate the possible mode shift of the conventional two-wheeler users.

References

1. Ramachandra TV (2009) Emissions from India's transport sector: state wise synthesis. Atmos Environ 43(34):5510–5517. https://doi.org/10.1016/j.atmosenv.2009.07.015
2. Government of India (2014) Census of India 2011. Report on post enumeration survey. New Delhi, India
3. Government of India (2017) Motor vehicles. Statistical yearbook India
4. Ramachandra TV, Sellers JM, Bharath HA, Vinay S (2019) Modeling urban dynamics along two major industrial corridors in India. Spat Inf Res 27(1):37–48. https://doi.org/10.1007/s41324-018-0217-8
5. British Petroleum Company (2018). BP statistical review of world energy
6. Solanki HK, Ahamed F, Gupta SK, Nongkynrih B (2016) Road transport in Urban India: its implications on health. Indian J Community Med Official Publ Indian Assoc Prev Soc Med 41(1):16. https://doi.org/10.4103/0970-0218.170959
7. Government of India, Ministry of Heavy Industries and Public Enterprise (2019) Publication of notification in Gazette of India regarding Phase-2 of FAME India scheme
8. Simsekoglu Ö, Klöckner C (2019) Factors related to the intention to buy an e-bike: a survey study from Norway. Transport Res F: Traffic Psychol Behav 60:573–581. https://doi.org/10.1016/j.trf.2018.11.008
9. Zhu L, Song Q, Sheng N, Zhou X (2019) Exploring the determinants of consumers' WTB and WTP for electric motorcycles using CVM method in Macau. Energy Policy 127:64–72. https://doi.org/10.1016/j.enpol.2018.12.004
10. Elliot T, McLaren SJ, Sims R (2018) Potential environmental impacts of electric bicycles replacing other transport modes in Wellington, New Zealand. Sustain Prod Consumption 16:227–236. https://doi.org/10.1016/j.spc.2018.08.007
11. Lin X, Wells P, Sovacool BK (2017) Benign mobility? Electric bicycles, sustainable transport consumption behaviour and socio-technical transitions in Nanjing, China. Transp Res Part A: Policy Pract 103:223–234. https://doi.org/10.1016/j.tra.2017.06.014
12. Fyhri A, Heinen E, Fearnley N, Sundfør HB (2017) A push to cycling—exploring the e-bike's role in overcoming barriers to bicycle use with a survey and an intervention study. Int J Sustain Transp 11(9):681–695. https://doi.org/10.1080/15568318.2017.1302526
13. Kroesen M (2017) To what extent do e-bikes substitute travel by other modes? Evidence from The Netherlands. Transp Res Part D: Transp Environ 53:377–387. https://doi.org/10.1016/j.trd.2017.04.036
14. Cheng EW, Li H (2002) Construction partnering process and associated critical success factors: quantitative investigation. J Manag Eng 18(4):194–202. https://doi.org/10.1061/(asce)0742-597x(2002)18:4(194)
15. Wong JK, Li H (2008) Application of the analytic hierarchy process (AHP) in multi-criteria analysis of the selection of intelligent building systems. Build Environ 43(1):108–125. https://doi.org/10.1016/j.buildenv.2006.11.019
16. Patil M, Majumdar BB (2021) An investigation on the key determinants influencing electric two-wheeler usage in urban Indian context. Res Transp Bus Manag 100693. https://doi.org/10.1016/j.rtbm.2021.100693
17. Piantanakulchai M, Saengkhao N (2003, Oct) Evaluation of alternatives in transportation planning using multi-stakeholders multi-objectives AHP modeling. In: Proceedings of the Eastern Asia Society for transportation studies, vol 4, pp 1613–1628
18. Poh KL, Ang BW (1999) Transportation fuels and policy for Singapore: an AHP planning approach. Comput Ind Eng 37(3):507–525. https://doi.org/10.1016/S0360-8352(00)00020-6

19. Saaty TL (1997) That is not the analytic hierarchy process: what the AHP is and what it is not. J Multi-Criteria Decis Anal 6(6):324–335. https://doi.org/10.1002/(SICI)1099-1360(199711)6:6%3c324::AID-MCDA167%3e3.0.CO;2-Q
20. Friedman M (1937) The use of ranks to avoid the assumption of normality implicit in the analysis of variance. J Am Stat Assoc 32(200):675–701

Speed and Headway Behaviour of Motorcycles on an Exclusive Motorcycle Lane

Nischal Gupta, Ch. Ravi Sekhar, and A. U. Ravi Shankar

1 Introduction

Two-wheelers generally offer flexible personal mobility with greater accessibility on the roads. In most of the South-Asian countries including India, the proportion of Motorized Two-Wheeler (MTW) is very high. The two-wheeler population in India is more than 70% as of 2017 [11]. Several second order cities in India suffer from a high degree of congestion problems at intersections due to the high composition of motorcycles. One can argue that the MTWs are an important transport mode for a large proportion of people in Indian cities who lack access to good quality public transport options, but the serious challenges of safety concerns and dependence on personal motorised vehicles that they give rise to, cannot be ignored. India records the maximum number of deaths from two-wheeler accidents in the world [6]. In 2016, 34% of total road accidents in India involved two-wheelers which are the highest for any vehicle category [11].

Since, the nature of traffic in India is highly heterogeneous without any lane discipline, different categories of vehicles share the same road space and thus conflicts are bound to increase. MTWs with their freedom to manoeuver in between gaps of other vehicles pose a threat not only to other vehicles but also to the two-wheeler riders thus making them more vulnerable to accidents. One possible solution to better manage the motorcycle traffic is to provide a separate lane and segregate them from the rest of the traffic stream. Segregation of these vulnerable road users can help

N. Gupta
Michigan State University, East Lansing, USA

Ch. Ravi Sekhar (✉)
Transportation Planning and Environment Division, CSIR-Central Road Research Institute, New Delhi, India

A. U. Ravi Shankar
Department of Civil Engineering, National Institute of Technology Karnataka, Surathkal, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_4

reduce conflicts. Few South-Asian countries like Malaysia, Taiwan and Thailand have implemented separate lanes for motorcycles. These are referred to as Exclusive Motorcycle Lane (MC lane). India is yet to explore the benefits of such facilities.

One can find immense literature that discusses the fundamental speed-flow-density relationships, capacity and level of service for traffic and pedestrian flows [7–9, 12]. Very few studies are available in the literature that discusses the speed-flow-density relations, headway, capacity and level of service of MC lanes. In order to better understand the motorcycle traffic operations, Minh et al. [10] collected and analysed the motorcycle speed, headway and traffic flow data along with the road geometrics data at four different sites in the city of Hanoi, Vietnam. Out of these four sites, two sites were having exclusive motorcycle flow while the other two sites were having mixed traffic with non-motorised and four-wheel vehicles. The average speed of motorcycle on a 3.7 m wide of homogeneous motorcycle traffic lane was found to be 32.5 km/h, with a standard deviation (SD) of 5.5 km/h. On an undivided and mixed traffic roadway of 5.54 m width, the average speed of the motorcycle is 21.3 km/h with a SD of 5.1 km/h. On a 3.27 m wide undivided roadway with mixed traffic, the average motorcycle speed was found to be 22.8 km/h with a SD of 4.6 km/h. In the same study, the average headway for all locations was found to be 1.16 s. The headway ranged from 0.34 and 4.31 s with a standard deviation of 0.65 s. Das and Maurya [4] studied about staggering behaviour of two-wheelers in mixed traffic condition on six lane divided urban roads in India.

In the present study, an attempt is made to study the traffic characteristics on an exclusive MC lane. The major objective of the present study is to investigate the speed and headway behaviour of MTWs on an exclusive MC lane. The speed and headway behaviour of MTWs are studied before and after segregation of MTWs.

2 Study Area and Data Collection

For the purpose of the present study, an urban arterial corridor on VIP road, Vadodara with high MTW proportion is selected. A study section between Manisha Circle and Akshar Chowk in Vadodara, Gujarat is identified for data collection. This study section is a 6-lane divided road with paved shoulders. The identified 100 m section is free from any interference or influence from intersections, pedestrian crossings, turns, street roads, gap in the median, etc. The data are collected and extracted for three different scenarios. The scenario where there is no separate motorcycle lane present (no segregation of traffic) is referred to as BAU scenario. On the same road, once the segregation of MTWs is done there will be two different types of lanes. One will be a separate lane for the exclusive use of MTWs and it is referred to as the MC lane scenario. The remaining carriageway will be used by other vehicle categories and this is referred to as the Mixed Vehicle lane or MV lane.

For the BAU scenario, the data is collected for a total of five hours (three hours in the morning and two hours in the evening). The carriageway width in this case (BAU) is 10.5 m with 3.5 m paved shoulder. Thereafter, a 3.5 m wide lane is physically

a) Before Segregation (BAU Scenario) b) After Segregation (MC lane and MV lane)

Fig. 1 Traffic data collection for different scenarios on VIP Road, Vadodara

separated from the rest of the carriageway using traffic barriers. The total length of the segregated carriageway is 100 m. Thus, the carriageway width for MV lane is now reduced to 10.5 m with obstruction on both the sides (median on one side and barricades on the other). The MTW users were requested to make use of the motorcycle (MC) lane instead of the MV lane. Traffic operations have been carried out with the help of traffic police personnel. The Motorcyclists were requested to segregate about 100 m before the start of segregation so that they have ample time to separate from the stream without any effect on their behaviour. After segregation of the MTWs, the video graphic data is collected for 2 h only. Figure 1 shows the site under the BAU scenario and MC lane scenario.

Vehicle categories with similar characteristics are clubbed together and a total of 8 different categories of vehicles are identified. These include small car (standard car), big car (SUV, MUV, etc.), Heavy Commercial Vehicle (HCV), Light Commercial Vehicle (LCV), Bus, MTWs (Scooter, Motorcycle, Moped, Scooty such as Activa), Three wheelers and Non-Motorized Traffic (NMT). The category wise vehicle composition is given in Fig. 2. It is observed from the figure that the composition of 2 W is 60% which is more predominant than the other categories of vehicles. The composition of car traffic is 27% which includes big and small car.

3 Speed Behaviour of Motorised Two-Wheelers

The video data collected for the BAU scenario and MC lane is utilised to extract the speed distributions of MTWs before and after their segregation. For this, the trap length method is used where a 30 m longitudinal trap is physically marked on the study road section with a cloth tape. The speed is determined by dividing the trap length by the time taken by the vehicle to traverse the trap. To get the timestamp of entry and exit of each vehicle more accurately, a software developed [3] during the Indo-Highway Capacity Manual study (Indo-HCM 2017) is used which gives the

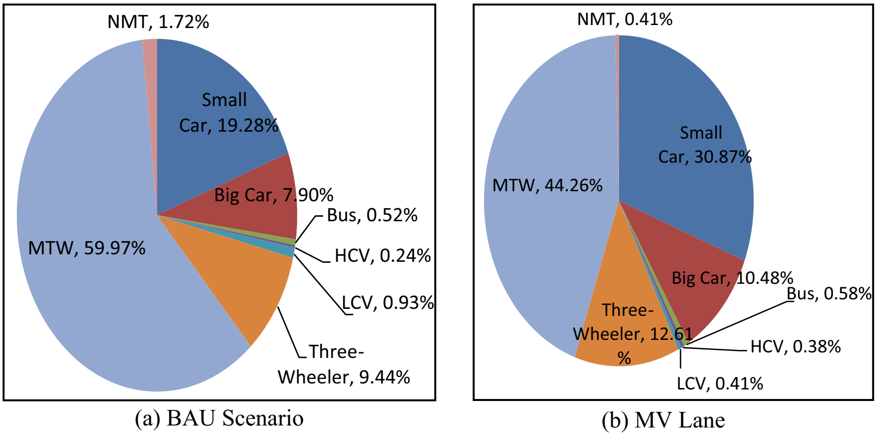


Fig. 2 Average traffic composition on VIP Road, Vadodara

time of entry and exit in seconds up to 5 decimal places. The summary of the speed data for the various scenarios is given in Table 1.

The speed distribution of MTWs under the three different scenarios is represented in Fig. 3. The median speed (50th percentile) of MTWs before any segregation (BAU scenario) is about 41 km/h which reduces to 36.4 km/h in MV lane after segregation. This is essentially due to the reduced width of the carriageway. The median speed in MC lane (3.5 m wide) is 30 km/h. Similarly, the operating speed (85th percentile) for BAU scenario, MV lane and MC lane is 51 km/h, 43 km/h and 36 km/h respectively.

Suitable probability distributions are fitted to the speed data obtained from the field. Six distributions namely Normal, Log-Normal, Logistic, Log-Logistic, Gamma and Log-Gamma distributions are considered. The goodness of fit of each probability distribution is assessed based on the Chi-Square statistic method. The results are summarised in Table 2 and it can be noted that the speeds of MTWs follow Log-Gamma distribution ($\alpha = 352.7$ and $\beta = 0.01$) under the BAU scenario. After segregation, the speed of MTWs on the MC lane follows normal distribution ($\mu = 5.81$ and $\sigma = 30.83$) and Log-Gamma distribution ($\alpha = 365.15$ and $\beta = 0.01$) on MV lane. The best-fitted distribution for each scenario is presented in Fig. 4. The basic functions of Log-Gamma and Normal distributions are represented in Eqs. (1) and (2) respectively.

Table 1 Summary of MTW speed data

Scenario	Average MC volume (mc/h)	Observed speed (km/h)			
		Minimum	Maximum	Mean	Standard deviation
BAU	1774	25.4	100	42.22	8.79
MV Lane	1202	21.2	60	37.13	7.3
MC Lane	807	15	77	30.83	5.81

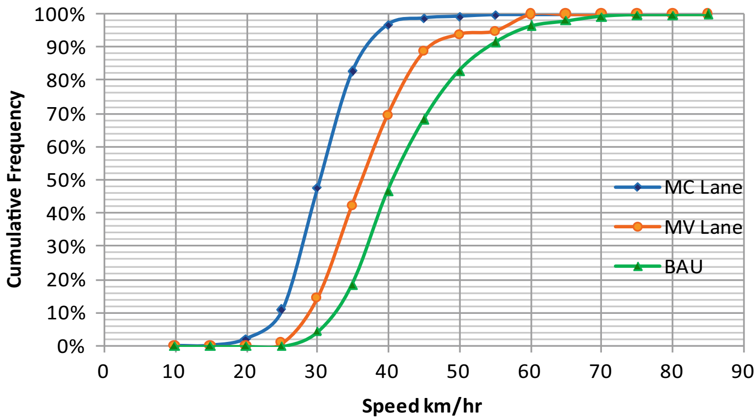


Fig. 3 Speed distribution of MTW under different scenarios

Table 2 Results of probability distributions fitted to MTW speed data

S. No.	Name of distribution	χ^2 Statistic		
		BAU	MV Lane	MC Lane
1	Gamma	22.2	9.8	142.5
2	Log-Gamma	7.9	3.0	183.9
3	Logistic	38.4	8.6	102.9
4	Log-Logistic	14.1	3.8	114.1
5	Normal	47.3	9.9	87.2
6	Log-Normal	8.6	5.7	162.9

Probability Density function of Log-Gamma Distribution

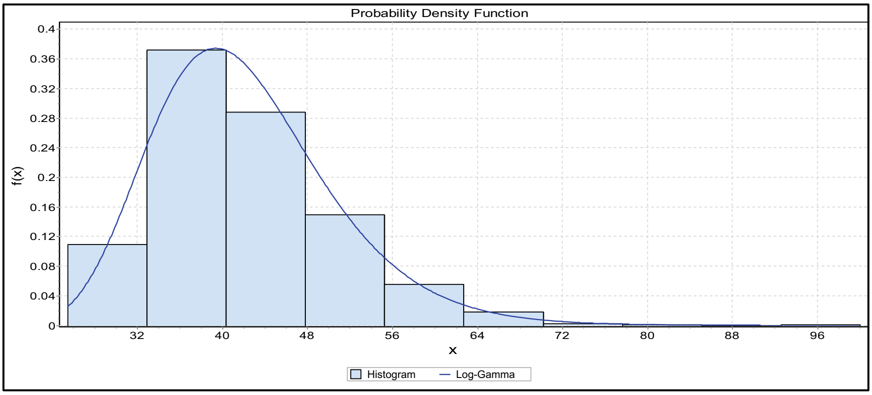
$$f(x) = \frac{(\ln(x))^{\alpha-1}}{x\beta^{\alpha}\Gamma(\alpha)} \exp\left(-\frac{\ln(x)}{\beta}\right) \tag{1}$$

where x is the random variable has the Log-Gamma distribution (Γ) with positive scale parameter α and positive shape parameter β.

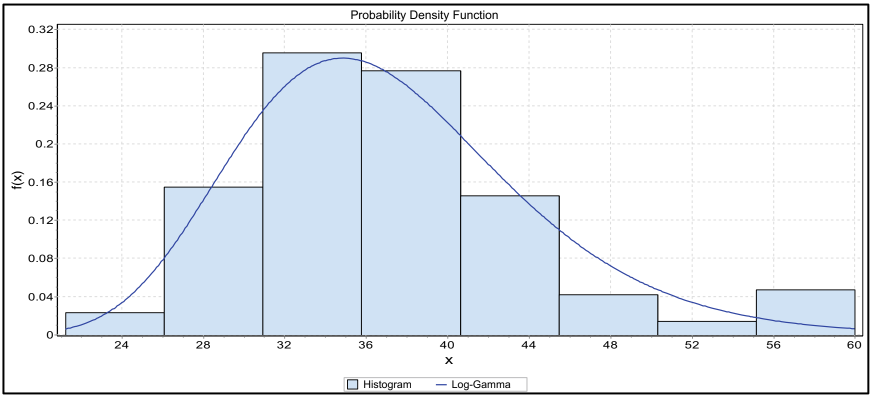
Probability Density function of Normal Distribution

$$f(x) = \frac{\exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right)}{\sigma\sqrt{2\pi}} \tag{2}$$

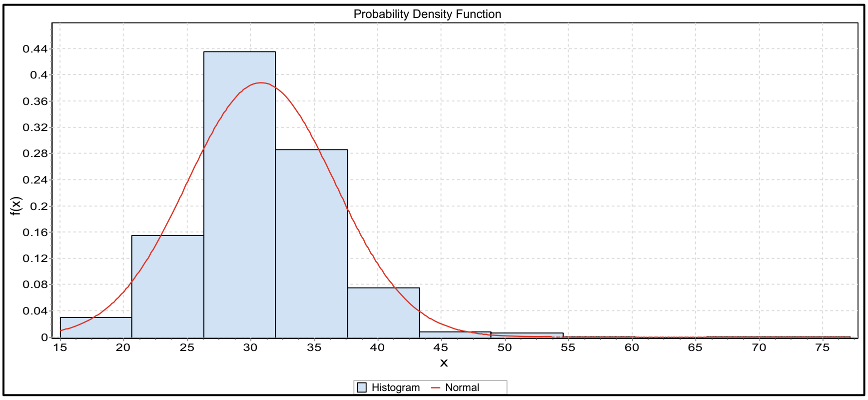
where x is the random variable has the normal distribution with mean (μ) and variance σ^2.



a) BAU Scenario (Log-Gamma)



b) MV Lane (Log-Gamma)



c) MC Lane (Normal)

Fig. 4 Best fit distributions to MTW speed for various scenarios

It was observed from the fitted probability distributions characteristics that during BAU scenario the variability in speeds of motorised two-wheelers is more. The probability distribution profile is skewed towards larger speeds. Whereas, MTW speed distribution on MC lane follows the symmetric distribution and most of the MTWs follow less than 30 km/h speed on the MC lane. The reduction in speed of the MTW in MC lane leads to a reduction in risk of accidents on this corridor.

4 Headway Behaviour of MTW

To understand the two-wheeler driver behaviour during mixed traffic conditions and homogeneous condition (MC lane), this study considers the headway distribution of two-wheelers. This parameter is useful in comparing the driver behaviour on the BAU scenario as well as for MC lane scenario. Further, this will be useful in formulating a suitable policy for MC lane in the future. The time headway data of MTWs before and after segregation is extracted from the field data. For this, the time of arrival of MTW on a predetermined point in the video is noted down for every successive MTW. The time headway is then determined by subtracting the consecutive arrival times. The summary of the headway data is presented in Table 3. Figure 5 shows the headway frequency distribution of MTWs for the three scenarios.

Seven different probability distributions namely Gamma, log-gamma, logistic, log-logistic, normal, log-normal and exponential distributions are considered to fit the field headway data and the best-fitted distribution for each scenario is presented in Fig. 6. The best fit distributions are recommended based on the chi-square method. Table 4 gives a summary of the Chi-square test statistic used to estimate the goodness of fit for each distribution. It is concluded that the time headway data follow Log-logistic distribution (with parameters $\alpha = 1.54$ and $\beta = 1.1$) before segregation of motorcycles but follows log-normal distribution ($\mu = 0.83$ and $\sigma = 1.16$) after segregation on MV lane. On MC lane also, the headway follows log-normal distribution ($\mu = 0.68$ and $\sigma = 1.05$). The basic functions of Log-Logistic and Log-normal distributions are represented in Eqs. (3) and (4) respectively.

Table 3 Summary of headway data under various scenarios

S. No.	Property	Scenario		
		BAU	MV Lane	MC Lane
1	Sample size	856	188	341
2	Mean (s)	1.99	4.39	3.44
3	Median (s)	1.21	2.05	1.7
4	Mode (s)	0	1.57	0.53
5	Range (s)	22.51	29.75	24.75
6	Standard deviation (s)	2.397	5.55	4.38

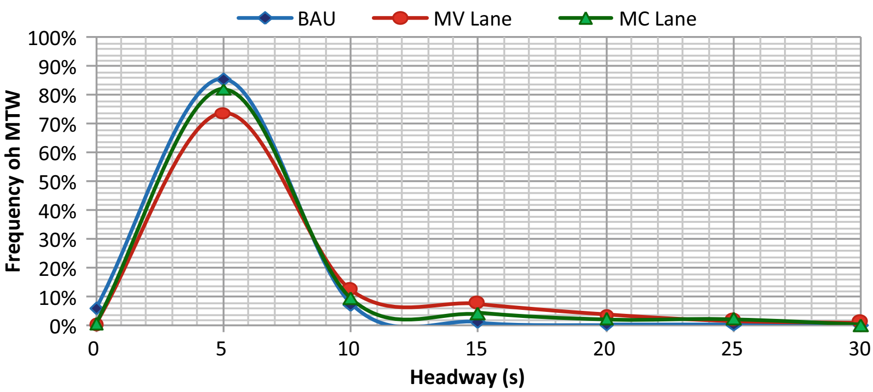


Fig. 5 Headway frequency distribution under various scenarios

Probability Density function of Log-Logistic Distribution

$$f(x) = \frac{\alpha}{\beta}\left(\frac{x}{\beta}\right)^{\alpha-1}\left(1+\left(\frac{x}{\beta}\right)^{\alpha}\right)^{-2} \tag{3}$$

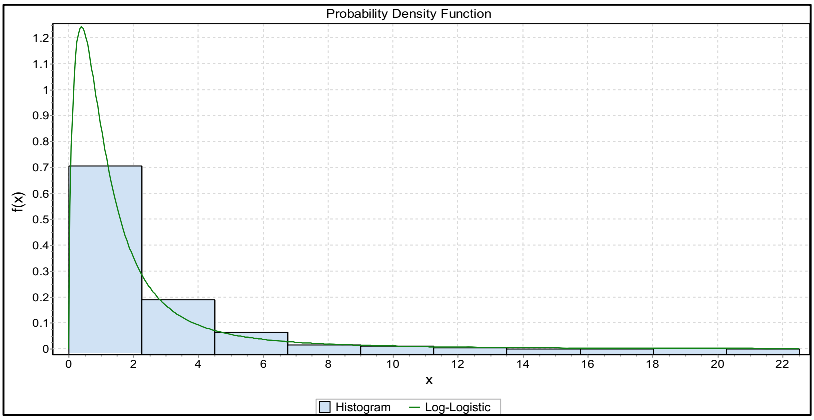
Probability Density function of Log-Normal Distribution

$$f(x) = \frac{\exp\left(-\frac{1}{2}\left(\frac{\ln x-\mu}{\sigma}\right)^2\right)}{x\sigma\sqrt{2\pi}} \tag{4}$$

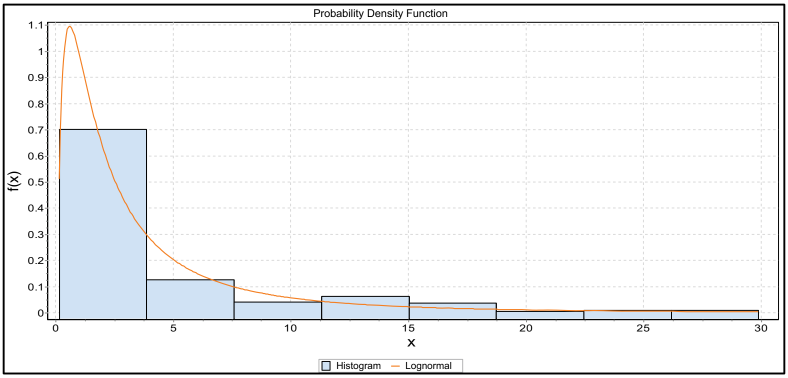
4.1 Statistical Validation

In order to test whether there is any significant difference in the time headway of MTWs before and after segregation, statistical validation has been done using the *t*-test. The time headway of MTWs on BAU is compared statistically with that on MV lane and MC lane. Before carrying out the *t*-test, *F*-test is done to determine whether there is any significant difference in the variance of the data set. Both the statistical tests have been carried out at 5% level of significance. The summary of the results of the *F*-test and *t*-test has been presented in Table 5.

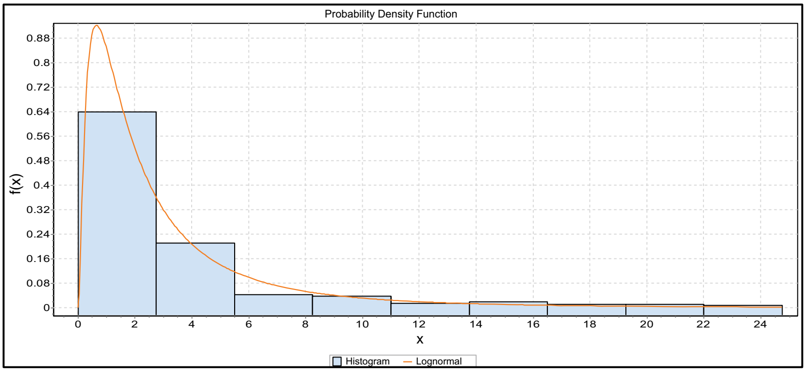
From the results of *F*-test, it can be concluded that there is no significant difference between the variance of the time headway of MTWs before and after segregation. Thus, t-test assuming equal variances has been carried out to test whether there is any significant difference in mean of time headway of MTWs. Since the t-critical value in both the cases is less than the absolute value of t-stat, it can be concluded that there is a significant difference in the meantime headway values of MTWs before and after segregation (on BAU and MV lane). Once segregated, the MTWs on a homogeneous



a) BAU Scenario (Log-Logistic)



b) MV Lane (Log-Normal)



c) MC Lane (Log- Normal)

Fig. 6 Probability headway distributions of motorised two-wheeler

Table 4 Results of probability distributions fitted to MTW time headway data

S. No.	Name of distribution	Chi-square statistic		
		BAU Scenario	MV Lane	MC Lane
1	Gamma	48.1	42.9	86.24
2	Logistic	177.5	45.4	109.9
3	Log-Logistic	27.5	7.14	15.3
4	Normal	218.3	39.4	131.6
5	Lognormal	82.1	2.9	11.5
6	Exponential	31.2	23.5	48.5

Table 5 Summary of statistical tests

Comparison	F-stat	F-critical	t-stat	t-critical
BAU versus MV lane	0.187	0.834	−9.32	1.96
BAU versus MC lane	0.299	0.864	−7.35	1.96

lane (MC lane) also exhibit significantly different time headway than in the mixed lane (BAU scenario).

The capacity is estimated for both the study sections under various scenarios using Greenshields model [5]. The capacity is estimated in the units of PCU/h where PCU stands for Passenger Car Units. The PCU for each category of vehicle is estimated using Eq. (5) [2].

$$\mathrm{PCU} = \frac{V_c/V_i}{A_c/A_i} \tag{5}$$

where V_c and V_i are the mean speeds for cars and ith type vehicle, respectively, in the traffic stream; and A_c and A_i are their respective projected rectangular areas (length × width) on the road. The projected rectangular areas are taken from the Indian Highway Capacity Manual [1]. Since, the traffic is heterogeneous in India; weighted space mean speed as given by Eq. (6) is used to determine the stream speed.

$$V_m = \frac{\sum_{i=1}^{k} n_i v_i}{\sum_{i=1}^{k} n_i} \tag{6}$$

where k = total number of vehicle categories present in the stream, V_m = mean stream speed (km/h), v_i = speed of ith vehicle of category (km/h), and n_i = total number of vehicles of category i. The results of the capacity estimation are summarised in Table 1.

5 Conclusions

The objective of this paper was to study the speed and headway behaviour of motorcycles on an exclusive motorcycle lane. For this, the urban arterial corridor in Vadodara city was considered and video graphic data was collected and the traffic volume data, speed and headway of two-wheeler traffic were extracted. The behaviour of speed and headway of motorcycles on Business-as-Usual (BAU) scenario and exclusive motorcycle lane has been studied. Further, the mathematical distribution of speed and headway was fitted and the best fit distribution was recommended. The following conclusions are drawn from the study.

- The average speed of the motorised two-wheeler (MTW) during BAU scenario is about 42 km/h whereas MTW speed is reduced to 31 km/h on MC lane after separation of two-wheelers. This speed reduction can be due to reduction in width of lane and did not get chance to overtake in 100 m segregated length.
- The speed of motorised two-wheeler on the MC lane follows normal distribution whereas BAU scenario the motorcycle speed distribution follows Log-Gamma distribution. This emphasises that the motorised two-wheeler speeds are highly skewed towards larger speeds when there is no exclusive lane this may lead to higher accident rates when MTW composition is predominant on the traffic stream.
- The headway data follows Log-Logistic distribution before segregation but follows Log-normal distribution on exclusive motorcycle lane.
- The mean headway of MTWs in MC lane is about 3.4 s which is higher than in BAU scenario. This is because the MTWs in MC lane feel restricted to overtake and hence prevented diamond skewing.
- There is a significant difference in the time headway of MTWs before segregation and after segregation. Also, the time headway of MTWs on a homogeneous lane varies significantly than on a mixed lane.

Acknowledgements Authors are very grateful to the Prof. Satish Chandra, Director CSIR-Central Road Research Institute for allowing us to publish this paper.

References

1. Central Road Research Institute (2017) Indian highway capacity manual. New Delhi, India
2. Chandra S, Sikdar PK (2000) Factor affecting PCU in mixed traffic on urban roads. Road Transport Res, 9(3):40–50
3. Data Extraction Software (2013) Department of Transportation, Anna University
4. Das S, Maurya A (2019) Modeling maneuverability of motorized two-wheelers during filtering in urban roads. Transp Res Rec J Transp Res Board
5. Greenshields BD, Bibbins JR, Channing WS, Miller HH (1935) A Study of Traffic Capacity. Highway Research Board Proceedings, 14(1):448–477

6. Gupta A, Jaipuria J, Bagdia A (2014) Motorised two-wheeler crash and helmets: injury patterns, severity, mortality and the consequence of gender bias. World J Surg 38(1):215–221
7. Highway capacity manual. Transportation Research Board, National Research Council, Washington, D.C. (2010)
8. Indonesian highway capacity manual. Part I—Urban and semi-urban traffic facilities. The Directorate General of Highways and the Indonesian Road Engineering (1993)
9. Indonesian highway capacity manual. Part II—Interurban roads and motorways. The Directorate General of Highways and the Indonesian Road Engineering (1995)
10. Minh CC, Sano K, Matsumoto S (2005) The speed, flow and headway analyses of motorcycle traffic. J East Asia Soc Transp Stud 6:1496–1508
11. Ministry of Road Transport & Highways. Road accidents in India 2017 (2017)
12. Ministry of Road Transport & Highways India. Road transport year book: 2013–14 and 2014–15 (2017)

Performance Evaluation and Enhancement of Unsignalized Intersection Using Microsimulation in VISSIM

Suman Ganguly, Mokaddes Ali Ahmed, and Manish Dutta

1 Introduction

In developing countries like India, Bangladesh, and Sri Lanka, traffic flow is considered as mixed. This traffic consists of motorized as well as non-motorized vehicles with different static and dynamic characteristics [1]. Another characteristic of this traffic is poor lane discipline which results in evasive movements of vehicles, especially near intersections. Moreover, the number of vehicles are also increasing day by day with the rapid urbanization, which altogether creates severe congestion on roads and highways. This congestion would deteriorate the level of service of the traffic facilities and increase vehicular emissions. The road transportation sector became a major contributor to CO, NO_x, and VOC emissions [2]. Thus, to serve the increasing demand for traffic, either new facilities should be provided, or existing facilities should be improved. A proper understanding of traffic behavior is necessary before implementing any decision in the field, as it is an expensive process. Microsimulation is a cost-effective tool that provides the user with an understanding of traffic flow and behavior and is suitable to model heterogeneous traffic [3].

Performance evaluation of intersections is necessary to find out the loopholes in any road network system. Intersections are very much affected due to the heterogeneity of traffic observed in developing countries. This heterogeneity can negatively impact the LOS and automobile emissions. Thus, performance evaluation of an unsignalized intersection is necessary for a better understanding of the situation.

S. Ganguly (✉) · M. A. Ahmed
Department of Civil Engineering, National Institute of Technology Silchar, Silchar, Assam 788010, India

M. Dutta
Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad, Gujarat 382481, India
e-mail: manish.dutta@nirmauni.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_5

In the last decade, microsimulation package VISSIM has gained popularity due to its multimodal characteristics and widespread applications. VISSIM is a microsimulation software based on Wiedemann (1974 and 1999) car-following model developed in Germany. It has default driving behavior parameter values based on studies conducted in Europe and several other developed countries. These parameter values need to be adjusted for heterogeneous traffic which exists in developing countries; i.e., calibration is needed before using the model for performance evaluation.

The primary object of this study is to develop a simulation model in VISSIM for unsignalized T-intersections using a suitable methodology relevant for heterogenous mixed traffic. The other objectives include performance evaluation and enhancement of the intersection by the use of the developed simulation model. This study considers LOS and vehicular emission as performance measure criterion.

2 Literature Survey

Various studies have been done to model the heterogeneous traffic at signalized intersections. Mathew and Radhakrishnan [3] first attempted to calibrate the model in Indian heterogeneous non-lane-based traffic conditions. This study choses stopped delay as a measure of effectiveness. Before calibration of the model, the parameters affecting the output in a significant way have to be found out. Sensitivity analysis was performed to find out those parameters. Mathew and Radhakrishnan used ANOVA as a tool to perform sensitivity analysis. Parameter calibration was done by optimization technique with the automation method. In a study conducted by Siddharth and Ramadurai [4], flow was taken as the measure of effectiveness. A corridor was chosen to perform the study. A two-level sensitivity analysis was conducted. Latin hypercube sampling technique was used for performing sensitivity analysis. One hundred fifty sample sets of 11 parameters were generated, and ANOVA was done to perform first-level sensitivity analysis. Second-level sensitivity analysis was done using a quasi-optimized trajectory in the EE method with the samples that were previously not sensitive. Parameter optimization was done with the use of GA. Manjunatha et al. [5] performed a multilevel sensitivity analysis where the relationship between sensitive parameters was also found out. Control delay was the measure of effectiveness. Local and global calibration methodologies were attempted. In Boltze et al. [6], VISSIM was calibrated using the driving behavior of different vehicle classes separately. Calibration was performed for the mid-block section and signalized intersection by taking travel time as the measure of effectiveness. Separate regression equations were developed for both cases. Single as well as multiple criteria optimizations was performed using GA.

Yun and Ji [7] analyzed stop sign and yield sign intersections in VISSIM. An unsignalized intersection was considered as the study area. It compared the effects of the two-way stop-controlled intersection, all-way stop-controlled intersection, and yield-controlled intersection. This study showed that priority rules are very important to model yield controlled intersection. Finally, warrants had been imposed on the intersection. Paul and Pitale [8] compared the capacity of an unsignalized intersection to a signalized intersection. Unsignalized T-intersection was taken as the study area. HCS 2000 software was used for capacity analysis. A signal system was planned for the intersection using Webster method, and capacity was found to be greater in a signalized intersection. Vajeeran and De-Silva [9] analyzed delay at signalized intersections using VISSIM in Sri Lanka. The main objective of their study was to find out the causes of delay and subsequently find ways for improvement. Model calibration was done using the trial-and-error process. This study recommended new signal timing and phasing plans for the intersection to decrease the delay. It was found from the study that combining measures like new signal phase and timing plan, lane width increment decreases the delay and increases the capacity of the intersection by a reasonable amount. Dutta and Ahmed [10] calibrated the VISSIM model for unsignalized intersections in Indian conditions. This study took three three-legged junctions as the study area. The measure of effectiveness of this study was flow and occupancy time. One of the main aspects of this study was to set up the priority rules for unsignalized intersections in mixed traffic. A trial-and-error approach was used for calibration purposes. Local and global calibration was performed, and it was satisfactory in both cases for the three intersections. Paul et al. proposed (2017) [11] a calibration methodology to model unsignalized intersections under heterogeneous traffic conditions in VISSIM. 85th percentile accepted gap time was taken as a performance measure criterion. Morris sensitivity analysis was used to find the calibration parameters, and optimization was done by using genetic algorithm.

It can be concluded from the past studies that most of the works concentrated on developing simulated models for signalized intersections and arterial roads in heterogeneous traffic. Also, there is less work that directly addressed the performance assessment and improvement of unsignalized intersections. Thus, this study focuses on developing a suitable calibration methodology focusing on unsignalized intersection using VISSIM and measure the performance of the intersection.

3 Methodology

The methodology adopted in this study includes certain distinct features like conducting traffic survey, vehicular, and geometric representation of intersection into the simulation model, setting up priority rules, identifying calibration parameters using sensitivity analysis followed by optimizing the calibrated parameters. Validation is the last step of modeling. To verify the need for calibration, the first step is to determine the average delay faced by the vehicles to cross the intersection in the field and compare it with the average delay obtained from simulation. If the error is significant, which usually is the case, there is a need for calibration. After calibration, the next step is to apply the model to evaluate the performance of the intersection. Lastly, the application of traffic management measures to enhance performance (if needed) has to be done. The methodology is explained in the following steps:

3.1 Data Requirements

The data required for developing simulation model consist of traffic flow, intersection geometry, lane widths, desired speed data, traffic composition, and field delay at intersections. Field delay refers to the delay faced by the vehicles to clear the intersection area. It is calculated by the difference in travel time for a vehicle crossing the intersection in peak hour and non-peak hours.

3.2 Vehicle and Intersection Geometry Representation

Simulation models have some standard types of vehicles by default which does not realistically represent the vehicles of mixed traffic. In the first step, it is required to represent the motorized and non-motorized vehicles according to its length, width, and desired speed distribution. The second aspect is to represent the geometry of the network defined by the number of lanes, width of lanes, flared section area of the intersection. The third aspect is the priority rules setup. This representation includes headway and minimum gap time selection, which depends on the specific case study intersection.

3.3 Traffic Representation

Traffic behavior observed in real time cannot be represented in simulation accurately with default settings. This phase includes the identification of certain local traffic characteristics based on case study and tuning of some parameters so that simulated traffic behaves like real-world traffic. Though it is not possible to exactly match the simulation traffic behavior to real-world traffic behavior, one can try to bring them as close as possible.

3.4 Simulation with Default Parameters

VISSIM runs with default parameters, and average stream delay is obtained. If the error between field and simulated value is significant, then there is a need for calibration; otherwise, the model can be used in its default state.

3.5 Sensitivity Analysis

The identification of sensitive parameters is the next step after assessing the need for calibration. It is done by conducting an analysis of variance with average delay as the measure of sensitivity. Parameters chosen for sensitivity analysis are identified by the previous studies [3, 4]. Chosen parameter values are increased one by one to 10% or 20% with all other parameter values fixed to its default value, and its effect in output is observed by performing analysis of variance (ANOVA).

3.6 Latin Hypercube Sampling and Multiple Linear Regression

Sensitive parameters identified in the last step need to be calibrated, so that simulated delay matches with field delay. To serve this purpose, Latin hypercube sampling is used. This Latin hypercube sampling technique reduces the combination of parameters to a reasonable limit. Possible combinations of parameters are given input to the model one by one, and the output value is recorded. Then, at last, regression equations may be developed by taking sensitive driving behavior parameters as the independent variable and desired output as the dependent variable. A total of 100 parameter value sets are generated, and multiple runs are being conducted for every set to reduce the error.

3.7 Model Optimization by Genetic Algorithm

Optimization using genetic algorithm is the last step of the calibration of the model. Regression equations that are developed will be given input to the genetic algorithm solver with appropriate parameter bounds. GA is based on Darwin's theory of evolution. It works on the principle of survival of the fittest. Basic operators of GA are natural selection, cross-over, and mutation. The objective function of optimization is represented in a way that the absolute difference between field and simulated delay becomes minimum with mean absolute percentage error (MAPE) less than 15%, as mentioned by Dowling et al. [12]. The formulation of the objective function is given as

$$\begin{aligned} &\text{Minimize } \varepsilon_{abs} = \sum_{i=1}^{n} \left| d_s^i - d_f^i \right| \\ &\text{Subjected to, } \alpha_{\text{Min}}^j \le \alpha \le \alpha_{\text{Max}}^j \quad \forall j \end{aligned} \tag{1}$$

where

d_s^i	Average stream delay obtained from the simulated model for ith intersection
d_f^i	Average stream delay obtained in the field for ith intersection
$\alpha_{\min}^j, \alpha_{\max}^j$	are the lower and upper bounds of the jth parameter α^j.
n	number of intersection consideration for study.

3.8 Validation

A new traffic dataset should be used to check the flexibility and robustness of the model. If using different data, absolute error between field and simulated delay remains within limits, then the model can be used conveniently.

3.9 Performance Evaluation

After successful calibration and validation, the model can be considered to sufficiently replicate the field conditions. Performance is measured in terms of level of service of the intersection and quantity of vehicular emission near intersection. Level of service and quantity of vehicular emission near intersection can be found out from node evaluation result in VISSIM.

4 Study Area, Data Collection, and Analysis

Unsignalized T-intersection is selected for this study in urban area from the city of Silchar, Assam in India. Silchar city is considered to be an important city in north-eastern India after Guwahati. Urban areas in Silchar city have dense traffic composition, and due to this, frequent traffic jams occur, which create significant problems for road users. Thus, this study focused on an urban intersection in Silchar. Intersection location is shown in Fig. 1 on Google map. A video-graphic survey has been conducted for peak as well as non-peak hours. Traffic inflow, mode-distribution of vehicles have been obtained from video-graphic surveys. Desired speed distribution has also been calculated with the use of a radar gun.

4.1 Data Extraction

Extracted data consist of traffic volume, proportion of turning traffic, desired speed distribution of vehicles, length and width of vehicles, etc., which are given in Table 1.

Fig. 1 Study area

Table 1 Vehicle composition, geometry, desired speed distribution

Vehicle type	Length (m)	Width (m)	Desired speed (km/h) 85th percentile (major and minor road)		Vehicle composition (%)
			Major	Minor	
Heavy vehicle	8.65	2.34	40	–	0.62
Car	3.98	1.97	42	25	13.43
Auto-rickshaw	2.94	1.32	43	27	16.37
Motorcycle	1.90	0.85	43	27	47.20
Rickshaw	2.52	1.22	–	–	13.09
Bicycle	1.93	0.63	–	–	9.48
Flow (veh/h)					4326

Table 2 15 min flow count in peak-hour period in vehicles per hour

Movements	0–15 min	15–30 min	30–45 min	45–60 min
A–B	1776	2120	1660	2208
B–A	1372	1676	2408	1720
A–C	68	96	104	92
B–C	96	148	256	160
C–A	120	148	144	180
C–B	180	184	216	172

Traffic volumes of all vehicle classes, proportion of turning traffic have been counted manually from the videography and are given in Table 2. Average delay faced by the vehicles to clear the intersection has been taken as the measure of effectiveness. This average delay is the difference in travel time to clear the intersection area in peak and non-peak hours. As manually counting, the travel time of individual vehicles for each stream for an hour is time-consuming; only, critical movements are considered here, like right turning movement from major to minor road and two movements from minor to major road. These three movements are non-priority movements in the traffic stream and are supposed to face maximum delay. Travel time of each critical movement in a traffic stream has been calculated manually by placing markers on the video screen of the intersection area and running the video frame by frame with the use of media player classic software. The time gap between the front bumpers of any vehicle enters the intersection area and the rear bumper of that vehicle leaves the intersection area is considered as the travel time of that vehicle in a particular direction. Intersection entry and exit areas are marked on the video screen. Average travel times have been calculated for each 15 min count in an hour for peak as well as non-peak period for those three critical movements (non-priority), as shown in Table 3. This average travel time of individual stream movements (non-priority movements) is calculated by taking a sample size of 20 vehicles, and the mode split is taken as obtained from the field data. Deduction of average travel times (15 min count) of peak and non-peak hours for a particular non-priority movement gives the average field delay for that movement. Finally, average stream delay considering all non-priority movements (15 min count) has been obtained by taking the weighted average of field delay of those three non-priority movements. This process has been followed for each 15 min interval within an hour, and values are shown in Table 4.

4.2 *Development of VISSIM Model*

The factors to be considered for developing an intersection model in VISSIM are vehicle representation, geometric representation, priority rule setup, and traffic representation.

Table 3 Travel time, field delay count at 15 min interval

Movements	Average travel time during peak in seconds				Field delay in seconds				Average travel time in non-peak
	0–15	15–30	30–45	45–60	0–15	15–30	30–45	45–60	
Right turn from major	14.23	22.33	14.54	23.65	10.11	18.21	10.42	19.53	4.12
Right turn to major	14.42	20.78	14.91	26.4	10.09	16.45	10.58	22.07	4.33
Left turn to major	13.34	21.66	13.34	24.18	10.21	18.53	10.21	21.05	3.13

Table 4 Average field delay (s) of the non-priority turning streams at 15 min interval

0–15 min	15–30 min	30–45 min	45–60 min
10.12	17.56	10.43	21.13

Vehicle and geometry representation. Table 1 shows the static and dynamic variation of the vehicles present in this study. Non-motorized vehicles like cycle-rickshaw, bicycles are not available in VISSIM by default. Thus, those have been modeled according to their static and dynamic characteristics. VISSIM also requires desired speed distribution of different vehicle categories. Desired speed distribution values have been inserted in VISSIM in the form of the speed-cumulative frequency of each vehicle class. VISSIM has the google earth view by default which makes it possible to find out the location of the intersections. A preliminary network has been set up using links and connectors according to the number of lanes and width of each lane. The study intersection has a four-lane divided major road with each lane having 3.5 m width, and minor road is two-lane having both-way traffic, each lane having 2.5 m width. Right turning radius from major to minor road is measured as 20.5 m, right turning radius from minor to major road is measured as 19.5 m, and left turning radius from minor to major road is measured as 13 m. Virtual lanes are created using links as described in the study by Mathew and Radhakrishnan [3].

Priority rule configuration. Due to the absence of stop signs or yield signs in heterogeneous traffic, vehicles do not stop before entering an intersection, or vehicles wait after entering the intersection area, which cannot be modeled by using conflict area alone. Conflict area can represent the traffic which is properly regulated and disciplined. Priority rules consist of two markers, green and red [13]. Red markers have been placed in the lanes, which have to wait before passing. Green markers are conflicting markers. Priority rules have been set depending upon the relative importance of the movements. Through movement has given priority over-turning movement and right turning movement from the major road has given priority over minor road right turn movement. Priority rules have been set in terms of two parameters: (1) minimum gap time (2) minimum headway.

Traffic representation. Heterogeneous traffic has been represented in VISSIM by applying certain changes to the model [5]. Allowing smooth close-up behavior, free lane selection in case of lane change, allowing advanced merging, allowing diamond queuing, the desired position of vehicle in any position are some changes that have been done so that traffic representation looks similar to a real-world scenario.

4.3 Simulation with Default Parameters

Travel time detectors have been placed in the models as it was placed in the field. VISSIM runs for five different seeds after the setting up of the network, and average

Table 5 Parameter list, ANOVA results, calibrated values

Parameters	ANOVA			Default value	Calibrated value
	1st trial	2nd trial	3rd trial		
ax	0.09	0.04	0.11	2	2.98
bx_{add}	13E-06	0.07	0.19	2	0.081
bx_{mult}	0.11	0.27	0.13	3	0.052
min h_d	0.14	0.39	0.13	0.50	0.772
t_{dis}	0.09	0.11	0.12	60	31.37
l_v	0.19	0.22	0.21	–	–
lat min (0)	0.13	0.06	0.04	0.20	0.60
lat min (50)	0.33	0.61	0.12	–	–
lat min bus (0)	0.27	0.23	0.54	–	–
lat min bus (50)	0.24	0.33	0.71	–	–
d_c	0.19	0.21	0.24	–	–
ctg	0.22	0.41	0.21	–	–

delay value has been obtained and compared with the field value. It has been found that the error was significant; thus, calibration was necessary.

4.4 Analysis of Variance

ANOVA has been performed to find out the sensitive parameters that affect the model output. Wiedemann 74 model parameters were used for this study as explained in the VISSIM manual for urban traffic. Increasing one parameter at a time and keeping all other parameters to their default value, ANOVA has been carried out. Parameters had ANOVA values less than 0.20 taken as sensitive parameters. The results are shown in Table 5.

4.5 LHS and Multiple Linear Regression

By taking the range of the sensitive parameters from the previous research studies [3, 4], 100 datasets have been generated using LHS in MATLAB. These 100 sample sets have been given input to VISSIM model, and model output, i.e., average delay has been obtained. Regression equation has been developed by taking sensitive parameters as independent variable and absolute difference in average delay as dependent variable. Equation 1 is the regression equation that has been developed. Descriptive statistics of the regression parameters are given in Table 6.

Table 6 Descriptive statistics of all parameters of regression

Parameter	Coefficient	Standard error	t-statistics	p-value
Intercept	9.29	2.44	3.80	0.01
ax	−2.65	0.36	−7.41	5.74E−11
bx_{add}	2.03	0.36	5.68	1.51E−07
bx_{mult}	0.48	0.29	1.68	0.095
min h_d	−0.51	0.59	−0.87	0.39
lat min (0)	−1.22	1.20	−1.02	0.31
t_{dis}	0.012	0.01	1.32	0.19

$$\begin{aligned}\text{Absolute error} = &-2.65(ax) + 2.03(bx_{\text{add}}) + 0.49(bx_{\text{mult}}) \\ &- 0.52(\min h_d) - 1.23(\text{lat min}(0)) + 0.012(t_{\text{dis}}) + 9.30 \quad (2)\end{aligned}$$

4.6 Model Optimization Using GA

Equation 2 has been given input to GA toolbox in MATLAB, and parameter limits have been specified. Here, the objective is to minimize the absolute error as described in Eq. 1. Optimized parameter values have been found out and given in Table 1. After achieving the optimized parameter values, VISSIM runs have been conducted for five different seeds, and finally, MAPE has been found out. MAPE is found to be 11.99%; MAPE less than 15% is accepted as per Dowling et al. [12].

4.7 Validation

Another one-hour data for the same intersection have been used for validation purposes. Again, the field data for flow, average delay have been extracted, and the validation result shows MAPE of 14.93%, which is accepted.

4.8 Performance Evaluation

Successful calibration and validation of the model ensure that the model can be used for performance evaluation of those intersections. Performance measures that have been considered in this study are LOS and automobile emissions. A node has been created in VISSIM model such that the node covers the whole intersection area. Approximately, the polygon size that has been used is 35 m in length in each direction.

Then, VISSIM runs have been conducted, and node evaluation results in VISSIM gave the LOS values for the intersection. Here, LOS is the average LOS of different movements within the node. Average LOS has been found D for the intersection. Node evaluation results also give the estimates of the automobile emissions of carbon monoxide (CO), oxides of nitrogen (NO_x), volatile organic carbon (VOC) that have been emitted by the vehicles close to the intersection. CO, NO_x, and VOC emission values have been found to be 1185.41 g, 218.64 g, and 260.95 g, respectively. To improve the performance of the intersection, this has been converted to a signal-controlled intersection and accordingly signal phasing and signal plans have been designed.

4.9 Traffic Management Using Signal-Controlled Intersection

Signal-controlled intersection has been modeled using Webster method of signal system design [14]. A suitable phasing scheme has been chosen, as shown in Fig. 2. Green time, amber time has been given in Table 7. A fixed time signal program has been created in VISSIM using the data from Table 7. Then, signal heads have been placed near the junctions in model. After setting up the signal system, VISSSIM runs have been conducted, and different performance measures have been found out using the same node that was created previously. Average LOS found to be B and carbon monoxide, nitrogen dioxide, and volatile oxidized carbon emissions have been found out. A comparison of emission values between signal-controlled and unsignalized intersection is shown in Fig. 3. There has been observed a mild decrease in emission values of carbon monoxide and oxides of nitrogen, whereas a sharp reduction in volatile oxidized carbon values has been observed in signal-controlled intersection.

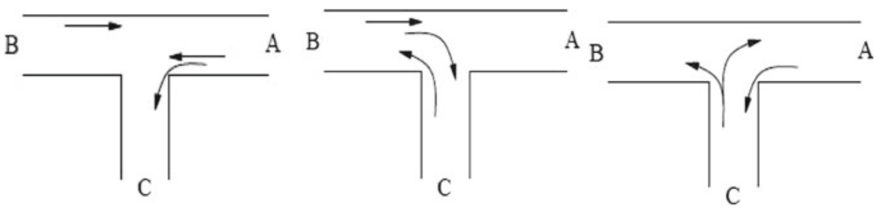


Fig. 2 Proposed signal phasing scheme

Table 7 Signal timings for three-phase signal system

Intersection	Green time (s)			Amber time (s)		
	G_1	G_2	G_3	A_1	A_2	A_3
Silchar	32	22	5	2	2	2

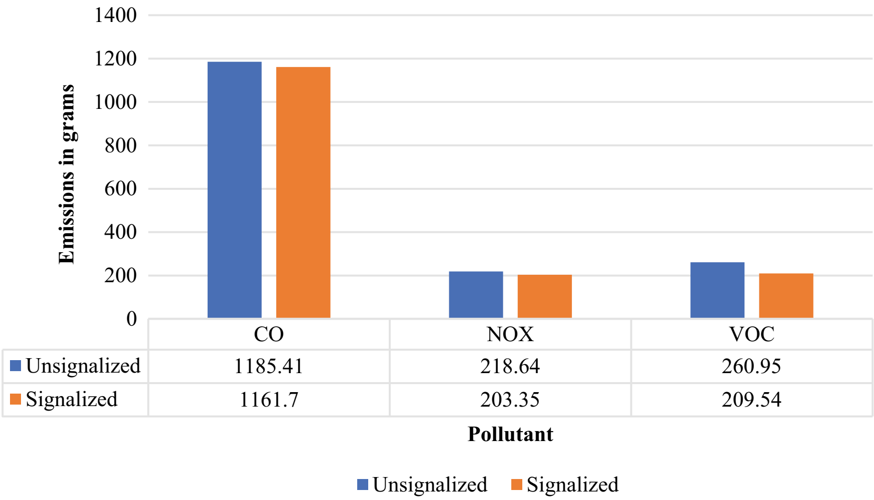


Fig. 3 Automobile emission comparison before and after improvement

5 Conclusion

This study proposed a suitable methodology for modeling unsignalized intersections in VISSIM. The most important aspect for modeling unsignalized intersections is to setting up priority rules for different movements. Calibration of simulation model involves generating latin hypercube samples to reduce the search space. It is followed by finding out the sensitive parameters using analysis of variance (ANOVA). Finally, calibration process was achieved by optimizing the parameters using genetic algorithm. Node evaluation results gave the LOS value and automobile emission values near intersection. On signalizing the intersection, the performance was improved in terms of LOS and vehicle emission. Further studies can focus on using more than one measure of effectiveness while calibrating the simulation models and analyzing other cost-effective techniques for enhancing the performance of intersections such as road bumps, raised tables, and rotary intersection.

Acknowledgements This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sectors. No other person was involved in anyway related to this research except the authors.

References

1. Chu CM, Sano K (2003) Analysis of motorcycle effects to saturation flow rate at signalized intersection in developing countries. East Asia Soc Transp Stud 5:1211–1222

2. USEPA: National Emissions Inventory (2011). https://www.epa.gov/air-emissions-inventories/national-emissions-inventory-nei. Last accessed 12 June 2021
3. Mathew TV, Radhakrishnan P (2010) Calibration of microsimulation models for nonlane-based heterogeneous traffic at signalized intersections. J Urban Plan Dev 136:59–66. https://doi.org/10.1061/(asce)0733-9488(2010)136:1(59)
4. Siddharth SMP, Ramadurai G (2013) Calibration of VISSIM for Indian heterogeneous traffic conditions. Procedia Soc Behav Sci 104:380–389. https://doi.org/10.1016/j.sbspro.2013.11.131
5. Manjunatha P, Vortisch P, Mathew T (2013) Methodology for the calibration of VISSIM in mixed traffic. Transp Res Board 92nd Annu Meet 11
6. Maheshwary P, Bhattacharyya K, Maitra B, Boltze M (2017) A methodology for calibration of vehicle class-wise driving behavior in heterogeneous traffic environment. World Conf Transp Res 00:1–15
7. Yun M, Ji J (2013) Delay analysis of stop sign intersection and yield sign intersection based on VISSIM. Procedia Soc Behav Sci 96:2024–2031. https://doi.org/10.1016/j.sbspro.2013.08.228
8. Paul B, Pitale H (2017) Comparison of capacity at signalised and unsignalised intersection. IJSTE-Int J Sci Technol Eng 3:108–111
9. Vajeeran A, De Silva GLDI (2017) Delay analysis at a signalized T intersection. In: 3rd International Moratuwa engineering research conference, MERCon 2017, pp 325–330. Institute of Electrical and Electronics Engineers Inc. (2017). https://doi.org/10.1109/MERCon.2017.7980504
10. Dutta M, Ahmed MA (2019) Calibration of VISSIM models at three-legged unsignalized intersections under mixed traffic conditions. Adv Transp Stud 48:31–46. https://doi.org/10.4399/9788255254723
11. Paul M, Charan V, Soni V, Ghosh I (2018) Calibration methodology of microsimulation model for unsignalized intersection under heterogeneous traffic conditions. Presented at the Dec 13. https://doi.org/10.1061/9780784482025.063
12. Dowling R, Skabardonis A, Halkias J, McHale G, Zammit G (2004) Guidelines for calibration of microsimulation models: framework and applications. Transp Res Rec 1–9 (National Research Council). https://doi.org/10.3141/1876-01
13. PTV (2017) PTV VISSIM 10 User manual, pp 265–297
14. Webster F (1958) Traffic signal settings. Road research technical paper, London

Modeling Phase Changing Behavior of Traffic Constables at Manually Controlled Intersection—A Case Study in India

Manish Dutta, Mokaddes Ali Ahmed, and Sanjit Srivastava

1 Introduction

Intersections are traffic facilities on road network at which multiple paths cross one another. In developing countries, traffic at intersections is difficult to regulate, mainly because of mixed nature of traffic and lack of willingness of drivers to follow traffic rules. So, intersections with heavy traffic volume are often regulated by traffic constables and are generally termed as manually controlled intersections.

The functioning of a manually controlled intersection is primarily based on intermittent stopping and allowance of vehicles to flow on traffic streams by the constable on duty. A typical layout of a three-legged manually controlled intersection is presented in Fig. 1. The constable generally controls the traffic flow at streams B–A, A–C and C–B which passes through the major conflict points. Traffic streams (A–B, B–C and C–A), passing through the minor conflict points (1, 2 and 3), are generally uncontrolled. When stream B–A has the right-of-way, flows at streams A–C and C–B are stopped. Similarly, vehicles on streams A–C and C–B are given the right-of-way one after another by obstructing the movements on the other two manually controlled streams. The traffic constable decides to allocate the right-of-way to streams B–A, A–C and C–B according to his/her judgment. So, it is very important to find out what prompts the traffic constables to change phase. In this study, a binary logistic regression has been used to model this phase changing behavior of traffic

Present Address:
M. Dutta (✉)
Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad, Gujarat 382481, India
e-mail: manish.dutta@nirmauni.ac.in

M. A. Ahmed · S. Srivastava
Department of Civil Engineering, National Institute of Technology Silchar, Silchar, Assam 788010, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_6

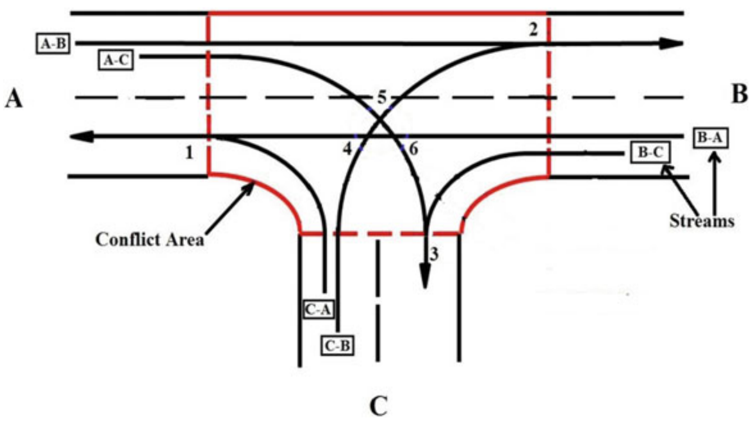


Fig. 1 Conflict area at a manually controlled intersection

constables. This is followed by estimation of critical phase times which is an estimation of the optimum duration for which a certain phase is given the right-of-way by traffic constables.

2 Literature Review

Studies on pretimed and manual traffic control show that manual traffic control performs better than automated control under oversaturated conditions if the intersection is isolated [1–3]. But pretimed signal control performs better than manual traffic control if the intersection in under-saturated [4, 5]. Some researchers were conducted assuming that constables regulate traffic using constant cycle lengths and phase splits [1, 4, 5]. Studies conducted by Marsh revealed that the manual traffic control is efficient due to variable cycle length and phase splits [6]. Hakkert and Gitelman studied the process of traffic regulation at manually controlled intersections and recommended a set of guidelines for systematic monitoring of the performance of traffic constables [7]. Simulation of manual traffic control considering fixed cycle length will oversimplify the field scenario which can deliver unrealistic results [8]. Parr and Wohlson modeled the phase change behavior of manual traffic-signal control during emergencies and special events using logistic regression [8]. So et al. has done a comparative analysis of the performance of manual traffic-signal control and optimized signal control at oversaturated intersections [9]. The results showed that there is a statistically significant reduction in delay if the intersections are controlled manually. Studies conducted by Ding et al. [10] and Parr and Wolshon [8] also lead to the same conclusion that manual traffic-signal significantly improves the performance of intersection.

The review of existing studies reveals that studies on phase change modeling at manually controlled intersections is very scarce. The authors could not find any studies on phase change modeling that was conducted at manually controlled intersections under mixed traffic conditions. Hence, establishing a suitable methodology for phase change modeling at such intersections is considered essential to understand the phase changing behavior of traffic constables in India.

3 Methodology

3.1 Phase Change Model

The phase changing decisions made by constables while directing traffic has been modeled using binary logistic regression. A timeline of events, which took place while regulating traffic, is prepared on a second-to-second basis. The time interval considered is one second. The traffic constable allocates the right-of-way to the streams passing through the major conflict points in three phases. The three phases are named as primary, secondary and tertiary based on the priority set by the traffic constable. The traffic constable on duty has a utility for providing the right-of-way to a phase. If the constable switches from primary phase (say) to the secondary phase, vehicle starts queuing up at the stream having the primary phase, but vehicles on secondary phase stream will be released. If the constable chooses to continue the primary phase, flow of vehicles on the corresponding stream will continue. But, vehicles on the other secondary and tertiary phase streams will have to wait further and more vehicles will join the queues. A simple utility function to represent this situation is as shown in Eq. (1).

$$U_i = V_i + \varepsilon_i \tag{1}$$

where U_i = total utility, V_i = observed utility, and ε_i = unobserved portion of the utility (error). The observed or deterministic component of utility, V_i, is a function of the factor(s) that affect the phase changing decisions made by traffic constables. The two factors considered in this study are time and gap. 'Time' refers to the duration for which a phase receives the right-of-way. 'Gap' is a binary variable which takes the value of 0 if platoon starts breaking up, and 1 if it is not. The utility equation for the phase change model is taken as shown in Eq. (2).

$$\ln \frac{P(t)}{1 - P(t)} = V_i = \alpha + \beta.t + \gamma.g \tag{2}$$

where

$P(t)$	probability that the constable will change the current phase
V_i	observed or deterministic component of utility

α, β, γ unknown coefficients to be calibrated
t phase time (s)
g gap (0 or 1).

The phase change models are calibrated separately for the three phases to estimate the critical phase times. Critical phase time is explained in Sect. 3.3. The phase change models for the three phases are given by Eqs. 3–5. The models are calibrated using 75% of the randomly selected data and the rest are used for validating the models.

$$\ln \frac{P(t)}{1-P(t)} = V_p = \alpha_p + \beta_p.p + \gamma_p.g_p \tag{3}$$

$$\ln \frac{P(t)}{1-P(t)} = V_s = \alpha_s + \beta_s.s + \gamma_s.g_s \tag{4}$$

$$\ln \frac{P(t)}{1-P(t)} = V_t = \alpha_t + \beta_t.t + \gamma_t.g_t \tag{5}$$

where

$P(t)$ probability that the constable will change the current phase
V_p, V_s, V_t observed or deterministic component of utility of primary, secondary and tertiary phase
α_p, α_s, α_t, β_p, β_s, β_t, γ_p, γ_s, γ_t unknown coefficients to be calibrated
p primary time (s)
s secondary time (s)
t tertiary phase time (s)
g_p gap variable for primary phase (0 or 1)
g_s gap variable for secondary phase (0 or 1)
g_t gap variable for tertiary phase (0 or 1).

3.2 Phase Change Model Validation

The calibrated phase change models are used to estimate the probability of changing a phase. The probability of phase change is rounded off to zero if it is less than 0.5, and if the probability value is found to be more than or equal to 0.5, it is rounded off to 1. This is done to compare the phase change decisions predicted by the models and the actual decisions made by traffic constables. The models are validated using Type I error, Type II error, sensitivity and specificity. Type I error is said to have occurred when the null hypothesis is rejected, but it is, in fact, true. Type II error occurs when a null hypothesis is accepted, but it is, in fact, false. Sensitivity (one minus type II error) represents the ability of a model to identify correctly whether the traffic constable has changed the current phase at a particular time, whereas specificity (one minus type I error) shows the ability of a model to identify correctly whether the traffic constable has not changed the current phase; i.e., he/she continues with the current phase.

3.3 Determination of Critical Phase Times

A phase change model provides relationship between phase time and probability of changing the phase. The duration of a certain phase is considered to have reached a critical value when there is an equally likely chance (probability = 0.5) that the constable will change or not change the phase. This duration of the phase is known as critical phase time. Since, there are three phases in this study, the critical phase times are named as critical primary time (p_{cr}), critical secondary time (s_{cr}) and critical tertiary time (t_{cr}). The critical phase times are determined by putting the value of probability as 0.5 in the phase change models (Eqs. 3–5).

4 Case Study

A manually controlled intersection in Barasat, West Bengal has been chosen as the case study intersection (Fig. 2). The data were collected using videographic survey for three working days during morning peak hours (10:00 a.m.–12 noon). All the approaches at the intersection are two-lane undivided. The camera was placed at a vantage point on an adjacent building so that all the approaches can be viewed. A

Fig. 2 Study intersection with legs named as A, B and C and position of traffic police marked in red

Table 1 Traffic composition of intersection A

Streams	Vehicle types						
	TW	AR	SPC	LCV	HV	BC	CR
AB	18.49	18.49	23.97	2.74	8.22	17.12	10.97
BA	9.9	24.75	16.83	5.94	14.85	15.84	11.89
AC	16.76	3.91	7.26	1.68	3.91	35.75	30.73
CB	17.54	7.02	14.04	1.75	31.58	10.53	17.54
CA	17.09	6.84	5.98	0.85	2.56	38.46	28.22
BC	24.14	17.24	13.79	3.45	6.9	24.14	10.34

total of 38 cycles were used for extracting the phase change data required for this study. The vehicles flowing through the intersections are classified into six categories based on their dimensions and operational characteristics. The stream-wise vehicular compositions are presented in Table 1.

4.1 Calibration of Phase Change Model

Field observation shows that stream B–A receives the right-of-way for maximum time, followed by streams A–C and C–B. So, the phases of streams B–A, A–C and C–B are named as primary, secondary and tertiary, respectively. The binary variable, gap (g), was not found to be significant because most of the time the constables switch to the next phase before the platoon starts breaking up. The traffic constables do so because the intersection has very high traffic volume. So, giving the right-of-way to a certain stream for a long time becomes impractical to avoid excessively long queue formation on the other manually controlled streams. Thus, constables are prompted to change phase before the flow starts breaking up on the stream that is having the right-of-way.

Logistic regression was performed to model the phase change behavior of traffic constables. The statistical results of the phase change models for streams B–A, A–C and C–B are reported in Tables 2, 3 and 4. The results indicate that with increase in phase time, the probability that the constable will change phase in the next second

Table 2 Descriptive statistics for the phase change model, primary phase

Coefficient	Coefficient	Standard error	Wald statistic	p-value
Intercept	−7.46	1.51	46.52	0.00
p	0.07	0.09	15.41	0.00
McFadden $R^2 = 0.71$	Log-likelihood function = −42.65		% of right predictions = 97%	

Table 3 Descriptive statistics for the phase change model, secondary phase

Coefficient	Coefficient	Standard error	Wald statistic	p-value
Intercept	−5.48	0.85	39.64	0.00
s	0.16	0.07	12.42	0.00
McFadden $R^2 = 0.68$	Log-likelihood function = −45.63		% of right predictions = 95%	

Table 4 Descriptive statistics for the phase change model, tertiary phase

Coefficient	Coefficient	Standard error	Wald statistic	p-value
Intercept	−6.48	1.15	25.61	0.00
t	0.29	0.05	15.48	0.00
McFadden $R^2 = 0.75$	Log-likelihood function = −25.47		% of right predictions = 93%	

keeps increasing. This happens because when a certain phase has got the right-of-way for a long time, queues build up at the other steams. As a result, the probability for the constable to change the current phase increases.

4.2 Validation of Phase Change Model

The phase change models are validated with 25% of the data collected. The prediction success tables for the phase change models are presented in Tables 5, 6 and 7. Type

Table 5 Prediction success table of the phase change model (primary phase)

	Success observation	Fail observation	Total
Success prediction	14	15	29
Fail prediction	2	881	883
Total	16	896	912
Sensitivity = 0.88		Specificity = 0.98	
Type II error = 0.12		Type I error = 0.02	

Table 6 Prediction success table of the phase change model (secondary phase)

	Success observation	Fail observation	Total
Success prediction	14	11	25
Fail prediction	2	403	405
Total	16	414	430
Sensitivity = 0.88		Specificity = 0.97	
Type II error = 0.12		Type I error = 0.03	

Table 7 Prediction success table of the phase change model (tertiary phase)

	Success observation	Fail observation	Total
Success prediction	14	14	28
Fail prediction	1	396	397
Total	15	410	425
Sensitivity = 0.93		Specificity = 0.96	
Type II error = 0.07		Type I error = 0.04	

Table 8 Critical phase times (s) obtained from phase change models

Phase change models	Critical phase times (s)
$\log_e [p/(1 - p)] = -7.46 + 0.07p$	107
$\log_e [p/(1 - p)] = -5.48 + 0.16s$	34
$\log_e [p/(1 - p)] = -6.48 + 0.29s$	22

I and Type II errors are found to be less than 0.25, whereas the values of sensitivity and specificity are above 0.85 for all the models. This shows that the model performs reasonably well in predicting the phase change behavior of traffic constables.

4.3 Calculation of Critical Phase Times

The critical phase times are calculated by setting the probability to 0.5 in the phase change models. The phase change models obtained from Tables 2, 3 and 4 and the estimated values the critical phase times are shown in Table 8. Critical primary time is found to be the highest followed by critical secondary time and critical tertiary time. This is obvious because of the fact that primary phase gets the right-of-way for maximum duration followed by secondary and tertiary phase.

5 Summary and Conclusion

A methodology for modeling the phase changing behavior of traffic constables at manually controlled intersections has been presented in this study. Manually controlled intersections under Indian road conditions are characterized by variable phase times which depend on the decisions made by traffic constables on the field. Phase change models are developed to quantify this behavior using binary logistic regression and critical phase times are estimated. The critical primary time, critical secondary time and critical tertiary time are found to be 107 s, 34 s and 22 s, respectively. Key insights from this study are: (1) the phase changing decision of traffic constables depends only on the duration of the current phase, (2) the values of critical phase times show that the three manually controlled streams do not have the same duration of right-of-way in a cycle, and (3) the critical phase times give an estimate about the optimum duration for which traffic constables assign the right-of-way to each phase. These values will in turn aid in estimating the capacity and delay at manually controlled intersections.

References

1. Mahalel D, Gur Y, Shiftan Y (1991) Manual versus automatic operation of traffic signals. Transp Res Part A Gen 25:121–127
2. May AD, Montgomery FO (1986) Control of congestion at highly congested junctions. Transp Res Rec J Transp Res Board 42–48
3. Sutermeister O (1956) Capacities of narrow streets with manual control and signal control. Highw Res Board Bull 1–15
4. Pretty RL (1974) Police control of traffic: a study at a Brisbane intersection. In: Australian Road Research Board (ARRB) conference, pp 83–95, Adelaide
5. Ye Z, Veneziano DA, Lassacher S (2009) Analysis of manual traffic control at all-way stop-controlled intersections during special events. In: Transportation research board 88th annual meeting. Transportation Research Board, Washington D.C.
6. Marsh BW (1927) Traffic control. Ann Am Acad Pol Soc Sci 133:90–113
7. Hakkert S, Gitelman V (2005) Measuring the efficiency and effectiveness of traffic police operations: developing guidelines for a systematic monitoring of police enforcement. In: Australasian Road Safety Research Policing Education conference, Wellington, New Zealand
8. Parr S, Wolshon B (2016) Methodology for simulating manual traffic control. Transp Res Rec J Transp Res Board 9–17
9. So J, Lee J, Park B (2013) Evaluation of manual traffic-signal control under oversaturated conditions using hardware-in-the-loop simulation. J Transp Eng 139:1068–1075
10. Ding N, He Q, Wu C, Fetzer J (2015) Modeling traffic control agency decision behavior for multimodal manual signal control under event occurrences. IEEE Trans Intell Transp Syst 16:2467–2478

Cost-Effective Analysis for Electric Vehicle (EV) Bus Procurement

Ashish Verma, T. V. Ramanayya, Hitendra Trivedi, and Mahim Khan

1 Introduction

A well-developed transportation system is an indicator of economic growth of any nation. The system has evolved through centuries and the continuous technological advancements are driving it toward its advanced configuration. In recent developments, the introduction of electric vehicles (EV) can potentially redefine the system due to its cost-effectiveness and sustainability. Many governments across the world are considering the EV buses as an alternative over conventional buses to reduce emissions.

It is interesting to know that the development of electric vehicles has been going on since the nineteenth century, in fact, the first rudimentary model of the electric vehicle was introduced by Ányos Jedlik in the year 1828. The first petrol powered Internal Combustion Engine Vehicle (ICEV) were introduced in 1885 which is typically over 5 decades later to the electric vehicles. Later till 1930, the ICEV became much more popular and dominated the automobile industry as compared to the electric vehicles. The high operating cost played an important role in the decline of the electric vehicles and by the year 1935, the electric vehicles almost disappeared from the market [1]. However, the rising price of fuel, environmental issues, and technological developments has given a chance of revival to the electric vehicles.

India being the third largest vehicle market in the world and with the current economic growth, the annual vehicle sale for the year 2020 is estimated as 10 million for passenger vehicles and 2.7 million for commercial vehicles [2]. The growth

A. Verma (✉) · H. Trivedi · M. Khan
IISc Sustainable Transportation Lab. (IST Lab.), Department of Civil Engineering, Indian Institute of Science (IISc), Bangalore, Karnataka 560012, India
e-mail: ashishv@iisc.ac.in

T. V. Ramanayya
Indian Institute of Management Bangalore, Bangalore, Karnataka, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_7

of vehicle sales will result in an increase in fuel demand of the country, which is majorly fulfilled through the imports. Consequently, governments across the world are striving to promote electric mobility to decrease their expenditure for fossil fuel consumption and reduce greenhouse gas emissions.

2 Background of the Study

The Government of India has also launched "National Electric Mobility Mission Plan (NEMMP) 2020" to achieve national fuel security by promoting hybrid and electric vehicles in the country. This mission aims to pave the way for widespread deployment of electric and hybrid vehicles in India through the state government organizations. As a part of the mission, Department of Heavy Industries and Public Enterprises (DHI) has formulated a scheme namely FAME-India (Faster Adoption and Manufacturing of Hybrid and Electric vehicles in India) to provide the financial assistance for procurement of EVs. Under this scheme, the procurement of EV bus is subsidized through financial incentives up to 60% (Max 10 Million INR) of the vehicle benchmark price. Various "State Transport Undertakings (STUs)" have initiated the procurement process for EV Buses and to avail the incentives, the procurements can be done through Gross Cost Contract (GCC) Model or Outright Purchase Model. Therefore, it is imperative to analyze the cost-effectiveness for both the models to opt for one of them. The present study is conducted with a case of Bangalore Metropolitan Transport Corporation (BMTC), which is a state government agency in Karnataka, India responsible for public transport bus service in the city Bengaluru, Karnataka, India. Table 1 concisely exhibits the performance of BMTC in terms of operations and financial indicators for the past years.

The scale of operations of BMTC are comprehended in Table 1, and it can be easily observed that the switching of conventional buses to the electric buses will have a significant impact on the operational and financial performance of the organization. Analogous challenges will be faced by other similar organizations which have proposed to undergo such changes in future (Fig. 1).

The cost of operations and maintenance of EV buses have varied significantly in the past and is expected to change further with the technological development and economy of scale. Similarly, the continuous increase in the cost per kilometer is a common challenge for the state transport undertakings and economics of the same is expected to be redefined with the replacement of the conventional buses with the electric buses. Therefore, it is important to take efficient decisions pertaining to long term financial viability for the sustainability of STUs.

Table 1 Key financial and operational indicators of BMTC

S. No.	Parameter	Financial years				
		2013–14	2014–15	2015–16	2016–17	2017–18
1	Schedules operated	6473	6244	6216	6219	6143
2	Vehicles held	6775	6522	6401	6161	6677
3	Effective km/day (in INR lakhs)	13.14	12.9	12.21	11.52	11.42
4	Rate of accidents (per lakh km)	0.07	0.08	0.07	0.07	0.07
5	Staff position	36,079	36,474	35,554	34,306	34,114
6	Effective km (in INR lakhs)	4795.9	4708.56	4469.82	4205.2	4164.53
7	Gross revenue (in INR lakhs)	201,394.2	225,684.4	220,748.4	210,610.4	222,699.5
8	Cost of operation (in INR lakhs)	216,153.1	232,174.8	219,375.7	236,701.4	244,461
9	Margin on gross revenue (in INR lakhs)	−14,758.2	−6490.38	1372.66	−26,091	−21,761.5
10	EPKM on gross rev. (in INR paise)	4199.3	4793.1	4938.6	5008.3	5347.5
11	CPKM (in INR paise)	4507	4930.9	4907.9	5628.8	5870.1
12	Margin on gross rev. (in INR paise)	−307.7	−137.8	30.7	−620.4	−522.5

Source BMTC; retrieved from http://mybmtc.karnataka.gov.in/info-4/Perfomance-Indicator/en

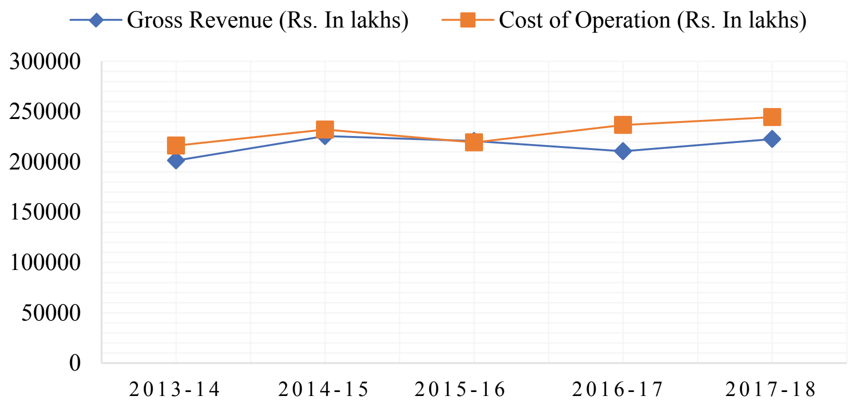


Fig. 1 Gross revenue versus cost of operations for BMTC

3 Methodology

The objective of the paper is to propose and develop a framework which can be used to calculate and compare the cost-effectiveness of EV bus procurement models, which can assist decision makers to identify and choose the cost-efficient option among them. The following steps briefly explain the methodology adopted for the present study:

- Identification of cost components for Gross Cost Contract and Outright Purchase model.
- Modeling the cost components for overall cost calculation.
- Formulation of different scenarios based on diverse parameters.
- Cost calculation for each of the scenario for both the procurement model.
- Analysis of the pattern of the overall cost for different scenarios.
- Finding benchmark running KMs to propose model selection criteria.

The cost component of overall cost can be classified as fixed and variable cost components which are related to procurement, operations, and maintenance. It is important to understand that the proposed framework is not aimed to calculate the cost of operations for both the models but to compare the differences in overall cost to determine the efficient model. Therefore, the cost components were broadly identified as factors contributing differences in overall cost and the factors which were similar in both the models. Cost factors having similar values for both the models such as infrastructure cost, depreciation on other assets were ignored for modeling as their net effect on cost comparison would be null.

The cost of procurement and operations in the Gross Cost Contract (GCC) Model is typically comprised of only variable cost whereas the Outright Purchase Model has various cost of fixed and variable nature. For the purpose of the study, the fixed and variable cost for outright purchase model was derived through the data of the Bangalore Metropolitan Transport Corporation (BMTC) for the year 2017–18. Some of the data as vehicle life and the interest rate on financing are assumed as per inputs from industry experts and BMTC officials. The data presented in the paper is indicative in nature and can differ for other STUs.

To determine the overall cost of procurement and operations, 9 scenarios are assumed with different quoted cost per kilometer (CKPM) and average daily running kilometers to exhibit the trend with different scenarios. All the cost components were translated into per kilometer cost and added to derive the overall cost per kilometer for both the models. The cost output for both the models is examined to ascertain the benchmark kilometers to depict the trend in different scenarios. The changing pattern of benchmark kilometers is assessed to derive benchmark kilometers for different scenarios.

4 Major Components of Procurement and Operational Cost

4.1 Procurement and Operational Cost in Outright Purchase Model

The overall cost is comprised of procurement cost (purchase cost, cost of financing, etc.) and the cost of operations (staff, maintenance, etc.), which are completely borne by the STUs under the outright purchase model.

4.1.1 Initial Purchase Cost

The initial purchase cost for EV bus typically varies from 70 Lakhs to 1.5 Crore depending on the specifications of the bus. The initial purchase cost of the bus for any department will depend on the model of the bus proposed for the procurement and the supplier. As a standard procedure the procurement, the organizations issue a call for a bid for the supply of bus as per their specific requirements. However, as a public sector undertaking the STUs can apply and avail financial assistance from various schemes of state and central government. For the purpose of the study, the cost of one bus is assumed as 1 crore and assumed that STUs will avail 60% subsidy as per the existing FEMA India scheme.

4.1.2 Loan Repayment

For the cost calculation for outright purchase model, it is assumed that the net cost of bus purchase will be financed through a loan. The net purchase cost can be derived by subtracting government financial incentive/subsidy from the initial purchase cost. The loan duration was assumed 10 years (equal to vehicle life) at 11% annual interest rate. Equal monthly installments were calculated and translated into the per kilometer cost by dividing it through monthly running kilometers as per the assumed scenarios.

4.1.3 Cost of Infrastructure

Infrastructure development such as the development of additional depots, in case of EV bus-developing a charging infrastructure, etc. will surely incur a cost to the operating organization. Since the cost of infrastructure in both the procurement model is usually borne by the operating organization, it would not affect the objective of the study to compare the cost-effectiveness for the procurement models. Therefore, the cost of infrastructure and other common cost are not included in the calculations for the present study.

4.1.4 Taxes

Various taxes are imposed on the expenditures related to purchase and operations of the buses. These taxes are imposed at different rates and may vary according to state and central government policies. In context of the present study, data for taxes was taken from BMTC data for the year 2017–18.

4.1.5 Battery Replacement Cost

The battery replacement cost is an important component of electric bus operations. Since the conventional buses utilize diesel or petrol as fuel but the electric buses rely on the batteries and the life span of battery depends on the number of cycles a battery is being charged. The charging time and vehicle running capacity depends on the battery specifications. Lifespan of a battery can be assumed approximately 2000 battery cycles, if charged once a day, one battery can efficiently perform for approximately 5.5 years [3]. Therefore, the battery replacement is expected to be five years after the year of procurement. Continuous fall in the battery price can be witnessed from the past trends and is expected to decline further in future (Fig. 2). For the study we assumed the procurement year as 2020, which means that the battery replacement is expected in the year 2025 or 2026. According to Bloomberg NEF Report, the price of the battery is projected to reach below 100$/KWh by 2025 [4].

Considering the present USD value as 71 INR, the battery cost/kWh is assumed as 7100 INR. Therefore, the cost of a battery with 180 kWh capacity would cost Rs. 1,278,000 in the year 2025. Since the battery replacement is proposed after approximately 5 years, the per kilometer battery replacement cost is derived based on average daily running kilometers as per different scenarios.

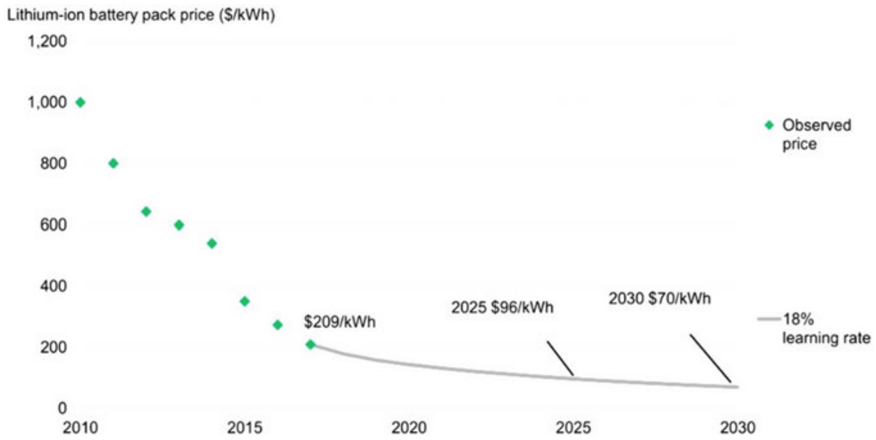


Fig. 2 Projected price drop of batteries of EVs [4]

4.1.6 Staff

The cost of all types of staff is completely borne by the organization in outright purchase model whereas in GCC model the cost of staff is borne partially. Typically, the cost of drivers, maintenance staff is included in the GCC per kilometer cost which can vary as per the conditions of vendor and client organization. Therefore, the incurred cost of staff would be different in case of GCC and outright purchase model.

4.1.7 Vehicle Depreciation

In case of outright purchase, the vehicle will be owned and operated by the STU itself, therefore, as per usage, the vehicle will depreciate over time. Typically, it is assumed that a conventional bus has a life of 10 lakh kilometers with a maximum span of 10 years. The same is assumed for the electric bus as well and thus the life of a bus will depend as per the daily usage of the vehicle and similarly per kilometer cost of vehicle depreciation is calculated.

4.1.8 Other Operational and Maintenance

Other costs such as maintenance cost, cost of spares, reconditioning cost, cost of tires, lubricants and other consumables, insurance or accidental compensation, etc. are derived from the historical data of BMTC and added together to cover these cost of operations which does not incur under the GCC model.

4.2 Procurement and Operational Cost in GCC Model

4.2.1 Quoted Cost Per Kilometer

The cost of procurement and operations through GCC model is simple and contain lesser components. Under the GCC model, the vendor provides a quote for overall operations of the bus on per kilometer basis which usually covers the cost of procurement, operations, and maintenance. The per kilometer cost can vary based on various factors such as contract conditions, bus specifications, minimum assured kilometers, number of buses deployed, etc. For the present study, various scenarios are assumed to depict the impact on overall cost borne by the STUs in different conditions.

4.2.2 Tax on the Quoted Per Kilometer Cost

The quoted per kilometer cost may or may not be inclusive of the applicable taxes. The payment of the taxes can vary with the conditions of the agreement. In general, the quoted per kilometer price is exclusive of the tax and it attracts 18% GST at the time of payment which is a significant percent of the payment to the vendor.

4.2.3 Staff

Since the maintenance and operational staff are provided by the vendor, only the ticket conductor (for revenue collection) is employed by the STUs. Therefore, the cost of staff is lesser in the case of GCC model as compared to outright purchase model.

4.3 Cost Calculations for GCC and Outright Purchase Model

Considering the above-mentioned cost components, the overall cost of procurement and operations for GCC model and outright purchase model can be expressed in the following equations:

$$\mathrm{CPKM}_{(\mathrm{GCC})} = \mathrm{QCPKM} + \mathrm{TAX}_{(\mathrm{GCC})} + \mathrm{Staff}_{(\mathrm{GCC})} \tag{1}$$

$$\mathrm{CPKM}_{(\mathrm{OP})} = \mathrm{BR} + \mathrm{VD} + \mathrm{LR} + \mathrm{OOM} + \mathrm{TAX}_{(\mathrm{OP})} + \mathrm{Staff}_{(\mathrm{OP})} \tag{2}$$

$$\mathrm{BR} = \frac{\mathrm{EBC}}{(\mathrm{BL} * \mathrm{ARKM})} \tag{3}$$

$$\mathrm{VD} = \frac{\mathrm{TVC} - \mathrm{GFI}}{(\mathrm{VL} * \mathrm{ARKM})} \tag{4}$$

$$\mathrm{LR} = \frac{\mathrm{ARA}}{(\mathrm{VL} * \mathrm{ARKM})} \tag{5}$$

The nomenclatures used in the equations are explained as follows:

$\mathrm{CPKM}_{(\mathrm{GCC})}$	Cost per kilometer for GCC model
QCPKM	Quoted cost per kilometer by the vendor for GCC model
$\mathrm{Tax}_{(\mathrm{GCC})}$	Tax applicable on QCPKM for GCC Model
$\mathrm{Staff}_{(\mathrm{GCC})}$	Staff Cost applicable for GCC Model
$\mathrm{CPKM}_{(\mathrm{OP})}$	Cost per kilometer for Outright Purchase model
BR	Battery Replacement Cost
EBC	Expected Battery Cost

BL	Battery Life (in years)
ARKM	Annual Running Kilometers
VD	Vehicle Depreciation
TVC	Total Vehicle Cost
GFI	Financial Incentive by Govt.
VL	Vehicle Life (in years)
LR	Loan Repayment
ARA	Annual Repayment Amount
OOM	Other Operational and Maintenance Cost
$Tax_{(OP)}$	Tax applicable for Outright Purchase Model
$Staff_{(OP)}$	Staff Cost applicable for Outright Purchase Model

To depict the variation on the overall cost, various scenarios were assumed considering different average daily running KMs and quoted CPKM for GCC model and the cost calculations are done for both the procurement models (Table 7). To manifest, the cost calculations Tables 2 and 3 elaborate the cost components for both the models and show how the calculations are done for scenario 1 which is assumed by considering average daily running kilometer as 150 KMs and Quoted CPKM for GCC model as 30 Rs./KM.

The overall cost of procurement and operations under GCC model was found to be Rs. 46.07 per kilometer whereas, for outright purchase model, it was found as 58.80. However, it should be noted that the objective of these cost calculations is to exhibit how the model can be used to calculate the cost for both the models. The cost components and their values can vary for different STUs and other organizations.

Table 2 Per km cost in GCC model (for scenario 1)

S. No.	Cost component	Rs./KM	Remarks
1	CPKM quoted	30	As per scenario assumption
2	$Tax_{(GCC)}$	5.4	Assumed 18% GST on CPKM
3	$Staff_{(GCC)}$	10.67	From BMTC data
	Total	46.07	

Table 3 Per km cost in outright purchase model (for scenario 1)

S. No.	Cost component	Rs./KM	Remarks
1	Battery replacement	4.67	Refer Table 4
2	Vehicle depreciation	7.30	Refer Table 5
3	Loan repayment	12.08	Refer Table 6
4	Other operations and maintenance	4.68	From BMTC data
5	$Tax_{(Outright\ Purchase)}$	2.33	From BMTC data
6	Staff cost	27.74	From BMTC data
	Total	58.80	

Table 4 Battery replacement cost calculations

S. No.	Component		Remarks
1	Battery life (in years)	5	Assuming 2000 charging cycles
2	Daily running KMs	150	As per scenario assumption
3	Battery cost (INR)	1,278,000	Refer to Fig. 2
4	Total KMs in 5 years	273,750	Battery life * annual running KMs
5	Per km cost of replacement	4.67	Battery cost/total KMs in 5 years

Table 5 Vehicle depreciation cost calculation

S. No.	Component		Remarks
1	Total vehicle cost	10,000,000	Assumption made for calculations
2	Financial incentive by Govt	6,000,000	As per FEMA scheme (60%)
3	Net vehicle cost	4,000,000	Total vehicle cost—financial incentive by Govt
4	Vehicle life (in years)	10	Assumption made for calculations
5	Daily running kms	150	As per scenario assumption
6	Total KMs in 10 years	547,500	Vehicle life * daily running KMs * 365
7	Per km depreciation	7.3	Net vehicle cost/total KMs in 10 years

Table 6 Loan repayment calculations

S. No.	Component		Remarks
1	Net vehicle cost	4,000,000	Refer Table 5
2	Interest on loan (%)	11	Assumption made for calculations
3	Loan term (in years)	10	Assumed vehicle life for 10 years
4	Annual repayment amount	661,200	Calculated as per interest rate
5	Daily running KMs	150	As per scenario assumption
6	Annual KMs	54,750	Daily running KMs * 365
7	Per KM cost of loan repayment	12.08	Annual repayment amount/annual KMs

The framework can be modified as per their specific contract conditions requirements and the cost can be derived for further comparison. The subsequent Tables 4, 5, and 6 explain the individual cost component calculated for outright purchase model.

As shown in Table 7, the overall cost per kilometer is calculated for both the models considering 9 different scenarios. These scenarios were formulated by varying two parameters of average daily running kilometers and quoted cost per kilometer for GCC model. As it can be observed easily that the average daily running kilometer plays a significant role in overall per kilometer cost for outright purchase model whereas the quoted cost per kilometer affects the overall per kilometer cost for

Table 7 Scenario-wise summary of cost for both models

Scenario No.	Average daily running KM	Quoted CPKM for GCC model	Overall Per KM cost for GCC model	Overall Per KM cost for outright purchase model	Benchmark KM
1	150	30	40.07	58.8	319
2	200			52.79	
3	250			49.18	
4	150	35	51.97	58.8	210
5	200			52.79	
6	250			49.18	
7	150	40	57.87	58.8	156
8	200			52.79	
9	250			49.18	

GCC model. To decide the cost-effectiveness of a procurement model, the STUs need to identify the benchmark average daily running kilometer where the cost for both these models is same. As it is clearly visible from the decreasing trend of overall per kilometer cost for outright purchase model with increase in the daily average running kilometers that, if the average daily running kilometers are above the benchmark kilometer for the specific quoted CPKM, the outright purchase model will be cost-efficient whereas with less average daily kilometers the GCC model will be cost-efficient.

5 Conclusion

The present study has attempted to develop a model considering major cost components as input to exhibit the trend of cost and expenditure in the long run for the STUs. The output derived from the proposed framework for assessment of the overall cost of procurement and operations can help the decision makers to understand and assess the overall cost for both the models. The model is developed in a way that the output can provide a benchmarking number of kilometers which can act as a decision criterion for selection of a cost-effective model for vehicle procurement. The cost output for additional scenarios can demonstrate the sensitivity of various other cost parameters/components with the overall cost.

Concisely, the trend for benchmark KMs is shown in Fig. 3 which exhibits the inverse relationship with the quoted CPKM. With the help of the derived trend, different benchmark KMs can be derived for different quoted cost per kilometer. Ceteris paribus, it can be inferred from Fig. 3 that if the quoted cost per kilometer for GCC model is Rs. 30/KM then the GCC model would be cost-efficient if the

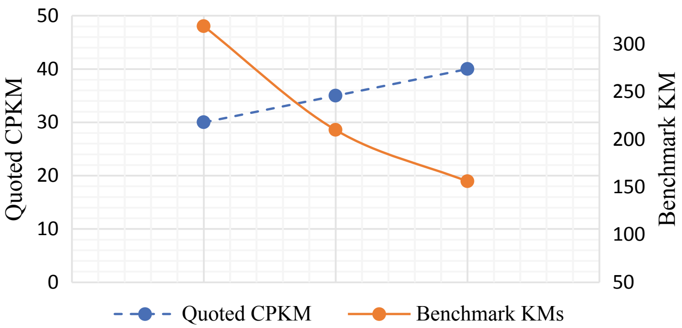


Fig. 3 Benchmark KMs for different quoted CPKM

average daily running KMs are below 319 km and vice versa. Similarly, if the quoted cost per kilometer is Rs. 40/KM, then the GCC model would be cost-efficient if the average daily running kilometers are less than 156 KMs, and if the average daily running kilometers are above 150 KMs, then the outright purchase model would be cost-efficient. The values for benchmark KMs can be derived for different quoted CPKM and vice versa. It is clearly visible that the quoted CPKM and benchmark KMs possess an inverse relationship depicted by the decreases in benchmark KMs with the increase in the quoted cost per kilometer. Further, the benchmark kilometer will change with the change in values of other cost input parameters.

The current government policies advocate an urgent need for all bus undertakings to shift to electric vehicles in next 10 years. In light of this, weightage is to be provided for additional factors rather than pure cost considerations or the benchmark kilometers. The additional factors which are required to be considered while choosing the mode of procurement of EV bused are briefly mentioned below:

- The STUs may not be able to utilize the complete life of the vehicle i.e., 10 lakh KMs during the contract duration. The remaining vehicle life and the scrap value of the vehicle remains to the STUs in outright purchase model.
- It is common knowledge that the fares will not remain constant for a long run. Therefore, the long-term contacts will have influence on quoted CPKM and revenue of STUs over time.
- Alternative payment methods as revenue linked payment in GCC model and flexible contract duration for better utilization of vehicle life can influence the financial viability of procurement model in long term.
- Usually, the STUs keep a fixed percent or fixed number of buses in reserve to take care contingencies like breakdown or accidents. This factor should also be considered in the decision making.
- Further, the possibilities of complex operational situations such as insurance or accidental compensation like in case of accident if the driver (provided from vendor in GCC) or conductor (provided form STUs) or both are responsible,

which organization will bear the cost on different conditions. Such factors also need careful consideration while taking a decision on procurement model.

6 Policy Implications and Recommendations

The study reveals two very important points from the perspective of the policy and decision makers:

- Same quoted CPKM can be cost-effective or expensive depending upon the daily average running kilometers.
- A minor per kilometer cost difference can become immense in long term and can substantially affect the economic health of the STUs e.g., cost difference of 1 Rs./KM will annually become Rs. 7.3 million for 100 buses running 200 KMs daily.

Further, the financial viability for bus operations is also dependent on the minimum daily assured kilometer criteria, which should be compared with the benchmark kilometers. It could be possible that the cost of procurement and operations is less in the GCC model, but due to minimum daily assured kilometer criteria, the payment expenditure could exceed the overall cost. Therefore, it is recommended that the decision should consider the benchmark kilometers derived as per the cost components, contract conditions, past operational trends for the respective STUs, and other factor influencing the organization in long term.

7 Limitations and Future Scope

The objective of the present study was to showcase a framework which can be used to calculate the overall cost of procurement and operations of EV buses for state transport undertakings. The values of various cost components undertaken for overall calculations were taken from the BMTC data for the year 2017–18. To improvise the accuracy of these parameter values, the values can be derived by observing the past trend and latest data for future calculations. Further, the values may differ for AC and Non-AC buses; and therefore, the calculations can be done considering a specific bus segment to improvise the model efficiency. The different state government policies and different working style of the STUs can also influence the values of these input cost parameters. Therefore, the benchmarking KMs can vary for different STUs considering the similar scenarios. Thus, the data of other STUs can be compared to compare and generalize the findings of the study. Further, additional scenarios can also be framed to incorporate the changes in other input parameters such as cost of the bus, percentage of subsidy, different tax structures, etc., to explore their influence on the overall cost of procurement and operations for both the models. Considering these possibilities, the model efficiency can be enhanced for better decision support.

References

1. Chan CC (2013) The rise & fall of electric vehicles in 1828–1930: lessons learned [scanning our past]. Proc IEEE 101(1):206–212. https://doi.org/10.1109/JPROC.2012.2228370
2. Saxena S, Gopal A, Phadke A (2014) Electrical consumption of two, three- and four-wheel light-duty electric vehicles in India. Appl Energy (Elsevier) 115:582–590. https://doi.org/10.1016/j.apenergy.2013.10.043
3. Strickland D, Chittock L, Stone DA, Foster MP, Price B (2014) Estimation of transportation battery second life for use in electricity grid systems. IEEE Trans Sustain Energy 5(3):795–803. https://doi.org/10.1109/TSTE.2014.2303572
4. Bloomberg NEF (2019) A behind the scenes take on lithium-ion battery prices. https://about.bnef.com/blog/behind-scenes-take-lithium-ion-battery-prices/. Last accessed 01 May 2019

Effect of Traffic Flow Characteristics on Saturation Flow at Signalized Intersections of Ahmedabad City

Bhavik Shah, Pinakin Patel, L. B. Zala, and Dhaval Parmar

1 Introduction

Intersection is an area where traffic from different road streams meet, which after going through conflicting movements like merging, diverging and crossing choose the appropriate route and leave the conflict zone. Due to their influence on each other, disturbance of pedestrians and bicycle to vehicles, and the loss of green time for beginning and clearance and so on, the capacity of intersections is very less than that of their approach channels. Thus, the intersections typically are the bottleneck of the network, the prominent and instantaneous source of the traffic accidents and traffic jam.

In recent years, the population in cities, vehicle ownership and traffic volume in links have increased dramatically because of the continuous high-speed growth of the economy, which causes traffic congestion of various levels in most of the cities. Due to heavy congestion, the velocity of the automobile/vehicle decreases easily.

The urban traffic of India is heterogeneous in nature. It consists of fast moving vehicles as well as slow moving vehicles. Because of the later type of vehicles, the capacity of the road is affected. The conflict, confusion, and irritation caused by this heterogeneous traffic result in a large number of accidents.

B. Shah (✉) · P. Patel · L. B. Zala · D. Parmar
Civil Engineering Department, Birla Vishvakarma Mahavidhyalaya, Mota Bazaar, Vallabh Vidyanagar, Anand, Gujarat 388120, India

P. Patel
e-mail: pinakin.patel@bvmengineering.ac.in

L. B. Zala
e-mail: lbzala@bvmengineering.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_8

The objective of the study is.

- To study the traffic flow characteristics at the signalized junction.
- To derive base Saturation Flow values for mixed traffic conditions.
- To analyze the effect of mixed traffic flow on Saturation Flow.

Ahmedabad is one of the developing cities of India. The traffic here is highly heterogeneous and requires congestion solving. 5 important junctions of Ahmedabad; which are Incometax Cross Road, Vijay Cross Road, Iim Cross Road, Panjrapole Cross Road and Pakwan Cross Road (2 Approaches); have been selected for the research. The above junctions are selected considering the variation in width, traffic composition and geometry. Therefore Ahmedabad is selected as the study area of this research.

The flow is measured in terms of Saturation Flow. Saturation Flow is defined as the rate at which vehicles that have been waiting in a queue during the red interval cross the stop line of a signalized intersection approach lane during the green interval. Saturation Flow is usually expressed in passenger car units per hour of green (PCU/h). In developed countries, PCU is constant but in developing countries like India where the traffic is highly heterogeneous, PCU value doesn't remain static. The dynamic PCU values are used to develop Saturation Flow models but they have certain limitations. The present study deals with developing a base saturation model and comparing it with INDO HCM and US HCM 2010.

2 Literature Review

Saturation Flow is defined as the rate at which vehicles that have been waiting in a queue during the red interval cross the stop line of a signalized intersection approach lane during the green interval. Saturation Flow is usually expressed in passenger car units per hour of green (PCU/h). A vehicle is considered "discharged" when its front axle passes the stop line. Saturation Flow can be measured directly in the field.

Nguyen [1] has conducted his research in a motorcycle dependent city; i.e. HO CHI MINH where there is a unique traffic situation due to MDCs. The study conducted by him introduced the particular traffic pattern in two-wheeler dependent cities where the motorcycle rate is excessively high. The field observation depicted that whenever the Saturation Flow rate decreases there is capacity reduction; after a period with green time. Chand et al. [2] have made the efforts to analyze the Saturation Flow at intersections by generating the dynamic PCU for the selected signalized intersections having heterogeneous traffic condition in the Indian cities (Noida, Delhi). They proposed a methodology to calculate the value of PCU and Saturation Flow conforming for the enumerated traffic conditions. Agarwala and Lammel [3] studied the commonly existing seepage behaviour of smaller vehicles, in order to develop and create a model to simulate the mixed traffic condition close to reality. As motorbikes/bikes have easier manoeuverability, they can pass through the gaps between the stagnant or non-stagnant vehicles. Ghasemlou et al. [4] researched the method of deriving heavy vehicle adjustment factor for the through vehicles at a signalized intersection. Heavy vehicles, having large size along with their slow acceleration, are responsible for a various negative effect on other category of vehicles.

Saha et al. [5] collected data from fifteen isolated signalized intersections situated in five cities of India, namely, Delhi, Chandigarh, Patiala, Panch Kula and Mumbai. This study proposed an improved method for computing delay at signalized intersections. Queue length was directly measured from the field, and based on the length of the queue of the cycle, the delay was calculated using Simpson's one-third rule. The conventional methods (HCM and Webster's) either overestimated or underestimated the delay. This is attributed primarily to mixed traffic, poor lane discipline and different driving pattern and behaviour in India, than observed in developed nations.

From the Literature review, it is quite evident that the traffic composition of various areas effect the general Saturation Flow model given by INDO HCM 2017. The base Saturation Flow value won't remain the same for different areas in India due to highly heterogeneous traffic conditions. Therefore, a new and a more accurate model is required to predict the exact value of the base Saturation Flow.

3 Methodology

The methodology for developing a model of Saturation Flow is expressed here. The first step is observing the traffic flow parameters, observation of traffic flow parameters, intersections geometric details, derivation of PCUs, development of discharge rate models, validation of the model and derivation of Saturation Flow values. The steps are as follows.

3.1 Pilot Survey

Pilot Survey was carried out in Ahmedabad City in order to find out the junctions which will suit for the study. The criterion that has been considered for the selection of the intersection from the literature review are as follows.

4-Legged Signalized Intersection, Free Left Turning lane, Appropriate Approach Width, Vehicle Composition Variation, Flat Gradient, Nearby Buildings for Videography and Right Turning Movement.

The various intersections of Ahmedabad city which were studied in the pilot survey are as follows: Vijay Cross Roads, Swastik Cross Road, Gurukul Cross Road, Mansi Cross Road, IIM Cross Road, Income Tax Cross Road, Panjrapole Cross Road, Judges Cross Road, Pakwan Cross Road, 132-Ring Road, Ashram Cross Road, Shyamal Cross Road, Shastri Nagar Cross Road, Pragati Nagar Cross Road and Zydus Cross Road. Out of the above, the following intersections are selected for videography based on the criteria's;

- Pakwan Cross Road–2 legs (14 m wide)
- Vijay Cross Road (9.5 m wide)
- IIM Cross Road (12 m wide)
- Income Tax Cross Road (10.5 m wide)
- Panjrapole Cross Road (10.5 m wide)

3.2 Data Collection

The data of the selected junctions has been collected using videography. The selected junctions are Incometax Cross Road, Vijay Cross Road, IIM Cross Road, Panjrapole Cross Road and Pakwan Cross Road (2 Approaches). Camera with stand was installed at the tall buildings nearby and the data was recorded.

The images of the Videography at various four-legged signalized intersections conducted in Ahmedabad is in Figs. 1, 2, 3, 4, and 5.

Fig. 1 Vijay cross road

Fig. 2 IIM cross road

Fig. 3 Income tax cross road

Fig. 4 Pakwan cross road

Fig. 5 Panjrapole cross road

3.3 Data Extraction

The Data is extracted manually using Avidemux (version 2.6) Software. Avidemux is a freeware software that plays the video in microseconds. This slow motion helps us in extracting a number of different types of vehicles easily and without any software error.

The screen is marked using Screen Marker (version 1.0.0.1) software. The Data entry was directly inputted in MS Excel. The Classified Volume Count of each Intersection was carried out in the software. The different types of vehicle were counted for 15 Green Time Cycles at an interval of 5 s for each intersection. The vehicles were also classified on the basis of their manoeuvering; i.e. straight going or right turning. The Data Extracted from the Videography was directly entered in Microsoft MS Excel. The Dynamic PCU values are calculated by using time occupancy method given by Satish Chandra and Kumar [7]. The obtained dynamic values for 2W, 3W, CS, CB, LCV, BUS and HCV are 0.2, 0.71, 1, 1.48, 1.81, 5.6 and 4.8 respectively.

4 Data Analysis

4.1 Traffic Composition

The analysis of data has been carried out in MS Excel. The first and foremost step is to identify the composition of traffic. The composition was identified and the best way to display it was using Pie Charts. The various graphs of the composition of traffic at various junctions are shown in Figs. 6, 7, 8, 9, 10, and 11.

The above graphs show that the highest composition is of two-wheelers compared to other category of vehicles in all the junctions followed by big car, small car and three-wheelers. The composition of HCV and Bus is less compared to other types of vehicles. This variation shows that traffic is highly heterogeneous.

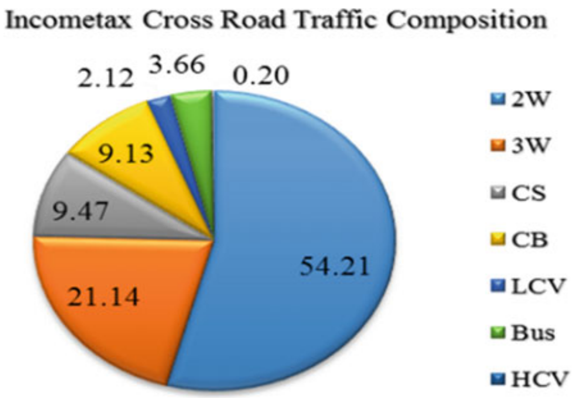


Fig. 6 Average traffic composition at income tax cross road

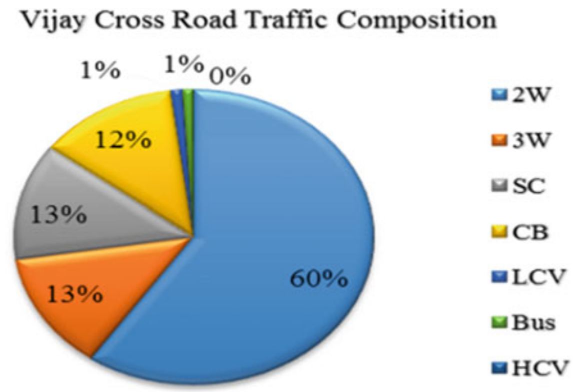


Fig. 7 Average traffic composition at Vijay cross road

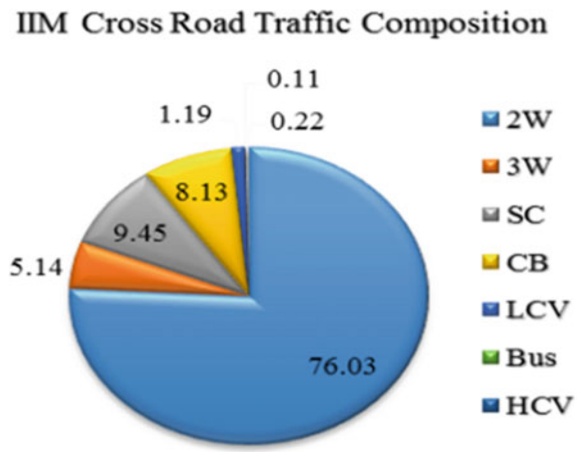


Fig. 8 Average traffic composition at IIM cross road

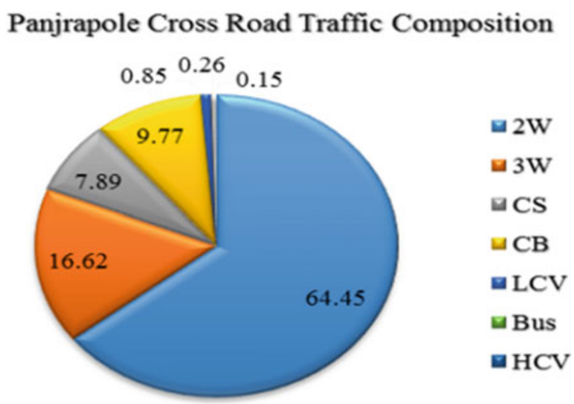


Fig. 9 Average traffic composition at Panjrapole cross road

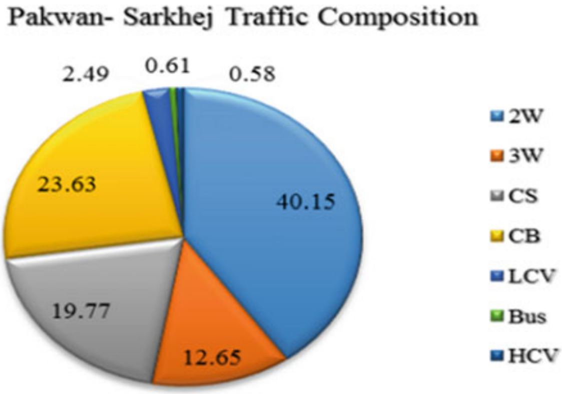


Fig. 10 Average traffic composition at Pakwan-Sarkhej cross road

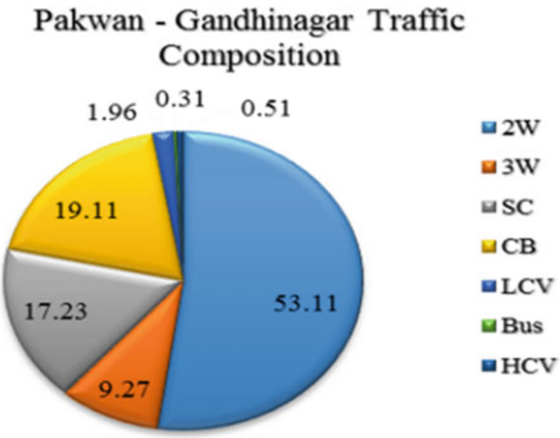


Fig. 11 Average traffic composition at Pakwan-Gandhinagar cross road

4.2 *Determination of Saturated Green Time*

The number of vehicles moving during the Saturated Green time shows the effectiveness of any junction. It helps in determining the capacity of the junction and states whether the junction is operating efficiently or not. IRC states that the initial 10 s and the last 5 s are generally neglected and the remaining time is taken as saturated green time. But it is not valid in most of the cases due to the heterogeneous nature of our Indian traffic. Therefore, in order to find the saturated the green time and justify the same Analysis of Variance (ANOVA) is used. The results of the ANOVA are shown in Table 1.

The saturated green time is accepted because of the F value obtained from the ANOVA test. If the F value is less than F_{critical} than the value is accepted. Thus the Saturated Green time obtained in the table is valid and can be used for further analysis.

Table 1 Summary of saturated green time of all approaches

Sr. No.	Intersection	Study approach	Total Green Time in seconds	Saturated Green Time in seconds	*P* value	*F*	F_{Critical}	$F < F_{\text{critical}}$
1	Income tax cross road	A-01	0–65	5–45	0.849	0.477	2.092	Yes
2	Vijay cross road	A-02	0–45	10–30	0.129	1.965	2.769	Yes
3	IIM cross road	A-03	0–65	5–50	0.105	1.697	2.012	Yes
4	Panjrapore cross road	A-04	0–40	5–30	0.081	2.660	3.219	Yes
5	Pakwan-Sarkhej	A-05	0–65	10–60	0.134	1.555	1.947	Yes
6	Pakwan-Gandhinagar	A-06	0–55	10–55	0.097	1.730	2.012	Yes

4.3 Variation of Composition with Respect to PCU

To understand the characteristics and trend of the Saturation Flow at the intersections, different charts showing the variation of percentage composition with respect to Saturation Flow (PCU per 5 s) are plotted. The data of the saturated green time is taken for analysis. The variation with respect to PCU per 5 s is depicted in Figs. 12, 13, 14, 15, 16, 17, and 18.

The Two-Wheeler graph shows that the slope is negative which means that there is a negative correlation of two-wheelers with the Saturation Flow. As the number of two-wheeler increases, the Saturation Flow in terms of PCU decreases. The reason is that; as the two-wheelers are small in size and higher in number in the traffic stream, they can penetrate in the space between other vehicles due to their high manoeuverability. This reduces their PCU Value in dynamic analysis of PCU Values

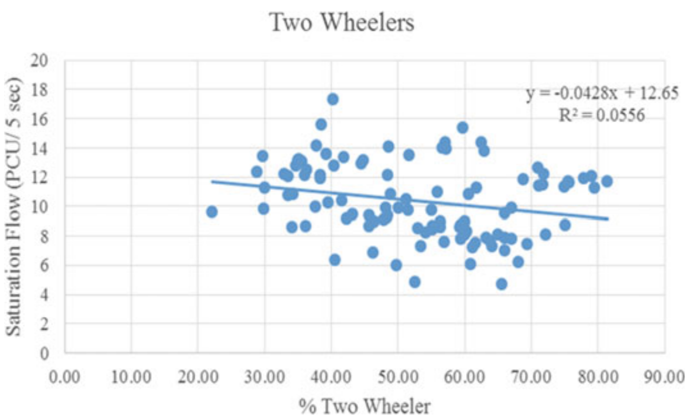


Fig. 12 Variation of Saturation Flow with 2W

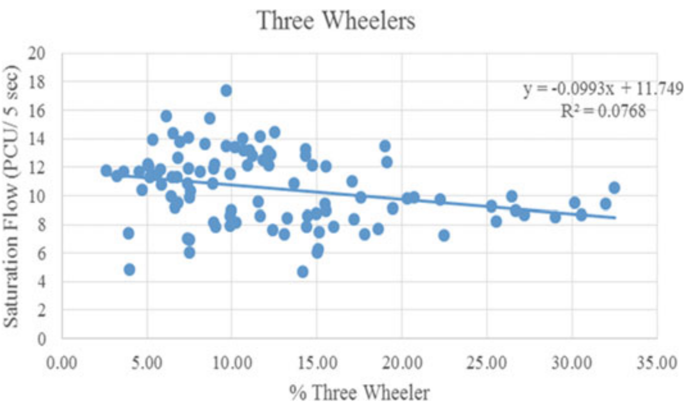


Fig. 13 Variation of Saturation Flow with 3W

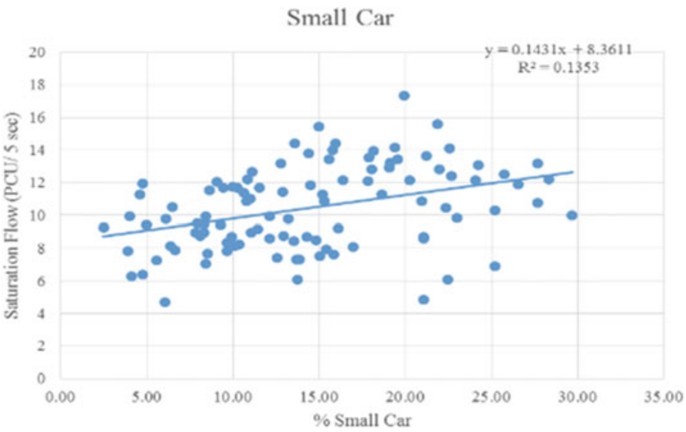


Fig. 14 Variation of Saturation Flow with CS

(PCU value in static analysis is 0.2 which is also very less). Thus as the PCU value will be less, the flow will also be less.

The Three-Wheeler graph shows that the slope is negative which means that there is a negative correlation of three-wheelers with Saturation Flow. As the number of three-wheeler will increase, there will be a slight decrease in the Saturation Flow in terms of PCU. The reason is that; PCU value of three-wheeler doesn't vary much from the standard value which means it can increase or decrease depending upon the junction. Thus, it can follow both the trends and is generally flat. Here the trend is negative.

The Small Car graph shows that the slope is positive which means that there is a positive correlation of small cars with Saturation Flow. As the number of the small cars will increase, there will be a slight increase in the Saturation Flow in terms of

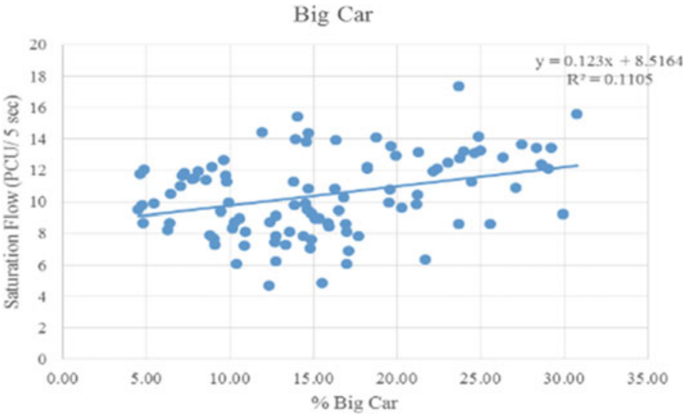


Fig. 15 Variation of Saturation Flow with CB

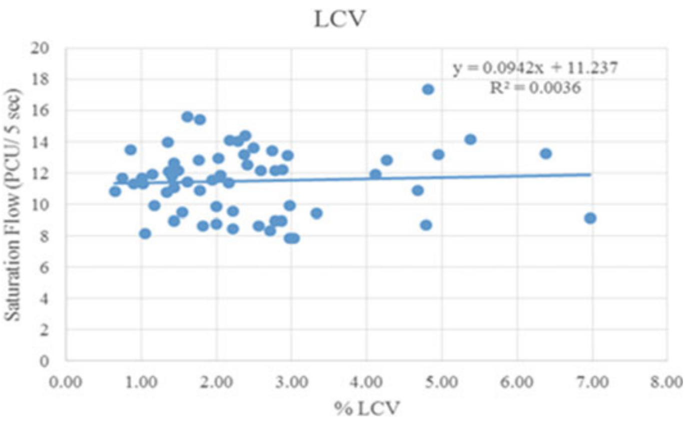


Fig. 16 Variation of Saturation Flow with LCV

PCU. The reason is that; PCU value of the small car is the standard value which means it shows the transition of PCU values. It can also follow both the trends depending upon the junction and is generally flat. Here the trend is positive.

The Big Car graph shows that the slope is positive which means that there is a positive correlation of big cars with Saturation Flow. As the number of big cars will increase, there will be an increase in the Saturation Flow in terms of PCU. The reason is that; PCU value of the big car is higher than that of the standard value, thus the increase in the number of big car will increase the Saturation Flow. Also with the increase in the number of big cars, its PCU value also increases (Dynamic PCU).

The Light Commercial Vehicle graph shows that the slope is positive which means that there is a positive correlation of LCV with Saturation Flow. As the number of LCV will increase, there will be an increase in the Saturation Flow in terms of PCU.

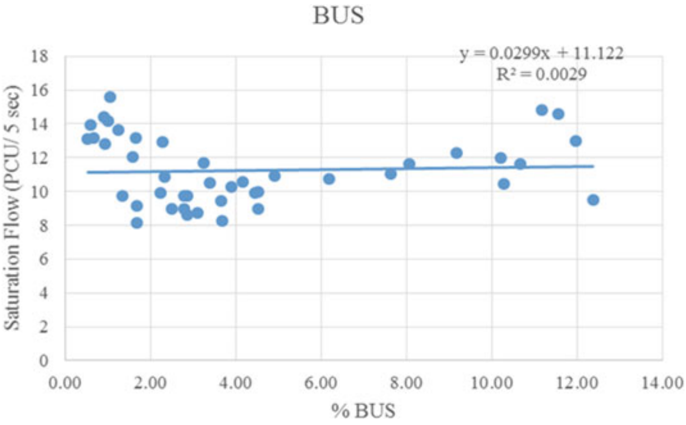


Fig. 17 Variation of Saturation Flow with BUS

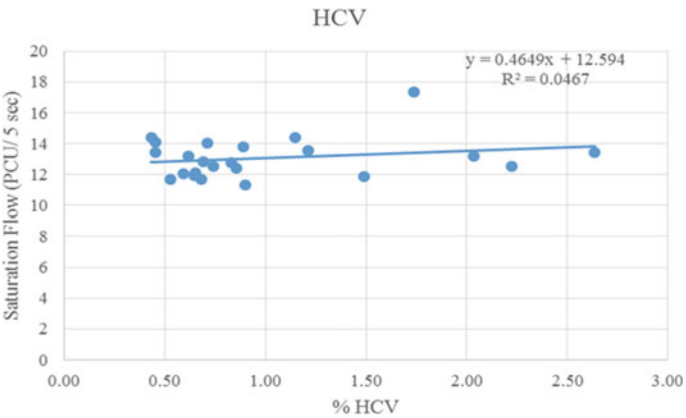


Fig. 18 Variation of Saturation Flow with HCV

The reason is that; PCU value of LCV is higher than that of the standard value, thus the increase in the number of LCV will increase the Saturation Flow.

The BUS graph shows that the slope is positive which means that there is a positive correlation of BUS with Saturation Flow. As the number of BUS will increase, there will be an increase in the Saturation Flow in terms of PCU. The reason is that; PCU value of BUS is higher than that of the standard value, thus the increase in the number of BUS will increase the Saturation Flow.

The Heavy Commercial Vehicle graph shows that the slope is positive which means that there is a positive correlation of HCV with Saturation Flow. As the number of HCV will increase, there will be an increase in the Saturation Flow in terms of PCU. The reason is that; PCU value of HCV is higher than that of the standard value, thus the increase in the number of HCV will increase the Saturation Flow.

From all the graphs, it can be concluded that the trend of two-wheelers is always positive in terms of VPH while it is always negative for the Saturation Flow in PCU. Three-wheeler and small car follow mix trend in both VPH and PCU. Their trend is positive and negative depending upon the junction due to less variation from the standard value. Moving on to the case of Big Car, LCV, Bus and HCV; their trend is always negative in terms of VPH while it is always positive in terms of PCU. It states that the increase in their proportion will increase the value of Saturation Flow in terms of PCU.

5 Saturation Flow Model Development

The regression analysis has been used to determine the equation for base Saturation Flow. Saturation Flow models are a systematic representation of the complex real-world traffic problems. Models are powerful tools for assessing the impact of traffic on real-world problems. They depict the on-field scenario as good as possible and forecast the future value accurately. Four Models are developed using regression analysis to find the equation for Saturation Flow.

Developed Model

This model is developed by considering Saturation Flow (PCU/h/m) as dependent variable. The dependent variable is the proportions of 2W, 3W, CS, CB, LCV, Bus, HCV and width of the approach. The model is as follows:

$$\begin{aligned}\text{Saturation Flow, } S(\text{PCU/h/m}) = {} & 3.898P_{2\text{w}} + 3.663P_{3\text{w}} + 3.79P_{\text{cs}} + 5.143P_{\text{cb}} \\ & + 12.281P_{\text{lcv}} + 16.513P_{\text{bus}} + 22.108P_{\text{hcv}} \\ & - 1.507P_{\text{rt}} + 19.639W\left(R^2 = 0.95\right) \end{aligned} \tag{1}$$

where

$P_{2\text{w}}$	Proportion of Two Wheelers,
$P_{3\text{w}}$	Proportion of Three-Wheelers,
P_{cs}	Proportion of Small Cars,
P_{cb}	Proportion of Big Cars,
P_{lcv}	Proportion of Light Commercial Vehicles,
P_{bus}	Proportion of Bus,
P_{hcv}	Proportion of Heavy Commercial Vehicles,
P_{rt}	Proportion of right turning vehicles,
W	Width (m).

Table 2 Regression statistics of developed model

	Coefficients	*t* Stat	*P*-value
%TW	3.897	8.050	3.73E−15
%3W	3.662	5.453	6.93E−08
%Small car	3.789	5.029	6.32E−07
%Big car	5.142	6.847	1.69E−11
%LCV	12.281	7.260	1.07E−12
%Bus	16.513	14.356	5.7E−41
%HCV	22.108	6.657	5.79E−11

The *t*-value of the model should be more than ±1.96 and *p*-values should be as small as possible. The values in the table follow the above and make the model a good model. The statistics of the developed model is as follows (Table 2).

6 Sensitivity Analysis

Sensitivity analysis is the method used to examine how the independent variable values will impact a particular dependent variable under the fixed/given circumstances. It is also known as what-if analysis. It helps to determine how sensitive the output will be when the input is varied. Sensitivity Analysis is carried out for the developed model. The dependent variable is Saturation Flow in terms of PCU/h/m while the independent variables are the proportions of TW, 3W, CS, CB, LCV, BUS, HCV, WIDTH (in m) and RIGHT TURN. Dual Axis Graphs have been developed for Sensitivity Analysis. The Graphs of Sensitivity Analysis are shown below.

Figure 19 shows that there is a linear relationship between the Saturation Flow (PCU/h/m) and the proportion of 2W and 3W. Both of them have a positive slope

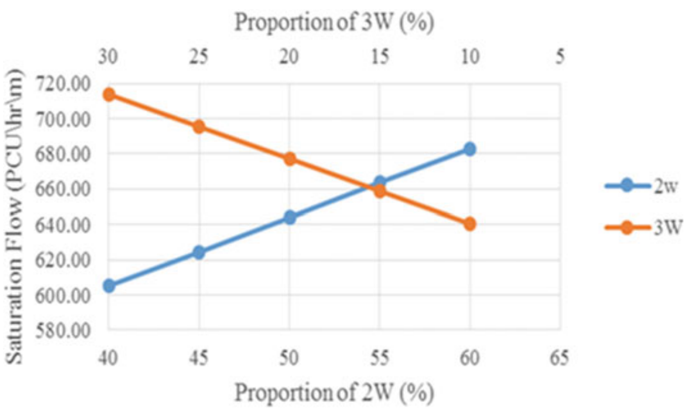


Fig. 19 Impact of %2W and %3W

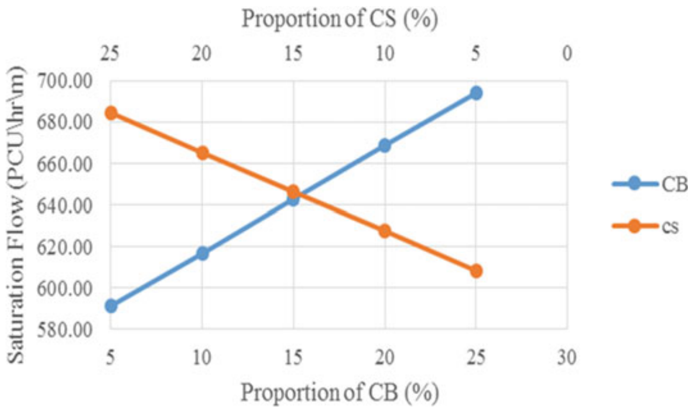


Fig. 20 Impact of %CS and %CB

which shows that they have a positive effect on the Saturation Flow. Figure 20 shows that there is a linear relationship between the Saturation Flow (PCU/h/m) and the proportion of CS and CB. Both of them have a positive slope which shows that they have a positive effect on the Saturation Flow.

Figure 21 shows that there is a linear relationship between the Saturation Flow (PCU/h/m) and the proportion of TW and LCV. Both of them have a positive slope which shows that they have a positive effect on the Saturation Flow. Figure 22 shows that there is a linear relationship between the Saturation Flow (PCU/h/m) and the proportion of Bus and HCV. Both of them have a positive slope which shows that they have a positive effect on the Saturation Flow. Figure 23 shows that there is a linear relationship between the Saturation Flow (PCU/h/m) and the proportion of RT

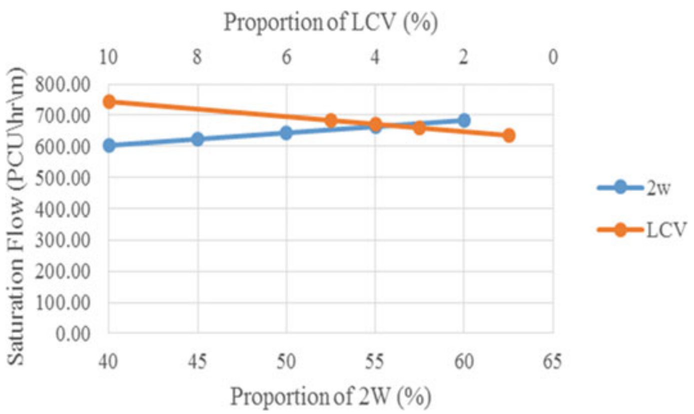


Fig. 21 Impact of %TW and %LCV

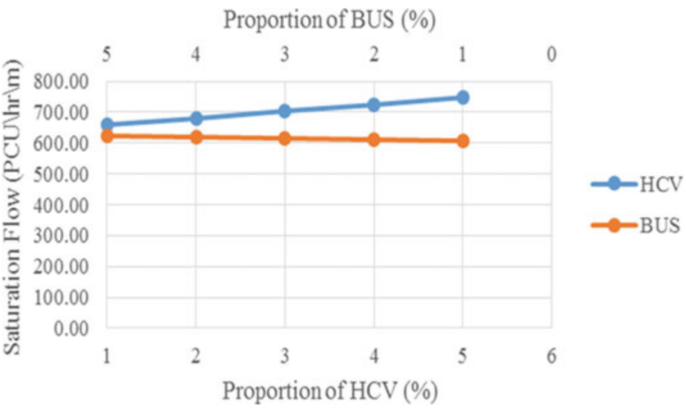


Fig. 22 Impact of %HCV and %BUS

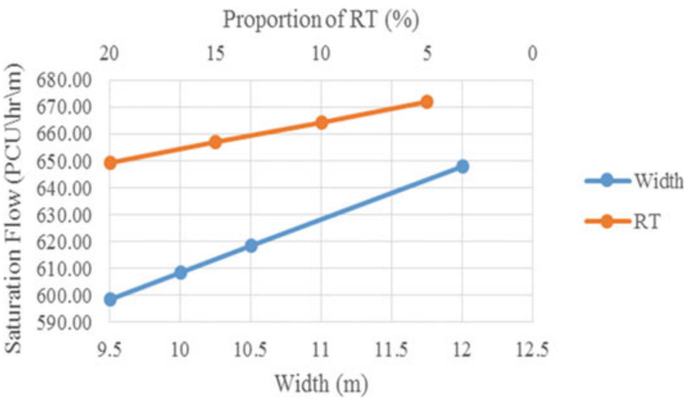


Fig. 23 Impact of %RT and width (m)

and width (m). Here the slope of Width is positive which shows that it has a positive effect on the Saturation Flow. While the slope of RT is negative which shows that it has a negative effect on the Saturation Flow.

From all the above graphs, it can be observed that the slope of CS, CB and RT have higher slope compared to other variables. It shows that they affect the model significantly. Minor changes in those variables will cause major changes to Saturation Flow.

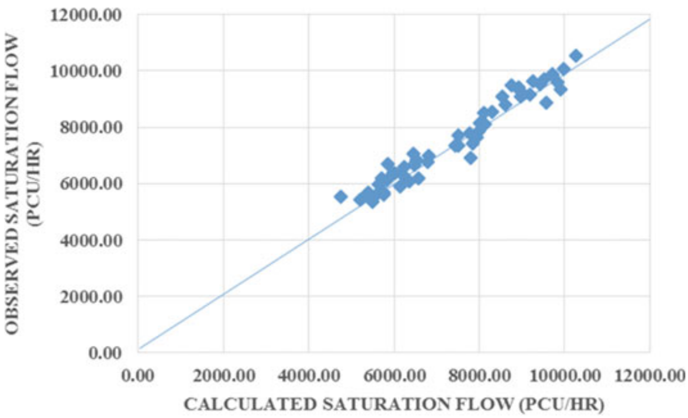


Fig. 24 Results of validation of Saturation Flow model

7 Model Validation

The Saturation Flow Model is validated using the flow measured from the 2 intersections of Surat City (RTO Cross Road and Athwa Cross Road). The data of the vehicles crossing the green line during the saturated green time was collected. Saturation Flow collected was calculated and compared with the Saturation Flow predicted from the model. The results are shown in Fig. 24. The plot of Observed Saturation Flow versus Calculated Saturation Flow indicates much closeness to the diagonal line. This indicates that the values estimated by the model are fairly accurate and hence can be used for the prediction of Saturation Flow.

8 Modification of INDO HCM

INDO HCM has given the following equation for the estimation of Base Saturation Flow.

$$\mathrm{USF}_0 = \begin{cases} 630; & \text{for } w < 7.0m \\ 1140 - 60w; & \text{for } 7.0 \le w \le 10.5m \\ 500; & \text{for } w > 10.5m \end{cases} \tag{2}$$

$$\mathrm{SF} = \mathrm{W} \times \mathrm{USF}_0 \times f_{\mathrm{bb}} \times f_{\mathrm{br}} \times f_{\mathrm{is}} \tag{3}$$

The above equation states the unit base Saturation Flow value should be taken as 500 for width greater than 10.5 m but that value doesn't hold true for heterogeneous traffic condition prevailing in India. The graphs shown below show the variation in field Saturation Flow and the Saturation Flow given by INDO HCM (Figs. 25, 26, and 27).

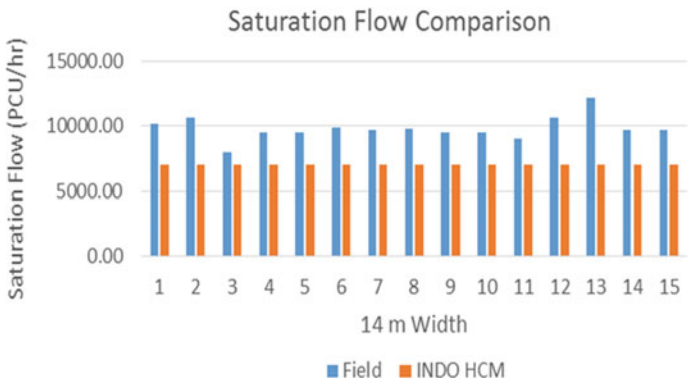


Fig. 25 Difference in Saturation Flow at Pakwan Gandhinagar approach

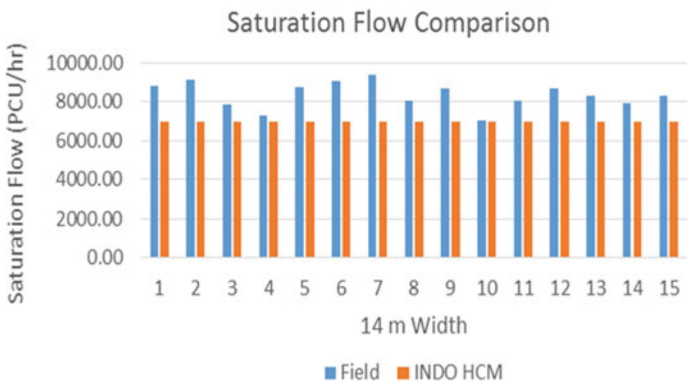


Fig. 26 Difference in Saturation Flow at Pakwan Sarkhej approach

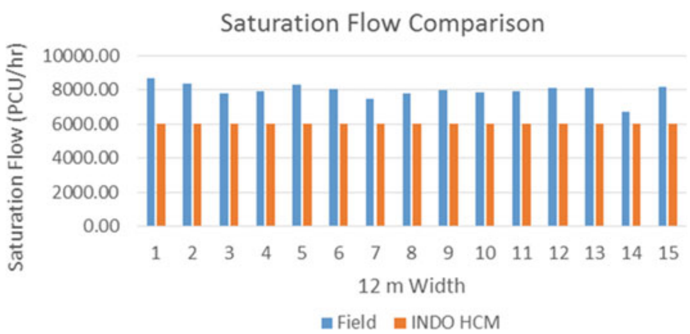


Fig. 27 Difference in Saturation Flow at IIM cross road

From the graphs, it is observed that the field Saturation Flow varies more than 43% from the base value given by INDO HCM for the width greater than 10.5 m. Therefore a modification is required in the current model so that it can predict the Saturation Flow value more accurately based on the conditions. The data of junctions having a width greater than 10.5 m is used for the development of the model. The developed model is shown below.

$$\begin{aligned} \text{Saturation Flow, } S(\text{PCU/h/m}) = {} & 500 + 4.790 P_{2\text{w}} + 5.325 P_{3\text{w}} + 4.185 P_{\text{cs}} \\ & + 7.071 P_{\text{cb}} + 12.740 P_{\text{lcv}} + 25.426 P_{\text{bus}} + 26.630 P_{\text{hcv}} \\ & - 2.452 P_{\text{rt}} - 24.020 W \left(R^2 = 0.85 \right) \end{aligned} \tag{4}$$

8.1 Validation

The Saturation Flow Model and the modification of INDO HCM model are validated using the flow measured from other intersections (RTO, Surat). The data of the vehicles crossing the green line during the saturated green time was collected. Saturation Flow collected was calculated and compared with the Saturation Flow predicted from the model. The results are shown in Fig. 28. The plot of Observed Saturation Flow versus Calculated Saturation Flow indicates much closeness to the diagonal line. This indicates that the values estimated by the model are fairly accurate and hence can be used for the prediction of Saturation Flow. The Mean absolute percentage error (MAPE) of the developed model is 6%. Thus, above model fairly predicts the Saturation Flow value for junctions having width greater 10.5 m for the heterogeneous conditions prevailing in India. It gives an upgrade to INDO HCM 2017.

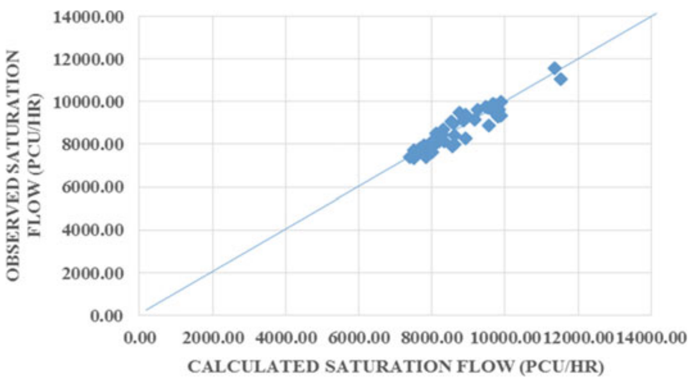


Fig. 28 Results of validation of modification of INDO HCM model

The t-statistics was carried at alpha equal to 0.05. The t-value for the model at 95% Confidence level is 0.16 with 47 degree of freedom and the value of t-critical (2-tail) is 2.011 for modification of INDO HCM 2017 Model. Here, the t-value obtained is less than t-critical. Hence the model predicts the value near to the field value.

9 Conclusion

Signalization is a traffic control strategy to ease the competition by providing a right of way in a cyclic manner to conflicting traffic at intersections. Saturation Flow is useful to determine the capacity of the intersection, signal design, etc. Many researchers have made an attempt to produce a model for mixed traffic conditions. For doing so, traffic flow data of selected signalized intersections of Ahmedabad and Delhi is collected using Videography during the peak hours. The data was collected from the nearby high-rise buildings. Based on the collected video, the data was extracted using Avidemux Software and the extracted data was directly inputted into Microsoft Excel. The overall traffic composition of TW, 3W, CS, CB, LCV, BUS and HCV is 51%, 12%, 15.41%, 16%, 2%, 1.5% and 0.5% respectively. It indicates that the proportion of two-wheelers is maximum and leads to high heterogeneous traffic conditions.

IRC suggests that there should be an initial loss of 10 s to calculate the saturated green time. But, it holds only for homogenous conditions, not for highly heterogeneous conditions. Therefore, Analysis of Variance is used to determine the saturated green time of all the selected junctions. The analysis of various composition with Saturation Flow shows the trend of various compositions. From all the graphs, it can be concluded that the trend of two-wheelers is always positive in terms of VPH while it is always negative for the Saturation Flow in PCU. Three-wheeler and small car follow mix trend in both VPH and PCU. Their trend is positive and negative depending upon the junction due to less variation from the standard value. Moving on to the case of Big Car, LCV, Bus and HCV; their trend is always negative in terms of VPH while it is always positive in terms of PCU. It states that the increase in their proportion will increase the value of Saturation Flow in terms of PCU.

Saturation Flow is an important factor in the signal design and is dependent on roadway and traffic conditions, which can vary substantially from one place to another. Saturation Flow models are developed using regression analysis considering various variables. Four models were developed and on the basis of t-statistics model 2 of PCU/h/m is selected. Sensitivity Analysis shows that the percentage of small car, the percentage of big car and percentage right turn have a major impact on the Saturation Flow model. The predicted values are very near to the observed values which showed that the model has fair accuracy and can be used to predict Saturation Flow. It also states that the right turning movement has a negative impact on the Saturation Flow.

A modification factor for the INDO HCM has been developed for width greater than 10.5 m. The model fairly predicts the Saturation Flow for width higher than 10.5 m. The validation of the model was carried out by comparing the field values and predicted values. The predicted values are very near to the observed values which showed that the model has fair accuracy and can be used to predict Saturation Flow. The t-statistics also supported the same. It also states that the right turning movement has a negative impact on the Saturation Flow.

The study carried out is for Ahmedabad city and the models are validated on intersections of Surat city only. The developed model can be used for predicting Saturation Flow for different intersections with similar characteristics in India.

Acknowledgements The authors acknowledge the opportunity to submit the research work at the 5th Conference of the Transportation Research Group of India held at Bhopal (India) from 18 to 21 December 2019, that forms the basis of this article.

References

1. Nguyen HD (2016) Saturation Flow rate analysis at signalized intersections for mixed traffic conditions in motorcycle dependent cities. In: International symposium on enhancing highway, vol 15. Transportation Research Procedia, pp 694–708
2. Chand S, Gupta NJ, Velmurugan S (2017) Development of Saturation Flow model at signalized intersection for heterogeneous traffic. In: World conference on transport research—WCTR 2016, vol 25. Transportation Research Procedia, Shanghai, pp 1662–1671
3. Agarwala A, Lammel G (2015) Seepage of smaller vehicles under heterogeneous traffic conditions. ABMTRANS 1–6
4. Ghasemlou K, Topal A, Tanyel S, Aydin MM, Kazemi Afshar AA (2014) Investigation the effect of heavy vehicles on capacity of signalized intersections based on Bayes' theorem. Transp Dev Inst ASCE 376–387
5. Saha A, Chandra S, Ghosh I (2017) Delay at signalized intersections under mixed traffic conditions. J Transp Eng Part A Syst ASCE 143–150
6. Anusha CS, Verma A, Kavitha G (2013) Effects of two-wheelers on Saturation Flow at signalized intersections in developing countries. J Transp Eng 139(5):448–457 (ASCE)
7. Chandra S, Kumar U (2003) Effect of lane width on capacity under mixed traffic conditions in India. J Transp Eng 129(2):155–160

Simulation-Based Optimization for Heterogeneous Traffic Control

Himabindu Maripini, Lelitha Devi Vanajakshi, and Bhargava Rama Chilukuri

1 Introduction

Deteriorating Level of Service (LoS) at Indian urban roads as a consequence of increasing delays at intersections is a challenge to traffic engineers. This could be because of sub-optimal signal settings at signalized intersections, which are insensitive to the high traffic fluctuations. Traditional traffic signal design methodologies such as Webster's method or HCM method are originally developed for homogenous and lane disciplined traffic conditions. On the contrary, Indian traffic is highly heterogeneous in composition with more than ten categories of vehicle classes, ranging from two-wheelers to tandem tridem vehicles. All these vehicle classes differ in their physical and operational characteristics. Such a diversity in vehicular features leads to varied behavioral maneuvers of road users, thereby contradicting the basic assumptions of most of the exiting microscopic and macroscopic traffic flow models, some of which are listed below.

(i) Since two-wheelers are relatively smaller in size with high operational capability, they intrude longitudinally and laterally into the space available within the traffic stream and occupy them. This disregards the lane disciplined behavior of traffic stream based on which all the microscopic models (such as car following and gap acceptance models) are designed.
(ii) Small-sized vehicles such as two-wheelers and auto-rickshaws reach the stop line taking advantage of their size, irrespective of their time of arrival. This

H. Maripini (✉) · L. D. Vanajakshi · B. R. Chilukuri
Department of Civil Engineering, Indian Institute of Technology Madras, Chennai, India

L. D. Vanajakshi
e-mail: lelitha@iitm.ac.in

B. R. Chilukuri
e-mail: bhargava@iitm.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_9

explicitly violates the basic First In First Out (FIFO) assumption of queue discharge process.

(iii) The queuing pattern formed by occupying all the available space near the stop line for an early discharge results in a diamond-shaped pattern. This implies that vehicles near the stop line do not necessarily experience the same amount of delay. Delay experienced would rather be computed from the vehicle's arrival and departure times than based on cumulative arrival and departure curves.

(iv) Moreover, the type of vehicle class being followed greatly affects the travel time consumed by the follower vehicle, especially when the leader is a heavy tandem truck and follower being a two-wheeler or auto-rickshaw.

(v) Saturation flow of intersection approaches is considered to be a critical parameter in signal design. However, it is observed that, in heterogeneous conditions saturation flow is dynamic and changes with respect to traffic composition.

Traditional signal design methods also are violated due to some of the above characteristics. Moreover, the traffic flow variables considered in these design procedures, such as arrival rate, saturation flow, and critical lane volume, do not represent the diverse nature of Indian traffic and may not be suitable for signal timing design. Thus, these typical Indian traffic characteristics signify the need for a novel traffic control and management strategy that can effectively handle the diverse Indian traffic in real time. Therefore, an optimal control design as a function of different set of realistic and scalable variables of the present traffic is necessitated.

An optimal signal control strategy involves objective of either minimizing performance measures such as queue length, travel time, delay, number of stops or maximizing throughput, average speed, throughput, or a weighted combination of any of them, with the decision variables being green splits, cycle lengths, and offsets. The choice of the objective function is dependent on the extent to which it represents the traffic and the ease of data collection process involved. Out of the available traffic flow variables, spatial performance measures are relatively easy to obtain and require a sample data. Among the spatial measures, travel time and delay are perceived by the users and operators alike and are obtained from probe data. With the advancement in technology and use of smart phones, Wi-Fi and BT data from representative sample of vehicles sufficiently represent the performance of the traffic stream. Hence, the present study considers minimization of total travel time experienced by all the vehicles in the network over a given period of time as its objective and obtains the signal control plan (with green splits and cycle length as decision variables) that yields the least possible value of this objective.

To formulate the general optimization problem, the objective function has to be expressed in terms of signal timing parameters. In general, travel time can be determined from cumulative arrival and departure curves with respect to signal state of the subject intersection. However, Indian traffic violates FIFO assumption, and an accurate estimate of travel time cannot be obtained. Since objective function lacks a functional form in terms of decision variables, black-box optimization can be relied on to perform optimization. In order to incorporate the traffic dynamics into the design, microscopic traffic simulation software is used. Such an optimization is termed as "simulation-based optimization." Various data sets of decision variables are fed as input to the simulator to run iteratively and compare the objective function values obtained as output from simulator. Therefore, this paper attempts an optimal signal control strategy with travel time minimization as objective and performs simulation-based optimization with VISSIM as microscopic traffic simulator. The performance yielded based on the obtained optimal signal settings is compared with that of conventional signal control methodologies.

The reminder of this paper is structured as follows. Section 2 presents a review of the literature regarding the optimal signal control strategies, simulation-based optimization (SO) methods in general, and an overview of the implementation of SO methods in the design of traffic signal control. Section 3 discusses the details of study area and the simulation. Section 4 presents the proposed formulation and optimal signal plan using various algorithmic approaches. The paper concludes by presenting results in comparison of the proposed optimal solution with that of Webster and HCM design methodologies in Sect. 5.

2 Review of Literature

A comprehensive overview of optimal signal control strategies adopted so far, SO methods in general, and the use of SO in managing urban traffic control at intersections is reviewed here.

2.1 Optimal Signal Control Design

Various optimal control strategies have been adopted in real time in managing traffic at signalized intersections. A few of them that are relevant to the present study and that would help in the choice of appropriate performance measure as objective are discussed in this section. A decentralized modeling approach of signal control is developed with an objective of maximizing throughput while penalizing for queue length [1]. Queue length minimization, which is a commonly used objective, involves placement of sensors at various locations and also fails to capture the diamond-shaped queuing pattern and hence not suitable for the heterogeneous and laneless traffic. Minimization of a weighted combination of the rate of delay and the number of stops is considered as the objective to arrive at an optimal start and duration of green at intersections [2, 3]. Number of stops might not be an appropriate measure of the inefficiency of signal control since vehicle might stop for various reasons such as side street traffic, roadside activities, vehicle failure or for any other personal reason [4, 5]. Attempted to minimize average vehicle delay with respect to HCM delay equation and a time-dependent delay formula, respectively, to achieve better performance. But, any of these delay equations do not hold good for our conditions.

Minimization of product of passenger occupancy and person delay of auto and transit vehicles, respectively, for a given cycle to determine whether a transit vehicle is ahead or behind schedule and design accordingly is proposed [6]. But, acquiring such data for individual vehicles of all the vehicle classes is impractical though it may be applicable only to a network size of two or three critical intersections during peak period. A weighted combination of travel delay and cycle length is considered as an objective [7] to ensure reduced delays and emissions along with safety. Such an objective might be reasonable in reciprocating the dynamics of the traffic and the required data may be acquired by a sample data as a measure of average travel time of the stream. Hence, this study considered the sum of travel times spent in traversing mid-block to mid-block locations of all the approaches of all intersections in the network as objective function of the proposed problem. A detailed discussion on the way in which optimization process was carried out is discussed in the subsequent subsection.

2.2 Simulation-Based Optimization in Optimal Traffic Signal Control Design

General framework of a simulation-based optimization process is slightly modified from [8] and depicted in Fig. 1. Both optimization and simulation run in parallel iteratively till termination criteria are met. Decision variable values as input variables are fed into the simulation software, and the simulation is run to yield the objective function value as an output from the program. The obtained functional values are verified with reference to a set of termination criteria to reach the best optimal signal

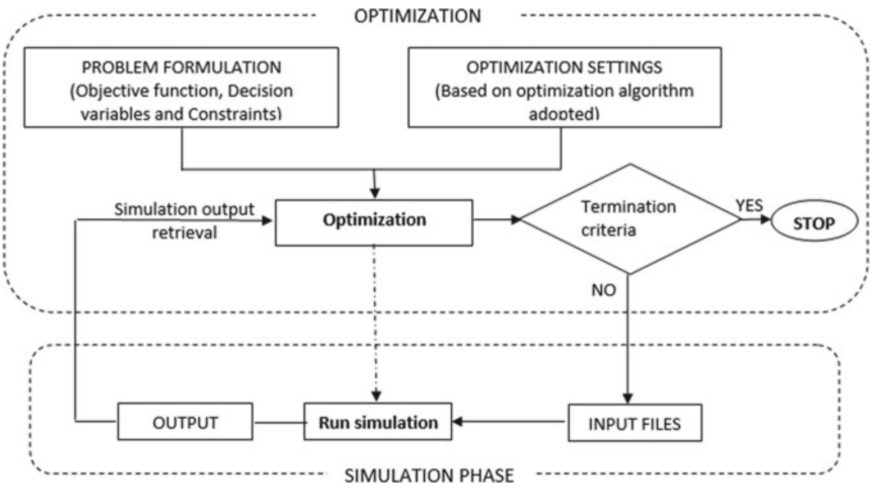


Fig. 1 General framework of a simulation-based optimization process

timings. The efficiency and accuracy of the optimization process are based on the choice of an appropriate algorithm.

SO problems are generally addressed by implementing any of the two basic algorithmic approaches based on the availability of derivative information of the objective function. First class of methods is being derivative-free optimization (DFO), where the optimization depends solely on the objective function evaluations based on input data. Such methods perform a grid or pattern search around the best initial approximate. Performance of derivative-free optimization solvers in handling various non-smooth functional forms, number of functional evaluations required till convergence was presented in [9]. The other is gradient-based optimization, which is further classified into direct gradient and metamodel methods [10]. The present study focuses on the category of DFO methods as a simple initial step toward optimal signal control. Therefore, further literature study focuses on the feasibility and applicability of these methods. DFO methods are classified into model-based DFO methods and direct search methods. However, for model-based DFOs, the number of functional evaluations required and their impact on achieving global convergence have always been a concern [11].

Direct search methods include algorithms such as exhaustive search, Hooke–Jeeves algorithms, coordinate search algorithm, mesh adaptive search algorithm, simplex algorithms including generalized pattern search (GPS) methods, Nelder–Mead simplex algorithm [8]. Metaheuristic methods such as evolutionary optimization, swarm intelligence, trajectory search (hill climbing, simulated annealing, etc.) that ensure global optima, though computationally intensive have been relied on often [8]. Out of these, optimization methods that are used for optimal traffic signal control are discussed in the subsequent paragraph.

Various combinations of simulation software and optimization algorithms have been used to attain better performance of signals at a network level [10, 12, 13]. Most of them led to signal plans with reduced variance in the average travel times, thereby improving travel time reliability. Though these methods are successful in achieving better network performance, a SO approach that optimizes any representative performance measure for heterogeneous traffic conditions has not been reported. Use of DFO methods that yield optimal or near-optimal signal plans at a faster rate of convergence is examined in this study and is presented in the subsequent sections.

3 Study Area and Input Data

A network with two four-legged two-way intersections was considered, and the same was modeled in VISSIM using Open Street Map as shown in Fig. 2. VISSIM calibrated for Indian traffic conditions [14, 15] was used in this study. Traffic volumes up to 4000 veh/h for major roads and up to 2000 veh/h for minor roads were generated based on field observations. The volume details are given in Table 1 and Fig. 3. Vehicles of four categories (two-wheelers, three-wheelers, four-wheelers, and bus) were considered, and the corresponding vehicle composition was fed as input as

Fig. 2 Road network in VISSIM

Table 1 Traffic volumes and link lengths of all approaches

Approach/link node	Length (km)	Volume (veh/h)					
		0–300 (s)	300–600 (s)	600–900 (s)	900–1200 (s)	1200–1500 (s)	1500–1800 (s)
WB1 (1)	1.2	650	1300	1950	2400	3000	3804
EB2 (2)	1.2	650	1300	1950	2400	3000	3804
NB1 (3)	1.4	300	650	1000	1350	1780	2013
SB1 (4)	1.4	300	650	1000	1350	1780	2013
NB2 (5)	1.4	300	650	1000	1350	1780	2013
SB2 (6)	1.4	300	650	1000	1350	1780	2013

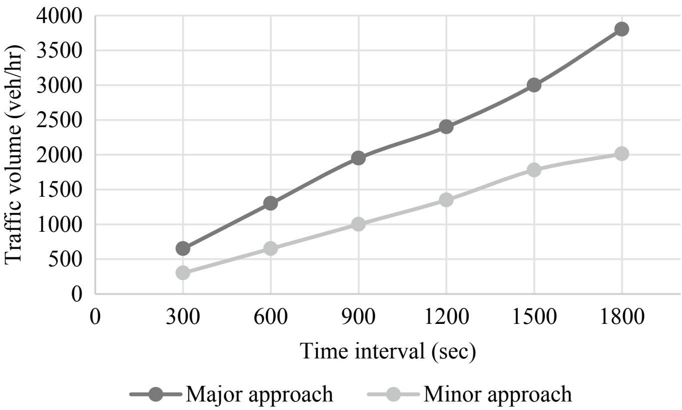


Fig. 3 Traffic flow variation of the given network during simulation period

shown in Table 2. Vehicular characteristics, including speed, acceleration, deceleration, braking, etc., were given as per the field scenario. The presence of pedestrians was ignored in the study.

Table 2 Details of vehicle composition and characteristics

S. No.	Vehicle class	Desirable speed distribution (km/h)	Relative flow
1	2W	60	0.476
2	3W	25	0.054
3	4W	50	0.447
4	Bus	20	0.023

The spacing between the intersections was 1.01 km for which an ideal offset of 17 s was used. The turning proportion from minor road to major road was taken as 40% of the total approach volume and toward the minor road approach as 25%. Since there were no dedicated turning lanes of the network, split phasing was assumed resulting in four phases at each intersection.

Various travel time measurement detectors from mid-block to mid-block sections along with queue counters at upstream and downstream locations were placed at all the approaches. The collected data included details of average flow, average speed, percentage occupancy, and delay.

4 Methodology

The initial part of this section includes the model formulation of the optimization problem discussed along with the initialization and termination criteria to define the problem in terms of optimization parameters. The later part deals with the implementation of SO algorithm based on the choice of DFO algorithms to execute optimization and attain least possible value of the objective function.

4.1 Problem Formulation

As discussed earlier, the objective function was to minimize the sum of travel times incurred in traversing all possible movements from all the approaches of all the intersections of the network with green splits and cycle length being the decision variables, which can be represented as

$$\text{Min.} \sum_{i \in I} \sum_{j \in J} tt_{ij}\left(g_{ij}, C_i\right). \tag{1}$$

Subject to the following constraints.

- All variables expressing signal timings and flow rate are non-negative

$$g_{ij} \geq 0, \quad C_i \geq 0. \tag{2}$$

- Constraint on minimum and maximum green times

$$g_{\min} \leq g_{ij} \leq g_{\max}. \tag{3}$$

- Constraint on minimum and maximum cycle lengths

$$C_{\min} \leq C_i \leq C_{\max}. \tag{4}$$

- Relation between cycle length and green times

$$\sum_{j=1}^{n} g_j = (C - L) \quad \forall i \in I. \tag{5}$$

where

I	Set of all intersections in the network
J	Set of all possible phases of an intersection
tt_{ij}	Travel time experienced in traversing jth phase of ith intersection of network
g_{ij}	Green duration of jth movement of ith intersection in the network
C_i	Cycle length of of ith intersection in the network
L	Total lost time per cycle

Initialization. The following values were assumed to initiate the SO process.

- Green bounds
  The minimum green time for vehicles on a major approach was taken as the minimum pedestrian green required to cross the minor street approach, which came to be 13 s for a carriageway width of 7.0 m.

$$g_{\min} = 13 \text{ s}$$

$$g_{\max} = 120 \text{ s}$$

  - Cycle length was assumed to be constant over all the intersections of the network.
  - Cycle length bounds
    The minimum cycle time was computed as sum of minimum green time with initial amber and clearance amber of 2 s each over all phases [16]. Maximum cycle length is assumed based on the field traffic volumes on Indian roads during peak periods.

$$C_{\min} = 70 \text{ s}$$

$$C_{\max} = 250 \text{ s}$$

Termination Criteria. Tolerance values for values of objective function and decision variable vector to stop the SO process were assumed as follows.

- Difference in the value of objective function

$$|f_k - f_{k-1}| \leq f_{\text{tol}}. \tag{6}$$

- Difference in the value of design variables

$$|x_k - x_{k-1}| \leq x_{\text{tol}}. \tag{7}$$

where

f_{tol} Tolerance level of functional value: 0.001
x_{tol} Tolerance level of decision variable: 0.001

4.2 *Implementation of SO Algorithm*

The choice of SO algorithm depends on the nature of the optimization problem being dealt such as linear or nonlinear programming problem, constrained, and unconstrained. Since the objective function of the proposed formulation does not have a functional form, the nature of the optimization problem cannot be determined. Therefore, certain DFO methods from the literature that can be applied to any type of signal control optimization problem without the need of gradient information were chosen and implemented using scientific package of python called SciPy [17] to compute the optimal values of decision variables. The description of various algorithms is listed below.

(i) Nelder–Mead simplex method: It is a heuristic search method, applied to nonlinear optimization problems for which derivatives may not be known. It evaluates the objective function values at vertices of a simplex, and each iteration typically requires a few objective function evaluations, rendering it computationally less expensive than many other methods [18].
(ii) COBYLA method: It is a constrained optimization BY linear approximation algorithm that handles both linear and nonlinear constraints of any kind of optimization problem. It is considered to be one of the advanced nonlinear optimization methods that iteratively approximates the actual non-linear optimization problem with linear programming problems and is used for distributed nonlinear urban traffic control problem [19].

The model-based algorithms chosen perform efficient local search around the initial input values to yield a local optimum of the optimization problem. Hence, the best approximation of initial value reveals the efficiency of the algorithm and the reliability of the obtained solution. The detailed flowchart on the operation of a SO is shown in Fig. 4.

As depicted in the flowchart Fig. 4, initial values are fed to the simulator for the first run to complete over the whole simulation period (an integral multiple of cycle length). The total travel time experienced in the first iteration as a result of the initial signal plan of the network is obtained as output from the simulator. The next best approximate of the input signal timing values is determined by the respective algorithm yielding a travel time experienced during the current iteration. Both the objective functional values of previous and current iterations are compared with respect to termination criteria, and the similar procedure is continued till the criteria are met. The obtained optimal results are presented in the subsequent section.

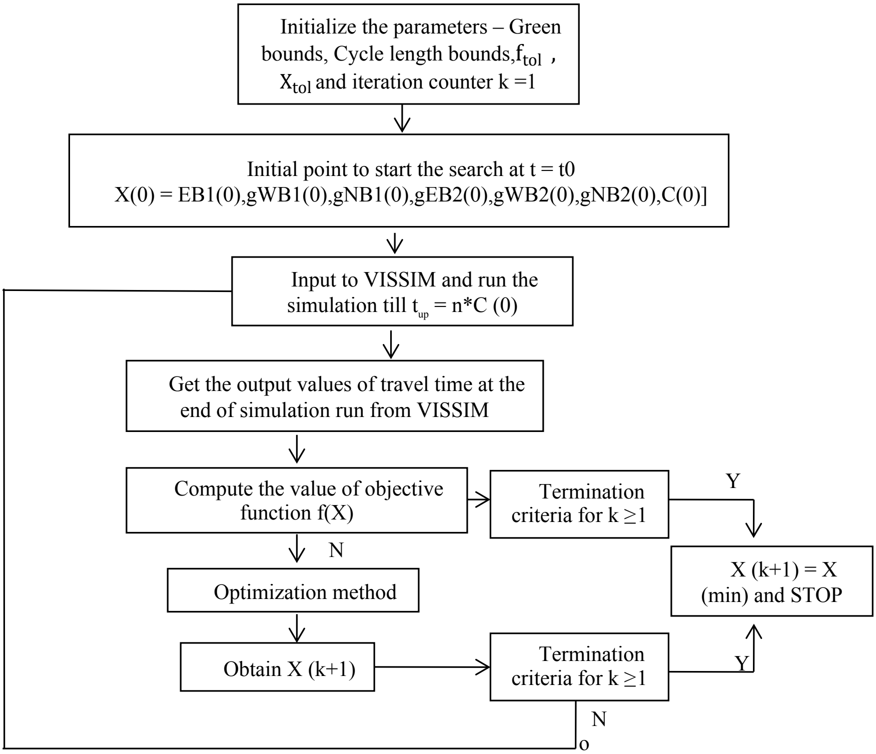


Fig. 4 Flowchart of optimization algorithm

5 Results and Discussion

The initial part of this section accomplishes signal design using Webster's and HCM methodologies. The latter part presents minimal objective functional values obtained by implementing various derivative-free algorithms.

5.1 *Signal Design Using Conventional Webster and HCM Methods*

This subsection describes the design of traffic signal based on both Webster's and HCM methods for the given peak hour volume from Table 3.

Webster method. Signal control parameter values used are shown in Table 3. Required cycle length was then calculated using Eq. (8) and was demonstrated in Table 4 for first intersection. The signal design of second intersection also resulted in similar values.

$$C = \frac{(1.5 * L + 5)}{\left(1 - \sum \frac{q_c}{s}\right)} \tag{8}$$

Table 3 Initial design parameters of Webster's design

Signal design parameters	
Lost time/phase (L)	2.5
No. of phases (n)	4
All red time (R)	2
Total lost time ($nL + R$)	12
Saturation flow(s) in veh/h/lane	4000
No. of lanes of each approach	2
Saturation flow total(s) in veh h	8000

Table 4 Signal timing for first intersection based on Webster's design

Approach	Flow rate (q) in veh/h	$Y = q/s$	Cycle length (s)
WB1	2013	0.252	−55.05
NB1	2013	0.252	
SB1	3512	0.439	
EB1	3804	0.476	
		$\Sigma Y = 1.42$	

Table 5 Initial design parameters and cycle length for first intersection based on HCM design

Signal design parameters		Cycle length (s)
Vc (tvu/h/ln)	2589	−42.17
Field PHF	0.83	
Sat. Flow (tvu/h/ln)	4000	
v/c (assumed)	0.9	
Lost time per phase (s)	4	
No. of phases	4	
Total lost time (s)	16	
Saturation flow total (veh/h)	8000	
Cycle length (s)	120	

The obtained cycle length based on Webster's design was found be negative, indicating that the current geometric and control conditions cannot handle the peak hour volumes.

HCM method of signal design. Similar procedure was adopted based on HCM design methodology [20], and the details are given in Table 5. The obtained cycle length of the intersection was obtained using Eq. (9) to start the simulation run.

$$C = \frac{L}{\left[1 - \frac{V_c}{\text{Sat flow}*\text{PHF}*\frac{v}{c}}\right]} \tag{9}$$

Similar to Webster's, HCM method also yielded a negative cycle length value because of high critical lane volume of the intersection. Therefore, it can be inferred that HCM signal design method may not be able to handle the current traffic scenario, which is frequent on a day-to-day basis, especially during morning and evening peak periods on Indian urban roads.

Therefore, in order to handle such a traffic situation subject to high volumes and fluctuations, an optimal signal design control that can handle the dynamics and magnitude of traffic volumes is necessitated. The subsequent section presents the signal timing resulted in implementing the proposed SO method.

5.2 Signal Design Based on the Proposed SO Method

As discussed in Sect. 4.2, two types of DFO algorithms were implemented to solve the proposed network travel time minimization problem by integrating VISSIM and Python. Initial signal times were assumed proportional to the peak approach volumes. The results obtained in terms of optimal green splits and cycle length along with the associated objective functional values are presented in Table 6 with Figs. 5 and 6

Table 6 Optimal signal design parameters of various optimization algorithms

Method	g_WB1	g_EB1	g_NB1	g_SB1	g_WB2	g_EB2	g_NB2	g_SB2	*C*	*f* (s)
Initial	99	99	43	43	99	99	43	43	300	3902.76
Nelder-Mead	102	97	42	43	96	102	42	44	309	3846.48
COBYLA	99	100	42	43	99	100	43	42	300	3898.07

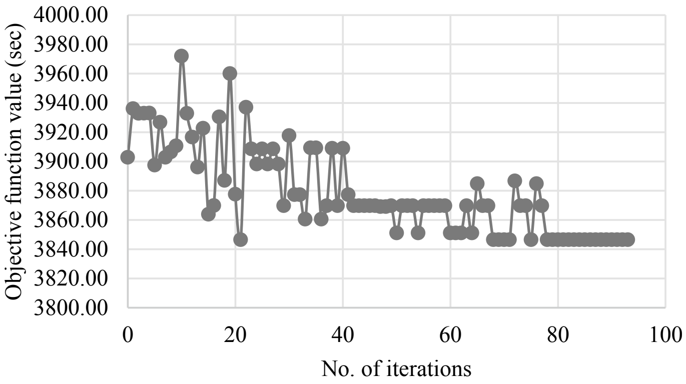


Fig. 5 Convergence of objective function value using Nelder–Mead

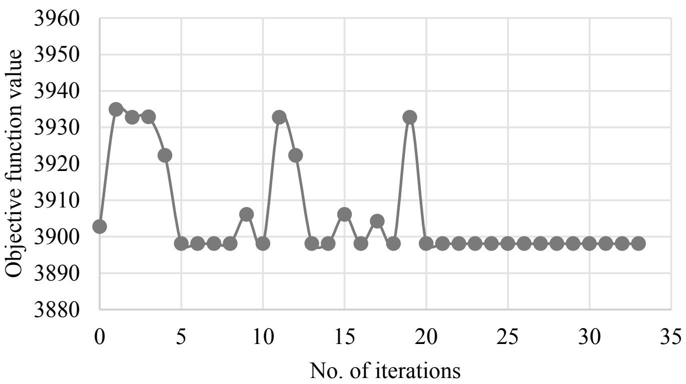


Fig. 6 Convergence of objective function value using COBYLA

presenting the convergence of Nelder–Mead and COBYLA optimization methods to the minimum possible value with respect to iterations.

From Table 6, it can be observed that the total travel time based on optimal signal timings obtained from Nelder–Mead was relatively better than that of COBYLA with almost same computational time. Also, from Figs. 5 and 6, it was also found that Nelder–Mead method explored for optimal value on either side of initial value as search domain while COBYLA performed only on one side of the initial value.

Hence, it can be concluded that Nelder–Mead outperforms COBYLA and explores over larger domain space making it more reliable for this application. Moreover, the existing signal design methodologies failed under the geometric and traffic conditions indicating the need for such real-time optimization solutions. However, the convergence toward global optima is uncertain in direct search DFO methods owing to lack of information on gradient of the objective function value.

6 Summary

The study is aimed at achieving optimal signal settings in real time that facilitates proportionate utilization of green durations to the traffic flow, thereby reducing the average vehicle delay at intersections. This would improve travel time reliability and lane carrying capacity of the congested urban roads with an improved Level of Service on urban networks. It was found that the traditional Webster and HCM signal design methodologies yielded negative cycle length values for a given peak input flow rate of congested traffic flow conditions. Therefore, this paper presented a simulation-based optimization approach toward optimal traffic signal design through VISSIM microsimulation software to cater to the fluctuations in high arrival flow rates during peak periods. Various DFO algorithms were used, and a comparative analysis among them was carried out to achieve minimum total travel time incurred in traversing through intersections of an urban network. It is found that an optimal cycle length of 309 s yielded the best possible minimum total travel time value of 3847 s over the entire network for the whole simulation period. It was observed that Nelder–Mead simplex algorithm performed better in attaining minimal value, and it may be opted as a reliable replacement to the conventional design methodologies such as Webster and HCM methods under heavy traffic conditions. It so happened in the proposed method that the best approximate of the initial solution nearer to the optimal value led to faster convergence.

This paper presented a simple methodology over a small possible network subjected to traffic volume variation for every five minutes in undersaturated conditions. Assumptions of the absence of side road activities, pedestrian volume, interactions between vehicles and pedestrians, safety measures, and other factors can be relaxed, and additional constraints to deal with such situations can be considered as future extensions of this work. Comprehensive approaches inclusive of all the above factors, using more robust SO algorithms such as approximate gradient-based SPSA algorithms or other metamodels, can be explored.

Acknowledgements The data collection effort in this study was made possible through sub-project "Enhanced Traffic Mobility using Signal Improvements" Project No: CR/17-18/CIE/001/LNTE/LELI. The authors acknowledge the support of the project sponsors Larson & Toubro Technology Services. Also, the authors acknowledge the opportunity to submit the research work at the 5th Conference of the Transportation Research Group of India held at Bhopal (India) from December 18–21, 2019, that forms the basis of this article.

References

1. Al Islam SMAB, Hajbabaie A (2017) Distributed coordinated signal timing optimization in connected transportation networks. Transp Res Part C Emerg Technol 80:272–285. https://doi.org/10.1016/j.trc.2017.04.017
2. Wong SC, Wong WT, Leung CM, Tong CO (2002) Group-based optimization of a time-dependent TRANSYT traffic model for area traffic control. Transp Res Part B Methodol 36:291–312. https://doi.org/10.1016/S0191-2615(01)00004-2
3. Chang TH, Lin JT (2000) Optimal signal timing for an oversaturated intersection. Transp Res Part B Methodol 34:471–491. https://doi.org/10.1016/S0191-2615(99)00034-X
4. Zakariya AY, Rabia SI (2016) Estimating the minimum delay optimal cycle length based on a time-dependent delay formula. Alexandria Eng J 55:2509–2514. https://doi.org/10.1016/j.aej.2016.07.029
5. Lee J, Williams BM (2012) Development and evaluation of a constrained optimization model for traffic signal plan transition. Transp Res Part C Emerg Technol 20:185–198
6. Christofa E (2013) Paper No. 13–3395 Arterial traffic signal optimization: a person-based approach. Trb
7. Memoli S, Cantarella GE, de Luca S, Di PR (2017) Network signal setting design with stage sequence optimisation. Transp Res Part B Methodol 100:20–42. https://doi.org/10.1016/j.trb.2017.01.013
8. Nguyen AT, Reiter S, Rigo P (2014) A review on simulation-based optimization methods applied to building performance analysis. Appl Energy 113:1043–1058. https://doi.org/10.1016/j.apenergy.2013.08.061
9. Moré JJ, Wild SM (2009) Benchmarking derivative-free optimization algorithms. SIAM J Optim 20:172–191. https://doi.org/10.1137/080724083
10. Osorio C, Bierlaire M (2013) A simulation-based optimization framework for urban traffic control. Oper Res 61:1333–1345. https://doi.org/10.1287/opre.2013.1226
11. Hare WL, Jaberipour M (2015) Adaptive interpolation strategies in derivative-free optimization: a case study. arXiv Prepr arXiv151102794
12. Baldi S, Michailidis I, Ntampasi V et al (2019) A simulation-based traffic signal control for congested urban traffic networks. Transp Sci 53:6–20. https://doi.org/10.1287/trsc.2017.0754
13. Li S, Bin LY, Fu BB, Dang WX (2016) Study on simulation optimization of dynamic traffic signal based on complex networks. Procedia Eng 137:1–10. https://doi.org/10.1016/j.proeng.2016.01.228
14. Anand A, Ramadurai G, Vanajakshi L (2014) Data fusion-based traffic density estimation and prediction. J Intell Transp Syst Technol Planning Oper 18:367–378. https://doi.org/10.1080/15472450.2013.806844
15. Mathew TV, Radhakrishnan P (2010) Calibration of microsimulation models for Nonlane-based heterogeneous traffic at signalized intersections. J Urban Plan Dev 136:59–66. https://doi.org/10.1061/(asce)0733-9488(2010)136:1(59)
16. Indian Roads Congress (1996) IRC:93–1985 guidelines on design and installation of road traffic signals
17. Jones E, Oliphant T, Peterson P (2014) {SciPy}: open source scientific tools for {Python}
18. Bowman CN, Miller JA (2016) Modeling traffic flow using simulation and big data analytics. In: Proceedings of the 2016 winter simulation conference, pp 1206–1217
19. Prikryl J, Novotny J, Šmídl V (2015) On distributed traffic signal control. IEEE Conf Intell Transp Syst Proc 894–899. https://doi.org/10.1109/ITSC.2015.150
20. Transportation Research Board (2010) Highway capacity manual 5th edition

Intersection Identification Using Large-Scale Vehicle Trajectory Data

Tingting Huang and Anuj Sharma

1 Introduction

Road intersection is an important facility in the traffic system. It generates many points of conflict in traffic flows, thus, both mobility and safety are vital issues in intersection management. Before any intelligent transportation application deployed on intersections, the first task should be correctly identifying the location of intersections in a large network. Right now, the geographic information of road intersection highly relies on the open source geographic database, which may not always up to date or have missing points. To generate a more complete geographic database for intersections from large-scale network, other automatic methods should also be explored.

Recently, lots of vehicles and navigation device have equipped with global positioning system (GPS) receivers or loggers. These devices could generate frequent and high resolution location information, which can be a new source of spatial data for traffic management. This kind of data could represent traffic patterns both in spatial and temporal because the full trajectory of a vehicle could be extracted from a series of GPS locations. Millions of GPS records could happen on a large-scale network, and the trajectories they formed can interact with each other, thus, the road intersection could be identified by mining the GPS data.

The advantages of using GPS data include its low cost (no additional devices needed) and non-aggregated information. There are many studies have been conducted using GPS trace to find geographic information of the road network. Yang

T. Huang (✉)
ETALYC, Inc., Ames, IA, USA
e-mail: tingting.huang@etalyc.com

A. Sharma
Iowa State University, Ames, IA, USA
e-mail: anujs@iastate.edu

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_10

et al. [1] used a multi-hierarchy feature extraction strategy to generate lane-based intersection maps from crowdsourced big trace data. They used turning change point pair as the fundamental elements to define intersections. They extracted those pairs from GPS trace and spatially clustered them based on distance and angle constraints. By trying different parameter combinations, they generated all the locations of intersections and compared them with satellite images, the accuracy of road intersection recognition was about 95% and the recall rate was about 92%.

Other methods on extracting information from GPS data have also been explored by researchers. Wang et al. [2] used entropy analysis and clustering method to identify intersections from GPS locations. They focused on the diversity of headings exists in intersection area, used Shannon entropy to analyze the headings in each cell of gridded network. As high entropy should indicate intersection area, further they clustered those close cells to form an intersection. There are also several studies working on traffic lane information extraction or map-matching using GPS trajectory data [3, 4]. Other methods like k-means clustering [5–7], localized shape descriptor [8] and spatial analysis in circular boundaries [9] were also developed by researchers. In this paper, we will focus on intersection identification with massive, noisy GPS trajectories.

This paper proposes a new method on intersection identification. First, we extract all the turning points of all trajectories. Second, we use a clustering method to find the groups of those turning points. Last, we use direction discrepancy index (DDI) and turning discrepancy index (TDI) to analyze each cluster, to remove those false detections. By using this method, a city-scale intersection map can be generated. Compare to the open source geographic database, Open Street Map (OSM), we can detect intersections not included in OSM, which makes this method a good supplement to open source database. Additionally, this method is scalable and easy to implement with few interventions.

2 Methodology

2.1 Turning Point Extraction

Generally, turning point is referring to the point that a vehicle changes the heading obviously. However, since the heading information is not available in our dataset, here we use the bearing (b) change as a substitute. Besides the bearing change, another factor that should also be considered is the duration of motion (t). Because the GPS data many have missing points in a trajectory, thus, only points recorded in a short period will be considered. Figure 1 shows the definition of a turning point.

In Fig. 1, $p1$, $p2$ and $p3$ represent the 3 consecutive GPS points of one vehicle; $t1$ and $t2$ are the time duration (in seconds) between $p1$ and $p2$, $p2$ and $p3$, respectively; $b1$ and $b2$ are the bearing (in degree) from $p1$ to $p2$, $p2$ to $p3$, respectively.

The turning point needs to satisfy the following conditions.

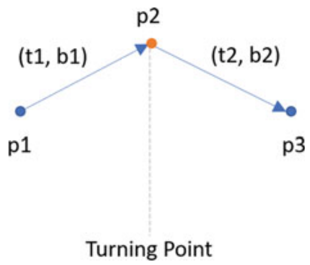


Fig. 1 An illustration of turning point definition

$$t1 + t2 <= 30 \tag{1a}$$

$$60 < |b1 - b2| < 300 \tag{1b}$$

2.2 *Density-Based Clustering*

To find the intersection area out of massive vehicle turning points, a density-based clustering method is used. We adopt density-based spatial clustering of applications with noise (DBSCAN) [10] as the clustering method, with the advantage that no need to know the number of clusters beforehand. For the parameter used in clustering algorithm, we tried different combination on our case and set epsilon as 0.0002 and minimum samples as 15 for our one-week data.

2.3 *Cluster Analysis*

As not all the turning movement happened at intersection, some cluster from turning points should be examined carefully. For example, lots of turning motion can happen on a ramp to highway, where all the vehicles are turning right. And some road may have curvatures, which can also lead to a group of turning points when they are driving on the curved road. Thus, we develop two types of index to analyze those behaviors in cluster.

The Direction Discrepancy Index (DDI) is defined as follows.

$$\mathrm{DDI} = \frac{\sum\left|\frac{Ni}{N} - 0.25\right|}{N} \tag{2}$$

where Ni is the number of points with driving direction (i.e., eastbound, westbound, etc.) i. If there are less than 4 directions, DDI is set to 1.

The Turning Discrepancy Index (TDI) is defined as follows.

Table 1 Sample raw data

Trip ID	Waypoint sequence	Capture date	Latitude	Longitude
1bae53423cdafa9fd11691c	13	1/30/17 17:23:52	41.6497	−93.6046
1bae53423cdafa9fd11691c	14	1/30/17 17:24:52	41.6495	−93.6257
1bae53423cdafa9fd11691c	15	1/30/17 17:25:52	41.6499	−93.6466
1bae53423cdafa9fd11691c	16	1/30/17 17:26:45	41.6508	−93.6675

$$\mathrm{TDI} = \frac{\sum\left|\frac{Nj}{N} - 0.5\right|}{N} \tag{3}$$

where Nj is the number of points with turning direction (i.e., left, right, through movement) j.

The threshold to filter out non-intersection turning point clusters from these two indices are tuned in our case study. A visualization tool is developed and the final tuned threshold for DDI is 0.35, and for TDI is 0.3.

3 Experiment

3.1 Data Used

The GPS location data in this case study is provided by INRIX [11] with an accuracy around 5 m. There are about 2 million of records in one week from 29, January to 04, February 2017 in the study area (the state of Iowa, USA), thus, we focus on the major city in Iowa–Des Moines metropolitan area. A sample of information included in the dataset is shown in Table 1 and Fig. 2. The individual GPS device information is anonymized.

3.2 Turning Point Extraction and Clustering

By applying the proposed method, we can extract all the turning points from all the trajectories. Figure 3 illustrates a subset of turning points on map. The blue dots are extracted points, and apparently, they are mostly located around intersection area.

After turning point extraction, we apply DBSCAN method to find clusters among those points. As mentioned in methodology, we tried different combination of parameter settings, and the final one is selected based on our visual examinations. The results of turning point clustering are shown in Fig. 4.

Fig. 2 An illustration of sample trajectory

Fig. 3 Turning point extraction results (partial)

3.3 DDI and TDI Filtering

After clustering the turning points, the in-cluster analysis is done by computing the DDI and TDI for each cluster. By using the tuned thresholds for each index, the non-intersection clusters are filtered out. Figure 5 illustrates how the two indices are detecting those invalid clusters.

Fig. 4 Turning point clustering results (partial). Note that the color is just to distinguish different clusters, no actual meanings involved

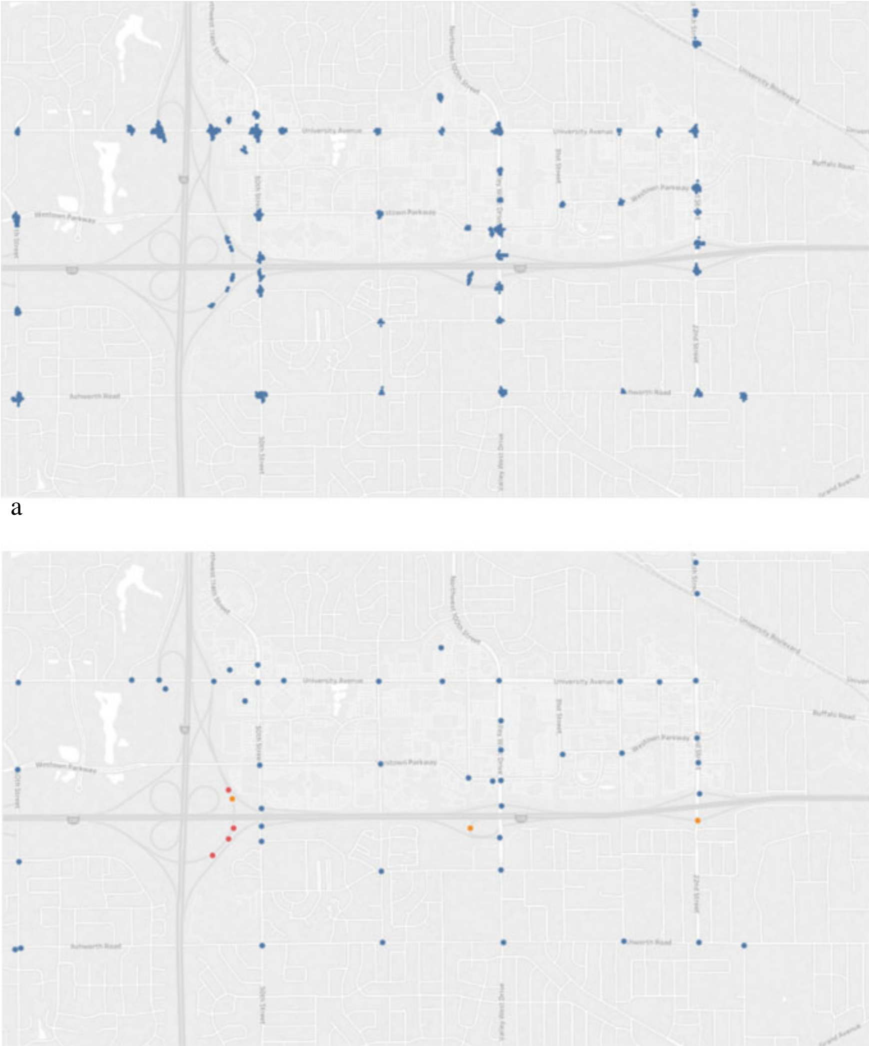
Figure 5a is showing the original turning points, while Fig. 5b is showing the DDI and TDI results on top of Fig. 5a. In Fig. 5b, the blue dots are regular intersection-like clusters (only show the center); the orange dots are clusters with TDI over 0.3; the red dots are clusters with both TDI over 0.3 and DDI over 0.35. It can be seen that those red and orange dots are located on the ramps with uniform turning direction and driving direction.

4 Results

After the experiment with one-week GPS trajectory data, we identified 483 intersections in Des Moines metropolitan area. Additionally, we also download the data from OSM to compare with our results. Figure 6 shows our results of intersection locations extracted from GPS data (blue dots), overlapping with OSM locations (orange dots).

In Fig. 6, most intersections can be captured by our method using GPS data, however, due to the data coverage, some place may not be identified. In the red box, there are a whole corridor of intersections is missed by OSM data but captured by our method with GPS data. Table 2 summarizes the number of intersections obtained from OSM and our method.

Since GPS data are generated by probe vehicles which can reflect the volume, thus, those intersections missed by OSM but with good GPS data coverage (potential high traffic volume) can be identified by our method. Then, our method can be a good supplement to the open source geography database.



a

b

Fig. 5 **a** Extracted turning points, **b** turning point clusters with DDI and TDI filters applied

5 Conclusion

This paper proposes a new method on intersection identification from massive GPS trajectory data on a large-scale network. To find the intersections, we focus on the turning movement and extract all the turning points from all the trajectories. By clustering the turning points, the intersection area can be identified. Further, we examine those clusters by using direction discrepancy index (DDI) and turning discrepancy

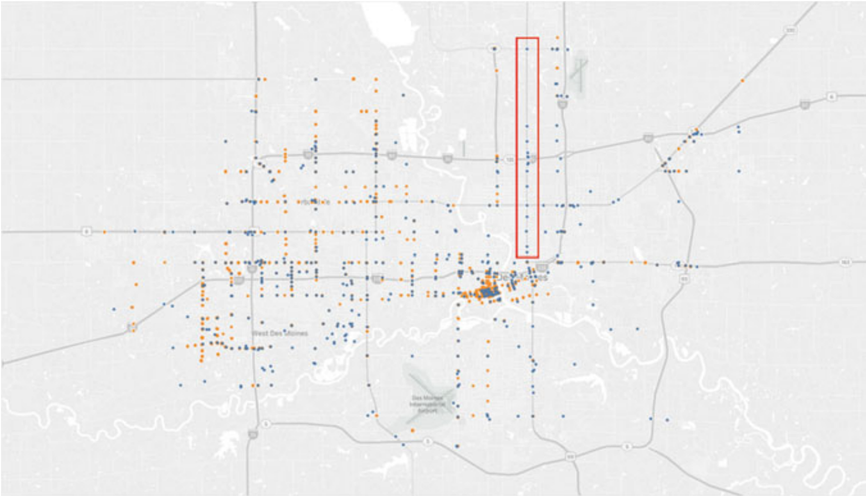


Fig. 6 Intersection location extracted from GPS (blue) and OSM (orange). The intersections missed by OSM are captured by our method (highlighted in red box)

Table 2 Comparison of our results with OSM database

Type	Number of intersections*
Our detection (total)	483
Our detection (not in OSM)	225
OSM database (total)	954
OSM database (not detected)	485

*Number of intersections in OSM is large because OSM is showing the location of every individual signal device, which generate multiple points for one intersection location

index (TDI). Those indices indicate the turning behaviors in the cluster and can be used as a filter to eliminate wrong detections.

A case study is conducted on one-week GPS data from the state of Iowa, USA. Two millions of records are processed and 483 intersections are identified in Des Moines metropolitan area. Compared to open street map (OSM) database, we identify 225 intersections that are missed by OSM.

This method can be a supplement to open sourced geographic information database. By using GPS data, large-scale network can be identified. After obtaining the intersection map, one can evaluate the mobility and safety performance of a network with focus on these conflict points, such as identify the bottleneck of the network with the knowledge of intersection locations. In future, the trajectory data can also be used for computing delay, travel time reliability and other performance measures.

References

1. Yang X, Tang L, Niu L, Zhang X, Li Q (2018) Generating lane-based intersection maps from crowdsourcing big trace data. Transp Res Part C Emerg Technol 89:168–187
2. Wang C (2017) Intersection and stop bar position extraction from crowdsourced GPS trajectories. In: Transportation research board 96th annual meeting, Washington D.C.
3. Tang L, Yang X, Kan Z, Li Q (2015) Lane-level road information mining from vehicle GPS trajectories based on Naïve Bayesian classification. ISPRS Int J Geo Inf 4(4):2660–2680
4. Lou Y, Zhang C, Zheng Y, Xie X, Wang W, Huang Y (2009) Map-matching for low-sampling-rate GPS trajectories. In: Proceedings of the 17th ACM SIGSPATIAL international conference on advances in geographic information systems—GIS'09, p 352
5. Schroedl S, Wagstaff K, Rogers S, Langley P, Wilson C (2004) Mining GPS traces for map refinement. Data Min Knowl Discov 9:59–87
6. Edelkamp S, Schrodl S (2003) Route planning and map inference with global positioning traces. In: Klein R, Six H-W, Wegner L (eds) Computer science in perspective, vol 2598. Lecture notes in computer science. Springer, Berlin/Heidelberg, pp 128–151
7. Worrall S, Nebot E (2007) Automated process for generating digitised maps through GPS data compression. In: Australasian conference on robotics and automation
8. Fathi A, Krumm J (2010) Detecting road intersections from GPS traces. In: Proceedings of the 6th international conference on geographic information science, Zurich, Switzerland, pp 59–69
9. Wang J, Rui X, Song X, Tan X, Wang C, Raghavan V (2015) A novel approach for generating routable road maps from vehicle GPS traces. Int J Geog Inf Sci 29(1):69–91
10. Ester M, Kriegel HP, Sander J, Xu X (1996) A density-based algorithm for discovering clusters in large spatial databases with noise. KDD 96(34):226–231
11. INRIX Homepage. http://inrix.com/. Last accessed 5 Jun 2019

Crowd Density Prediction Using Kalman Filtering Technique

H. Gayathri, P. S. Karthika, and Ashish Verma

1 Introduction

Monitoring the crowd movement using surveillance systems has become an integral part of mass gatherings of religious and non-religious nature. The motivation for actively making use of automatic surveillance systems such as CCTVs is rooted in the wake of several crowd disasters in past, for instance, the Love parade stampede in Germany. Religious gatherings and pilgrimages have been venues for 79% of stampedes in India [2]. Regular monitoring of crowd in such situations can help in identifying possible crowd risk situations including crowd crush and stampedes. The event management authorities can utilize quantifiable aggregate measures of crowd flow to make strategic dynamic decisions to ease the risk situations. Crowd density is an important parameter of crowd flow that can be used to make such strategic decisions for managing large number of people. The number of pedestrians per square meter gives the crowd density, which is an indication of the intensity of crowd. As crowd density increase, there are chances of pressure building up within the crowd, and in the absence of proper measures to dissipate this pressure, crowd crush can happen. Therefore, modeling the spatiotemporal variation of density patterns can be used as an input to give an early warning in case of high densities. The objectives of this study are to estimate concentration of density across different prime locations in the study area and to predict the density patterns using Kalman Filter Technique (KFT) and comparing the concentration of densities. Kumbh Mela (Ujjain-2016), a mass Hindu pilgrimage, where people across the world gather to take part in religious, is considered as our study area.

H. Gayathri · P. S. Karthika · A. Verma (✉)
Transportation Systems Engineering (TSE), Department of Civil Engineering, Indian Institute of Science (IISc), Bengaluru, Karnataka 560012, India
e-mail: ashishv@iisc.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_11

The remainder of the paper is organized as follows: In the next section, we present a short description of related studies. Details of the study area, data collection and data extraction are presented in Sect. 3. Section 4 includes the particulars of analysis conducted including the methodology adopted for predicting crowd density. Finally, we conclude with the discussions of the study, and possible future work.

2 Literature Review

Pedestrian behavior in crowd can be modeled macroscopically and microscopically. In macroscopic, the focus is on relationships between density, distributions, speed and flow such as hydrodynamic model and first-order flow models [3, 4], whereas in microscopic, models are developed such as social force model [5], cellular automaton model [6, 7], and other agent-based simulations. From a macroscopic standpoint, studying aggregate figures such as density is imperative to identify risk levels at various locations. Many researchers have reported maximum density values where the critical boundary begins such as 7 persons/m^2 [8], 6 persons/m^2 [9]. However, a study conducted in Hajj reported that average speeds does not even go to zero at densities as high as 10 persons/m^2 [10]. Estimating densities even when there are only a few pedestrians is challenging, as the number of samples will be low [11].

Several studies have used Kalman filter to predict different traffic variables including traffic flow, travel time etc. Kumar [12] predicted traffic flow using historic traffic flow data for two days, and real-time data for the considered day using Kalman filtering technique. Another study makes use of Kalman filter technique to predict travel time for buses in Bangalore [13]. Also, there are few studies on tracking pedestrians using Kalman filters where algorithms using extended and simplified complimentary Kalman filters to improve the accuracy with respect to positioning of raw data [14] and recognizing pedestrians and localizing them with Kalman filter estimators [15]. Mahakian [16] has combined Kalman filter and Kuhn–Munkres algorithm to predict pedestrian locations and get their walking trajectories.

3 Study Location, Data Collection and Extraction

Kumbh Mela is one of the world's largest mass religious gathering in which millions of people gather for worship and take a dip in a river that is considered sacred. Data for this work is collected during Kumbh Mela 2016, Ujjain. Three locations are considered for the study: Chardham (place of worship), Mahakal (place of worship) and Bombay Dharamshala (major resting place). The locations are chosen in such a way that those be either of prime importance where people perform certain activities or cross the sections to reach prime activity locations. For example, people cross Bombay Dharamshala to reach Ram ghat where they take a dip. The widths of these sections are different from one another. Major bathing dates are predetermined before

Fig. 1 Locations considered for the study

the Kumbh Mela starts. These days, the crowd is expected to be heavy compared to other days. CCTV footage for the selected three locations are collected during May 9, 2016, which is one of the major bathing dates. Data is manually extracted from the footages for two hours (7–9 a.m.). A trap length is marked on the video and the number of persons in the trap length is counted to get the densities for every one-minute. The same procedure is used for all the locations to get their densities. However, this method carries a major drawback, i.e., as the density is measured in one-minute interval, density depends discontinuously on time [17]. Figure 1 shows the three locations considered for the study. Figure 2 shows the snapshots of three locations with trap lengths marked.

4 Analysis

4.1 Comparison of Densities

Figure 3 shows the time series plot for all three locations. Density in pedestrians per meter square is plotted versus time in the one-minute interval. The maximum density

(a) Chardham (b) Mahakal

(c) Bombay Dharamshala

Fig. 2 Snapshots of three locations with trap lengths marked

observed at Chardham was around 1.5 ped/m^2. Chardham serves as an entrance to the Mela area as the different activity locations can be accessed through this node. Another point of attraction in the study area is Mahakal temple, where the pilgrims visit in large numbers. Maximum density of 1.2 ped/m^2 was observed at this node. The third location considered was the major resting place for pilgrims, Bombay Dharamshala, which in turn provides access to pilgrims wanting to take dip. Maximum density observed here is 1.8 ped/m^2. Being an important date for the event, various religious processions were happening throughout the study area. Due to this, the movement of pilgrims was restricted at various stretches, resulting in lower pedestrian densities even at the prime activity locations during the considered time period.

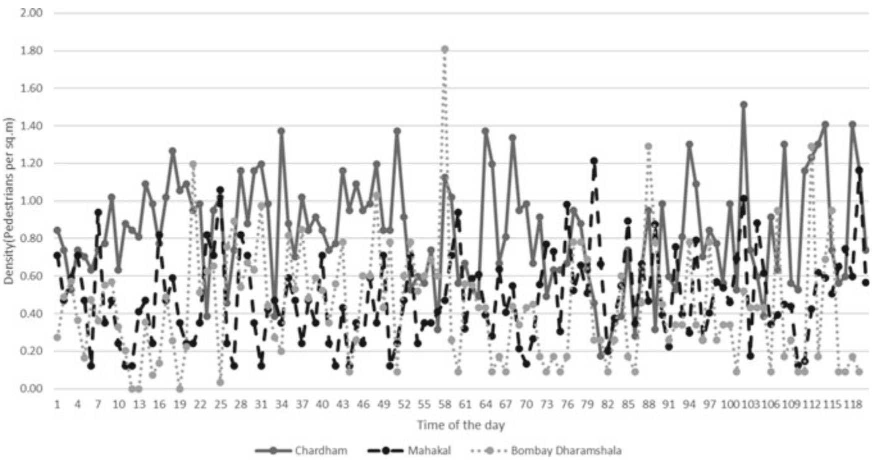


Fig. 3 Time series plot for all three locations

4.2 Predicting Densities Using KFT

Kalman Filter Technique. Video data extraction is a highly uncertain process. It includes errors from randomness in movement, measurement error, time-lapse error, etc. Kalman filter is an iterative process developed by Kalman [18] that uses a set of equations and data inputs to estimate the true value and eliminates error in the data. It estimates the future values by taking into account the error in the process and measurement. One advantage of Kalman filter is that it minimizes the mean square error of the estimated parameters that are difficult to measure in real grounds [19].

The Kalman filter tries to estimate the state $x \in \mathbb{R}^n$ of discrete-time controlled process headed by the linear stochastic differential equation

$$x_k = Ax_{k-1} + Bu_{k-1} + w_{k-1}, \tag{1}$$

with a measurement $z \in \mathbb{R}^m$, where

$$z_k = Hx_k + v_k, \tag{2}$$

where

v_k is process noise with normal distribution $N(0, Q)$

w_k is measurement noise with normal distribution $N(0, R)$

Q and R are their covariance matrices

A, an $n \times n$ matrix, is the system state from previous to current time step, i.e., $k-1$th to kth time step

B, an $n \times l$ matrix, is the optional control input in the state x

H, an $m \times n$ matrix, is the state of measurement in z_k.

With the input of variables before step k, Kalman filter estimates the parameters at step k, known as a priori state estimate $\hat{x}_k^-$ which is corrected using measurement z_k in the next step, known as a posteriori state estimate $\hat{x}_k$. The errors in these two estimates is calculated using the following equations

$$\hat{e}_k^- \equiv x_k - \hat{x}_k^-, \tag{3}$$

and

$$\hat{e}_k \equiv x_k - \hat{x}_k. \tag{4}$$

The a priori and a posteriori estimate error covariances are then, respectively,

$$P_k^- = E\left[e_k^- e_k^{-\mathrm{T}}\right], \tag{5}$$

and

$$P_k = E\left[e_k e_k^{\mathrm{T}}\right]. \tag{6}$$

The aim of Kalman filter is to minimize this P_k. The following are the set of iterative equations to estimate $\hat{x}_k$ [13]

1. Initialization
   Set time step $k = 0$, $E[x_0] = \hat{x}_0$ and $E\left|\left[x_0 - \hat{x}_0\right]\hat{}2\right| = P_0$.
   Where $\hat{x}_0$ is the predicted density at time 0.
2. Extrapolation of a priori state estimate

$$\hat{x}_k^- = A_{k-1}\hat{x}_{k-1}. \tag{7}$$

A priori error covariance

$$P_k^- = A_{k-1}P_{k-1}A_{k-1} + Q_{k-1}. \tag{8}$$

3. Calculation of Kalman gain

$$K_k = P_k^-\left[P_k^- + R_K\right]^{-1}. \tag{9}$$

4. Update
   A posteriori state estimate

$$\hat{x}_k = \hat{x}_k^- + K_k\left[z_k - \hat{x}_k^-\right]. \tag{10}$$

A posteriori error covariance

$$P_k = [1 - K_k]P_k^-. \tag{11}$$

5. Iteration

   Set $k = k + 1$ and the process is repeated from step 2 until the last time step ends.

Model Development. As discussed in the previous section, Kalman filter technique is used to predict the density for three different locations in the study area. Density data per minute extracted from the video is used in the Kalman filtering. State variable is the density at every time step, represented as ρ_t. Density at previous time step is used to predict the density in the current time step.

$$\rho_t = A\rho_{t-1} + w_{t-1}. \tag{12}$$

Here, we have assumed A to be a constant [19]. Additionally, the pedestrian flow data was collected to be used as a measurement variable. However, due to difficulties in manual extraction of flow data, measurement variable was not used in this study.

Predicting Densities for Study Locations. Based on the Kalman filter process explained in the previous section, the density for all three locations is predicted. Figure 4 illustrates the graph between observed and predicted densities versus the consecutive sample points at five-minute time interval for all three locations. One-minute densities are averaged to five minutes so as reduce noise in the data. It is seen that the variations are quite high in all the locations.

4.3 Corroboration of Results

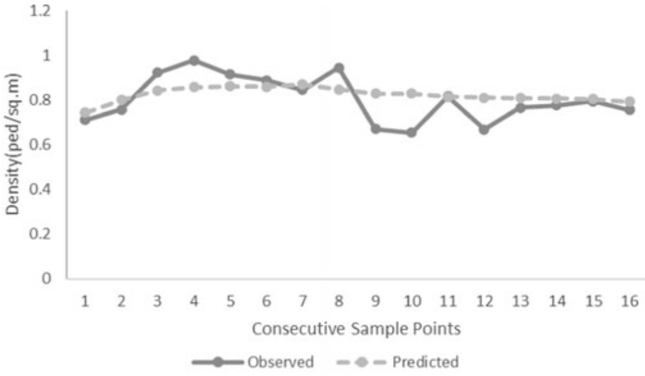
Mean absolute percentage error (MAPE) is used to check for accuracy in the estimation. This is calculated using the formula

$$\text{MAPE} = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{\text{Predicted density} - \text{Observed density}}{\text{Observed density}}\right| \times 100. \tag{13}$$

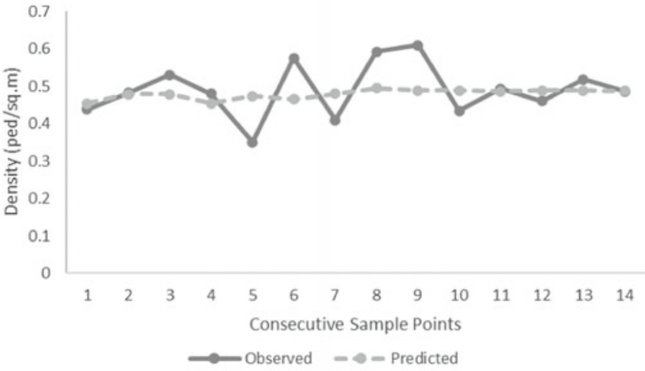
Error less than 10% is considered highly accurate, between 11 and 20% is good, 21 and 50% is acceptable and more than 50% is inaccurate [12]. Table 1 shows the MAPE observed for the study locations.

From Table 1, it is seen that the estimation accuracy is good for all the locations. However, the error percentage is quite high for Bombay Dharamshala due to the following reasons:

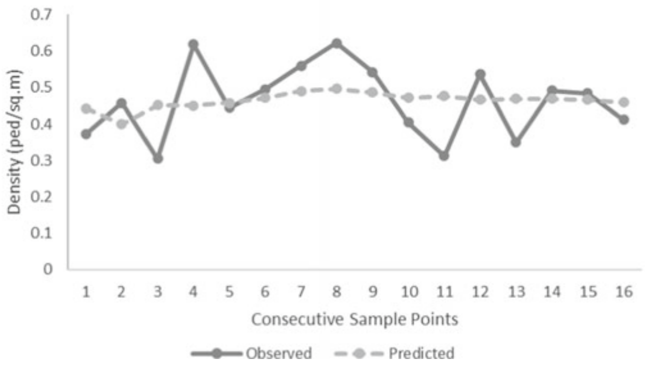
- Data was not available for the entire 7–9 a.m. as the flow was stopped due to a procession. Data for 7–7.40 a.m. was generated using random number generator. This could have led to poor estimation accuracy.
- Movement of vehicles was noticed in the video, which could have affected the density values.



(a) Chardham



(b) Mahakal



(c) Bombay Dharamshala

Fig. 4 Graphical representation of observed and predicted densities

Table 1 MAPE observed for the study locations

Location	MAPE in %
Chardham	13.1
Mahakal	10.26
Bombay Dharamshala	17.3

5 Conclusion

Predicting densities is imperative to identify possible crowd risk situations such as crowd crush and stampedes. Crowd density is an important parameter that can be used to make strategic decisions to manage large crowds. In this study, future densities are predicted from current densities using Kalman filtering technique for three locations within the study area and a MAPE of less than 20% is observed for all the locations, which is considered a good accuracy estimation. However, using other variables such as pedestrian flow, and average velocity to augment density prediction can capture the variations in density better. This will be explored further in our work.

Acknowledgements The work reported in this paper is part of the project titled The Kumbh Mela Experiment: Measuring and Understanding the Dynamics of Mankind's largest crowd, funded by the Ministry of Electronics and IT, Government of India (MITO-0105), Netherlands Organization for Scientific Research, NWO (Project no. 629.002.202), and Robert Bosch Center for Cyber Physical Systems, Indian Institute of Science, Bangalore (Grant No. RBCO001). The authors also express their gratitude toward Kumbh Mela administration and government of Madhya Pradesh, India for providing constant support and official permissions to carry out research work and establish Indo-Dutch collaborative research camp at Kumbh Mela 2016.

The authors also acknowledge the help rendered by Mr. Samar Singla, intern at IISc Bangalore, for this study.

References

1. Gayathri H, Aparna PM, Verma A (2017) A review of studies on understanding crowd dynamics in the context of crowd safety in mass religious gatherings. Int J Disaster Risk Reduct 25:82–91
2. Illiyas FT, Mani SK, Pradeepkumar AP, Mohan K (2000) Human stampedes during religious festivals: a comparative review of mass gathering emergencies in India. Int J Disaster Risk Reduct 5:10–18
3. Helbing D (1997) Verkehrsdynamik: Neue physikalische Modellierungskonzepte. Springer-Verlag, Berlin Heidelberg
4. Kormanová A (2013) A review on macroscopic pedestrian flow modelling. Acta Inform Pragensia 2:39–50
5. Helbing D, Farkas I, Vicsek T (2000) Simulating dynamical features of escape panic. Nature 407:487–490
6. Burstedde C, Klauck K, Schadschneider A, Zittartz J (2001) Simulation of pedestrian dynamics using a two-dimensional cellular automaton. Phys A 295:507–525
7. Schadschneider A (2002) Cellular automaton approach to pedestrian dynamics—theory. In: Schreckenberg M, Sharma SD (eds), pp 75–85

8. Helbing D, Johansson A, Al-Abideen HZ (2007) “Dynamics of crowd disasters: An empirical study”, Phys Rev E 75:046109
9. Oberhagemann D (2012) “Static and Dynamic Crowd Densities at Major Public Events”, Technical Report vfdb TB 13-01, German Fire Protection Association, Altenberge
10. Johansson A, Helbing D, Al-Abideen HZ, Al-Bosta S (2008) From crowd dynamics to crowd safety: a video-based analysis. Adv Complex Syst 11:497–527
11. Plaue M, Chen M, Bärwolff G, Schwandt H (2011) Trajectory extraction and density analysis of intersecting pedestrian flows from video recordings. In: Proceedings of the 2011 ISPRS conference on photogrammetric image analysis, Munich
12. Kumar SV (2017) Traffic flow prediction using Kalman filtering technique. Procedia Eng 187:582–587. https://doi.org/10.1016/j.proeng.2017.04.417
13. Deeshma M, Ashish V (2015) Travel time modeling for bus transport system in Bangalore city. Transp Lett 7:47–56. https://doi.org/10.1179/1942787514Y.0000000032
14. Martin H, Besada JA, Bernardos AM, Metola E, Casar JR (2014) Simplified pedestrian tracking filters with positioning and foot-mounted inertial sensors. Int J Distrib Sens Netw 10(9):850835
15. Bertozzi M, Broggi A, Fascioli A, Tibaldi A, Chapuis R, Chausse F (2004) Pedestrian localization and tracking system with Kalman filtering. In: IEEE intelligent vehicles symposium, pp 584–589
16. Mahakian RD (2018) Analyzing real-life pedestrian measurements using Kalman filters. ProQuest Dissertations Publishing. 10979263. California State University, Long Beach
17. Steffen B, Seyfried A (2009) Methods for measuring pedestrian density, flow, speed and direction with minimal scatter. Phys A Stat Mech Appl 389:1902–1910. https://doi.org/10.1016/j.physa.2009.12.015
18. Kalman RE (1960) A new approach to linear filtering and prediction problems. J Basic Eng 82(1):35–45
19. Welch G, Bishop G (1997) An introduction to the Kalman filter. Department of Computer Science, University of North Carolina

Calibration and Validation of VISSIM Parameters in Mixed Traffic

Anuj Kishor Budhkar and Avijit Maji

1 Introduction

Minor roads such as sub-arterials joining major urban corridors (hereafter referred as mainline) in the same direction constitute the simplest uncontrolled intersections. At a minor–major road grade separated intersection, the minor road traffic merges with major road through a ramp. There is a sudden increase in flow levels, and vehicles entering the mainline from the ramp need to find space (or gaps) in between vehicles on the mainline. This causes turbulence to the traffic stream. It is virtually impossible to avoid merging behavior of ramp traffic on major arterial roads in urban areas. Although roads can be substantially made signal-free, the merging behavior from other roads cause congestion leading to delays, and sometimes also require innovative techniques such as ramp metering for controlling traffic flow during merge. Series of merging sections may have large economic and environmental considerations on a city-wide or region-wide scale. Thus, their efficient control and management strategy for any city is important to reduce overall travel times.

Mixed traffic streams are different than conventional traffic, due to two peculiar phenomena (i) Weak lane discipline (random placement over entire width of the road) and (ii) Presence of heterogeneous traffic; or large number of vehicle types. The concepts of lane changing are not applicable in these streams, since the vehicles do not move in well-demarcated paths (i.e., lanes), and they veer frequently based

A. K. Budhkar
Department of Civil Engineering, Indian Institute of Engineering Science and Technology Shibpur, Howrah, West Bengal, India
e-mail: akbudhkar@civil.iiests.ac.in

A. Maji (✉)
Department of Civil Engineering, Indian Institute of Technology Bombay, Powai, Mumbai, Maharashtra, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_12

on adjacent vehicles' positions along with simultaneous acceleration or deceleration. Macroscopic traffic models have been devised for mid-block sections in these streams; however, less study has been conducted for vehicle merging behavior.

Merging behavior in mixed traffic conditions is unique, primarily because of the derived turbulence and resulting weak lane-disciplined traffic. Since drivers tend to minimize time and utilize any available effective space, it is observed that vehicles seldom yield to obstructing traffic, but rather slow down quickly at the conflict location. This results in larger queues and capacity drops. The drivers undergo a complex decision-making process consisting of optimum choice of speed, location of merge, acceleration/deceleration, and safe gaps between neighboring vehicles. Simulation of these merging sections is essential on a network-level basis to investigate overall delay and safety aspects of traffic. Calibration of parameters in a simulation model to represent field traffic is a primary step for this simulation.

A study is not conducted to estimate capacity of merging sections in mixed traffic and check its variation with traffic and roadway parameters. Such a study can notify immediate macroscopic effects of merging section on the highway and consequently at the network level. Simulation models such as VISSIM can effectively represent macroscopic traffic conditions for mixed traffic, thereby reducing time and error of data extraction in such traffic. This paper focuses on calibration and validation of VISSIM parameters to effectively represent merging traffic on an arterial road. Further, it also focuses on estimation of capacity, and change in speed and capacity in the uncongested regime, with introduction of ramp traffic.

2 Literature Review

This section describes previous work pertaining to merging process, merging models and usage of VISSIM in mixed streams. Most of the research work for merging behavior is conducted in the developed world, where traffic is more homogeneous in nature and vehicles travel in demarcated lanes.

The pioneering research in merging traffic was conducted by Drew et al. [1]. The report contributes largely to the study of merging process and includes the following studies: (i) Effect of traffic characteristics on merging process, (ii) Description of various merging processes possible, (iii) Effect of geometric parameters such as width of ramp, mainline and merging lane, type of merging on the merging behavior, (iv) Consideration of system such as interchange, land use, operational characteristics of ramp and freeway, etc., and (v) Various traffic control measures to enhance merging process. It holds the pioneering effort to model merging behavior by means of acceptance of minimum time-gap between vehicles in major road by the driver in minor road (termed as critical gap). Delay to the driver on minor road and decision to accept a critical gap are expressed as a function of freeway flow, critical gap, merging speed, relative speed and other factors.

2.1 The Merging Process

The merging process of a vehicle from ramp to the mainline is illustrated in NCHRP report 600 [2] and is adopted from Michael and Fazio [3]. Besides one vehicle from ramp accepting a gap in between two vehicles on the mainline (termed as conventional merging), other possible types of merging were studied by Drew et al. [1]. In the research, various freeway merges have been studied, such as optional, ideal or forced merging by the vehicle on minor roads. The merging process is challenging since it requires several tasks such as speed adjustment, steering and gap acceptance simultaneously. This process can be divided into a set of subtasks though the merging process is dynamic than mechanistic. It consists of initial steering and acceleration components on the ramp as the drivers transition from the ramp to acceleration lanes, gap acceptance or rejection and steering to join the mainline traffic stream. Drivers accelerate to have an unobstructed view of traffic on the mainline. Usually, drivers begin adequate gap search between vehicles on mainline after they reach the nose of entrance ramp. Drivers accept or reject gaps between vehicles and are in a dilemma zone whether to pass a particular vehicle on mainline or to follow it. At low traffic volume and sections with poor geometry, vehicles merge close to the nosing area. Whereas, if the traffic volume increases, vehicles follow zip-merging pattern, i.e., ramp and mainline vehicle merge one by one. If vehicles do not observe gaps, they may also choose to abort merging and stop at the end of merging zone. Cheng et al. [4] analyzed drivers' visual characteristics and found that drivers pay more attention and gaze more if traffic density on mainline increases. The turbulence is studied in detail in Beinum et al. [5].

However, in mixed traffic conditions where lane discipline is not followed, merging processes are unique. An explicit study of merging processes for ramps joining mainlines in mixed traffic is not conducted by researchers till date. However, merging process on T-junctions is studied in Kanagaraj et al. [6]. The authors have explored merging maneuvers on congested conditions on T-junctions in mixed traffic. Unique merging processes such as group and cover merging are illustrated, and the similar merging processes are also observed for ramps merging mainlines. Apart from normal and forced merging common in developed world, the researchers also studied group and vehicle cover merging phenomena. The merging depends on attainment of waiting time or queue length or both. Further, on-ramps joining mainlines, lateral vehicular interactions play an important role, since speeds are higher than those observed at intersections. Safe lateral gaps at various speeds may play a crucial role in evaluation of merging maneuvers. The lateral gap maintaining model developed in Budhkar and Maurya [7] with appropriate modifications maybe useful to simulate merging processes. Apart from modeling the driving behavior, the delay caused due to merging vehicles can be estimated based on stream speed reduction on merging sections. Macroscopic estimations such as these may be useful to determine merging capacities at a preliminary level.

2.2 Modeling the Merging Capacity

The vast study of merging behavior and maneuvering is aimed to improve the merging capacity apart from the safety. Merging capacity of streams are mathematically computed by Brewer et al. [8] using upstream service volumes and speeds on-ramps, as well as gap acceptance criteria.

Various researchers have evaluated merging capacities by different methodologies such as headway, gap acceptance, delay and speed reduction. Effect of merging distance on microscopic and macroscopic traffic parameters is also essential, as shown in Weng and Meng [9]. Some works are also carried to relate driver reaction (acceleration or deceleration) with the merging process, notably by Sarvi and Kuwahara [10]. The authors have calculated driver reactions at various zones, upstream, at merging and downstream of merging. Based on this, merging capacity is modeled and validated at several sections using an instrumented vehicle. A research work of this scale in mixed traffic conditions may be useful to model these phenomena for such traffic. Ramp metering as a means to increase capacity on mainline is estimated by Rudjanakanoknad [11]. Ramp metering is effective to improve flow and speed levels on both the roads, as well as reduce overall delay by avoiding gridlock-causing queues. Realistic theories regarding controlling ramp flow are developed.

Wu and Lemke [12] have developed a new model based on probability and queuing theory for determining levels of service of freeway merging and diverging sections. The authors have developed equations which determine service volumes for merging depending on capacity of ramps and upstream major road and volume to capacity ratio for given levels of service. Naturally, the volume to capacity ratio of a merge section is superposition of respective ratios of upstream mainline and ramp traffic volumes. The research work is also included in German highway capacity manual, 2011 [13].

The US Highway Capacity Manual (HCM) mentions that the base capacity of merging segments is the same as corresponding capacity of a basic segment, that is, a function of free-flow speed. The base capacities represent ideal assumed conditions, such as no presence of heavy vehicles, no heavy traffic on mainline or ramp. However, there is a high turbulence observed in case of mixed traffic, due to slow-moving vehicles such as three wheelers and trucks. Section capacity is related to maximum pre-breakdown flow, which varies from 1928 to 2330 vphpl for a merging section, as mentioned in HCM and Kondyli and Elefteriadou [14]. The HCM recommends reduced capacity values implemented by a capacity adjustment factor (CAF), which depends on turbulence due to vehicle mix and needs to be calibrated locally. The capacity estimates methodology assumes that vehicles on the ramp enter mainline only at two lanes immediately near to ramp. Accordingly, flow rates are lane-based, and highly depend on vehicle distribution factor across lanes. However, in mixed traffic, since space utilization is the priority, it is observed that ramp vehicles occupy empty spaces throughout the width of road, irrespective of proximity to the ramp. The capacity reduction in mixed traffic is evitable. This paper will also study the capacity reduction.

2.3 Usage of VISSIM to Simulate Mixed Traffic

The microsimulation model VISSIM is used by researchers widely to model mixed traffic too. Mathew and Radhakrishnan [15] have simulated traffic intersections in mixed traffic by considering delay as a validating variable. Manjunatha et al. [16] have further simulated intersections and validated them using this approach. Durrani et al. [17] have calibrated Wiedemann's car following model used as basis in VISSIM, for mixed traffic conditions. Studies related to capacity in mixed traffic were conducted by Meher et al. [18]. Capacity of expressways in mixed environment was also studied by Bains et al. [19]. Arkatkar [20] has studied road geometry effect on capacity of highways using VISSIM. A similar calibration procedure was used by Siddharth and Ramadurai [21]. Li et al. [22] have calibrated VISSIM for roundabouts by comparing deceleration rate and speed distribution parameters. Therefore, it is observed that most of research is based on either intersections or mid-block sections, but a study is not conducted to simulate merging sections.

2.4 Gaps in Literature Review Pertaining to Mixed Traffic

The review of literature yields that the merging behavior of drivers (their decision and reactions) especially at on-ramps have not been studied in detail in mixed traffic conditions. Optimum merging speed models based on geometry of section and given traffic composition are not developed for mixed traffic. Measures of effectiveness of a merging section may include time loss, delay to vehicles on major road and speed reduction.

Merging traffic sections can be evaluated by designers or planners to estimate traffic conditions at the conflict areas, expected delay to the traffic and thereby its capacity. For this condition, it needs to be simulated for experimental traffic and geometric variations. The next section presents calibration and validation of VISSIM parameters from extracted field data, whereas capacity estimation and comparison with existing literature is presented in Sect. 4.

3 Data Collection and Network Development in VISSIM

3.1 Field Data Collection

The field data were collected from an arterial road—Thane–Belapur road, located in Navi Mumbai city, Maharashtra, India. Two separate merging locations on this road, located about 5 km from each other were considered for calibration and validation. Indian code IRC 1061990 is used to convert mixed traffic into passenger car units (PCU). Per-minute traffic flow is recorded from these locations, in terms of passenger

Table 1 Extracted traffic parameters from field

Parameter		Cars	Two wheelers	Three wheelers	Trucks	Buses	LCV
Vehicle composition (%)	Location 1-mainline	38.6	21.6	6.1	19.1	3.1	11.5
	Location 1-ramp	53.1	16.8	3.9	14.7	2.7	8.8
	Location 2-mainline	44.1	30.9	11.6	5.5	2.5	5.4
	Location 2-ramp	44.7	37.8	7.8	3.3	0.7	5.7
Desired speed (km/h)	Mainline	54.96	46.47	38.03	38.54	44.37	42.5
		8.74	8.43	6.98	5.27	7.44	9.58
	Ramp	64.27	57.01	43.54	34.76	39.85	36.9
		9.89	7.75	4.66	6.47	3.35	7.18

car equivalents (PCEs) considering six vehicle classes mentioned in Table 1. Speeds of 25–30% of total vehicles are randomly considered for average speed calculation per minute for every section. The average speed is measured at three sections by noting in-time and out-time of vehicles over a 30-m trap length.

The first location (used for calibration) is located at Turbhe, consists of merging of traffic coming from Mumbai and Vashi with that of Thane–Belapur Road (southbound). Widths of mainline and ramp are 8.0 and 7.5 m, respectively, and merging length is 84 m. Peak flow rates of 5110 PCU/h and 4880 PCU/h are observed on mainline and ramp, respectively, over a five-hour duration. The second location (used for validation) is in Koparkhairane, which consists of merging of traffic coming from Sector 19 with that of Thane–Belapur Road (northbound). Widths of mainline and ramp are 7.6 and 7.0-m, respectively, and merging length is 68 m. Maximum flow rates of 3920 PCU/h and 2340 PCU/h are observed on mainline and ramp, respectively, over a three-hour duration. Although the sections are similar in geometry, but flow levels in Section 2 are much lower. The mainline and ramp at both sections are designed for two lanes of traffic, but movement of traffic indicates no following of lane discipline, and sometimes three virtual lanes were also observed. Table 1 mentions vehicle composition at both these locations and observed desired speeds for six vehicle classes. It is observed that desired speeds of ramp and mainline at both locations are similar.

On both traffic locations, the speed data were obtained by means of a traffic video recorded from a montage point away from merging area, during lean-peak hours on May 31 and June 1, 2018. The locations had ideal pavement characteristic, no horizontal or vertical curves and absence of gradient. Macroscopic traffic data (speed and flow) were extracted from each of the two locations, at three sections (i) Section 1—on mainline, 30-m before merging, (ii) Section 2—on ramp, 30-m before merging, and (iii) Section 3—on the mainline, immediately after merging section.

Fig. 1 Merging sections chosen for calibration and validation. Image copyright: wikimapia.org

Therefore, the ramp vehicles coming from Section 2 to merge the vehicles on the mainline at Section 3, whereas the vehicles on the mainline travel from Section 1 to Section 3, as explained in Fig. 1.

3.2 VISSIM Network Development

The first field location is replicated as a VISSIM network by using satellite imagery of the location. Three links—mainline, ramp and exit road (sections 1, 2 and 3, respectively) were constructed and joined by connectors. Since there is no lane discipline, each link is created using a single lane equal to width of actual road, and vehicles are allowed to overtake from both sides within the lane. Vehicle parameters calibrated from the field includes the following

1. Variation of acceleration and deceleration with speed for various vehicle types, adopted from Bokare [23]. Cars are assumed to consist of equal proportion of petrol and diesel models.
2. Desired speed distribution: Thirty free-flowing vehicles of each type were evaluated from each section for their desired speeds based on assumption that the vehicle is at its desired speed if there is no visible vehicle in its front and is not observed to accelerate. All vehicle types have normally distributed desired speed distribution profiles and the values are input in the VISSIM graphs accordingly.
3. Link behavior: The roads under consideration are urban arterial roads with limited access control. Therefore, urban (motorized) link behavior was allocated to all the links.
4. Driving behavior: Wiedemann-74 car-following parameters are used in VISSIM to represent traffic on urban arterial roads. Driving behavior adjustments include car following, lane changing and lateral distance keeping. For lateral distance keeping, vehicles are allowed to overtake from both sides in the same lane. Lateral distance-keeping parameters at standstill and 50 km/h speed are adopted using clearance-speed relationships evaluated in Budhkar and Maurya [7]. Look-ahead distance (minimum and maximum) and Wiedemann-74 car-following parameters (average standstill distance, additive and multiplicative part of safety distance) are unknown and need to be calibrated for this section. Minimum and maximum values are adopted from VISSIM's user manual.

The following network objects are used for traffic simulation.

1. Vehicle inputs were provided from sections 1 and 2 with similar flow and composition as observed in the field. Additional vehicle types (such as, motorized three wheelers and light commercial vehicles specific to the field) are imported as separate 3d vehicle models.
2. A conflict area is created at the merging zone. It is observed that there is no priority followed by traffic streams as they enter merging section. However, vehicles do interact with each other; therefore, the priority rule is left as 'undetermined.' VISSIM has options to modify driver behavior at merging sections using several conflict-zone attributes. Sensitivity analysis of all these conflict area parameters is conducted in the next section.
3. Vehicle travel time objects are placed before and after merging (total three), with a section length of 30-m each. They note the vehicle count, type, time of entry and speed of vehicles. Further, another vehicle travel time object is placed over a 500-m stretch on the mainline, such that the merging section lies midway.

4 Calibration and Validation

4.1 Identification of Sensitive Parameters

It is important to identify the sensitivity of parameters which are difficult to be obtained directly from the field and out of scope of this paper. For this purpose, the three attributes of Wiedemann-74 model, look-ahead and look-back minimum and maximum distances, and five of seven parameters of determination of driver behavior at conflict area were evaluated. They are mentioned in Table 2. Variation of the parameter ObjAdjLns or 'observe adjacent lanes while lane-changing' is not applicable for mixed conditions, since there is no lane concept, thereby lane-changing does not apply within the same stream. A parameter 'AddStopDist' or additional stop distance before the stop line at the merging as well as ramp is considered in VISSIM for appropriate yielding movement. It is observed that vehicles from either streams do not yield but rather scramble for space in the merging sections, thereby the parameter 'AddStopDist' is not considered and its value is given as 0. Each of considered parameters is varied by 20% of its default value (on positive as well as

Table 2 *p*-value of sensitivity analysis after 20% variation of VISSIM parameters

Parameter group	Name	Section A	Section B	Section C
Conflict area	Front-gap (FrontGapDef)	0.43	0.04	0.28
	Rear-gap (RearGapDef)	0.67	0.03	0.08
	Safe distance factor (SafDistFactDef)	0.24	0.01	0.12
	Anticipate routes (AnticipRout)	0.57	0.33	0.27
	Avoid blocking (AvoidBlock)	0.47	0.4	0.35
Car-following	Look ahead dist (maximum)	0.02	0.05	0.08
	Look ahead dist (minimum)	0.00	0.01	0.00
	Look back dist (maximum)	0.13	0.07	0.22
	Look back dist (minimum)	0.22	0.09	0.68
Car-following (Wiedenmann-74)	Average standstill distance	0.02	0.00	0.07
	Safety distance (additive)	0.11	0.06	0.01
	Safety distance (multiplicative)	0.03	0.23	0.01

negative side), and the resultant stream speed at each of the sections is observed. If there is a significant difference in stream speeds in any of the streams in either case (positive or negative), the parameter needs to be considered for calibration. The two samples include average stream speed every minute (Sample size 60 each), with default and changed parameter. *T*-test is applied for checking significant difference between the two samples, for each section. If a significant difference ($p < 0.05$ for t test) is observed between two samples of any section, the variable is considered for calibration. This process follows Manjunatha et al. [16]. Table 2 provides the *t*-test result of variation of each of these parameters for worse performer among positive or negative variation.

From Table 2, it is observed that sensitive parameters include all three Wiedemann-74 model attributes, look-ahead distance (max), look-ahead distance (min), FrontGapDef, RearGapDef and SafDistFactDef of conflict area parameters. Therefore, these 8 parameters are considered for calibration.

4.2 Calibration of VISSIM Parameters

The calibration of VISSIM parameters is performed using genetic algorithms (GA) run in a MATLAB code. A communication port (COM port) provides external modification of VISSIM attributes and allows stream to run for one-hour duration. Values of eight sensitive parameters (mentioned in previous section) are varied initially between upper and lower limits mentioned in Table 3, and input in VISSIM network designed in Sect. 4.3. The output file of VISSIM consisting of vehicle speeds, vehicle count and time of entry of each vehicle type is accessed by MATLAB to compare stream behaviors of field and VISSIM. Speed distribution for cars, trucks and two-wheelers at each section, and the speed-flow distribution at each section are used as the measures of effectiveness of VISSIM model. Only these three vehicle types were considered since other vehicle types constituted only a minor percentage of

Table 3 Variable limits and optimized values used for calibration

Variable name	Minimum value	Maximum value	Optimized value
Front-gap (m)	0	2	1.541
Rear-gap (m)	0	1	0.218
Safe distance factor	0.4	1.5	0.611
Look ahead distance (max) (m)	50	200	129.961
Look ahead distance (min) (m)	10	50	23.232
Average standstill distance (m)	1	4	2.352
Safety distance (additive) (m)	0.1	1.5	1.537
Safety distance (multiplicative) (m)	1	5	3.209

vehicular traffic and enough samples were not available for these types to generate a speed distribution.

The objective function of MATLAB is average of normalized root mean square error (NRMSE) between respective attributes of field and model. The RMSE is normalized by dividing by sum of square of residuals of expected value and subtracting from unity. Objective function is the average of (i) average NRMSE between speed distribution (histogram) of cars, trucks and two-wheelers at every section, thereby average of nine values; and (ii) average of NRMSE between speed-flow data-points of the field for each section, as per best fit curve and corresponding data points of the model (average of three values).

Each iteration in GA has population size 20, Elite count 4, crossover fraction 0.8 and uses the Gaussian mutation function. The parents are selected using tournament selection method (size 4), and the children are created using a scattered crossover function. The next generation is constrained to produce flow levels within the field's range. At the end of 23rd iteration, no further optimization is possible, and previous five iterations produced consistently the same value of optimization function. Therefore, the optimization process is terminated. Minimum and maximum values of eight parameters and the optimized values at the end of 23rd iteration are provided in Table 3. Figure 2 provides the variation of optimization function with each optimization plot. Similar result was obtained with changing the seed value in VISSIM simulation.

In this way, the traffic stream in VISSIM is calibrated to replicate the effect of car following and merging behaviors of the field section. Final optimized value of function is 0.61. Speed distribution of cars, two wheelers and trucks for the field and model, for each of the sections is presented in Appendix Figs. 4 and 5 for calibration and validation sections, respectively. Speed-flow models are presented in Appendix Figs. 6 and 7. The optimized parameters are used for further analysis in VISSIM.

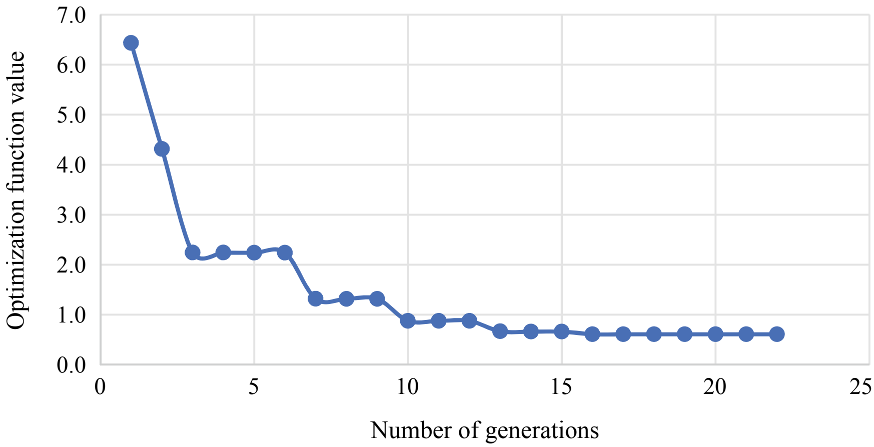


Fig. 2 Variation of optimization function with number of iterations

4.3 Validation of VISSIM Model

The second field section (30 min traffic data) is considered for validation. It is replicated in VISSIM similar to calibration procedure. All eight calibrated (optimized) parameters are input in this section and vehicles are input as per flow levels and composition in the field (Table 1). They are assigned the desired speed from Table 1 (normally distributed curve) and acceleration curves from literature [23]. The validation is conducted by two measures—(i) For macroscopic measure, speed-flow diagrams at each section are compared; (ii) For microscopic measures, speed-distribution at each section are compared for individual vehicle types, i.e., cars, two wheelers and three wheelers. Time headway distribution is a better microscopic measure; however, in mixed traffic conditions, since vehicles do not follow demarcated lanes, it is difficult to verify time headway between staggered leader and follower pairs. Further, an observer cannot ascertain if the two vehicles are interacting or not. Therefore, the authors have instead compared speed-distribution diagrams. Although obtained vehicle speed distributions fail to be similar for several vehicle types, using common distribution comparing tests (such as, K-S test, or chi square test), obtained normalized root mean square error (NRMSE) is closer to unity, indicating a good fitness level. T-test is applied between speed-flow diagrams of field and model, for every 1000 vph flow levels to determine similarity between field and VISSIM model at different flow levels. The results of both these tests are formulated in Table 4. From Table 4, it is observed that the model can represent speed distribution and speed-flow diagrams for most sections.

5 Effect of Ramp Traffic

This section attempts to predict capacity of the section (with help of speed-flow diagrams) with and without the effect of ramp traffic. For this purpose, the authors have generated two cases (i) generation of traffic streams at various flow levels on the mainline and ramp. This procedure is conducted for different widths of mainline (7, 10.5 and 14 m), and (ii) generation of traffic stream only on the mainline. Mainline speeds are measured over Section 3. For various combinations of flow levels and road widths, hourly flow levels and corresponding average speeds are plotted. A comparison is conducted over the uncongested regimes of speed-flow section (speeds above 25 km/h), to observe if there is any significant effect of ramp traffic on the speed-flow diagrams. If there is an effect, it would imply that the streams behave differently during merging. At various flow levels, t-test is applied to verify if the speed-flow diagrams generated are similar. For this purpose, datasets in each flow level are checked and is not grossly non-normal, for the applicability of t-test. Figure 3 shows the variation of speed-flow for uncongested regimes, for 2, 3 and 4 lane-width equivalents of mainline section. Table 5 provides the results of t-test conducted at various flow levels.

Table 4 Results of tests between field and VISSIM model (validation data)

Section	Speed distribution (normalized root mean square error) (closer to unity, better is the fit)			Speed-flow diagram (*p*-value of *T*-test), for flow levels (PCU/h) N.D. indicates no data					
	Cars	Three wheelers	Two wheelers	1000–2000	2000–3000	3000–4000	4000–5000	5000–6000	6000–7000
1	0.665	0.664	0.576	N.D.	0.79	0.05	0.81	N.D.	N.D.
2	0.699	0.756	0.757	0.46	0.06	0.02	N.D.	N.D.	N.D.
3	0.747	0.777	0.758	N.D.	N.D.	N.D.	0.91	0.01	0.08

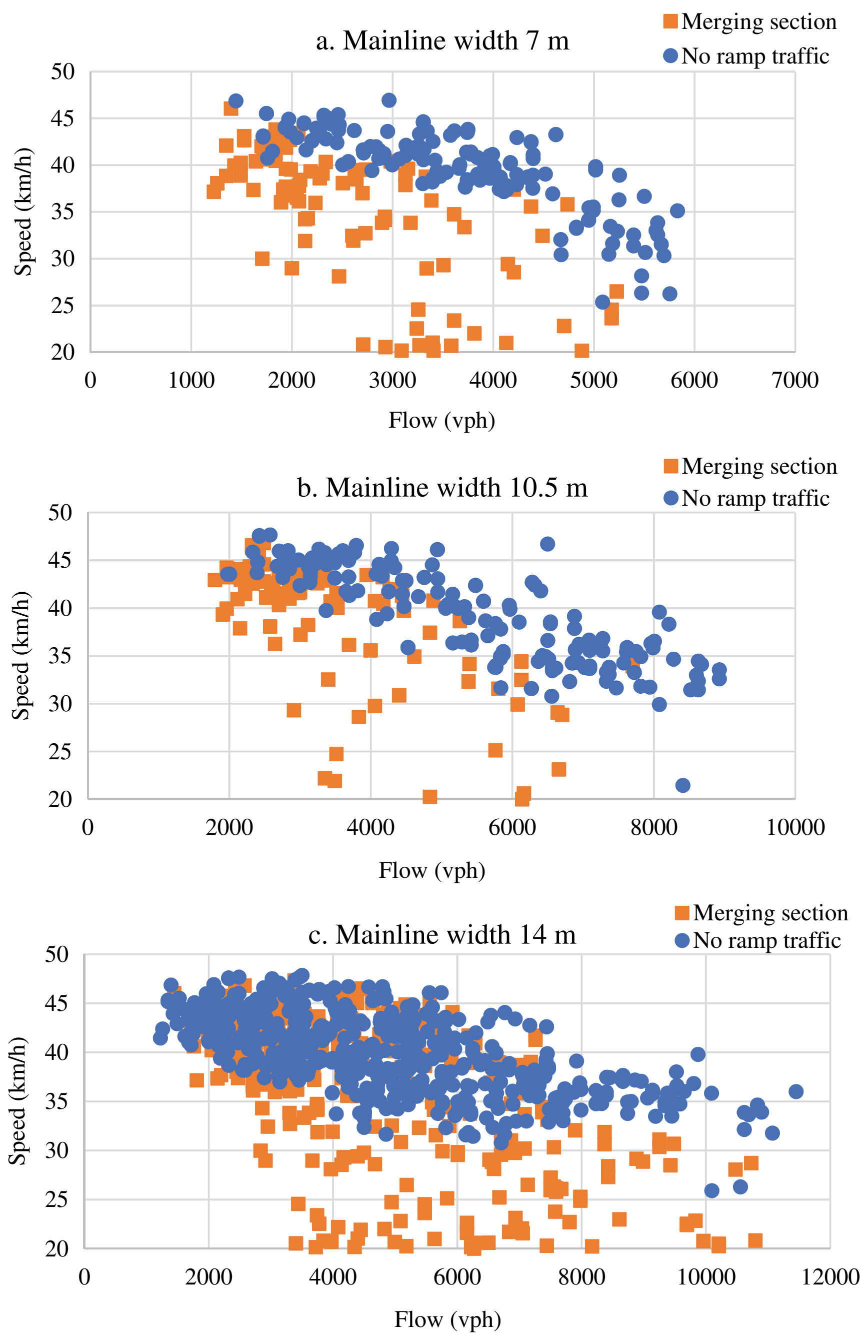


Fig. 3 Variation in uncongested regimes for different mainline road widths

Table 5 Capacity and *t*-test in uncongested regimes with and without interfering

Mainline width (m)	Capacity (without ramp traffic) (PCU/h)	Capacity (with ramp traffic) (PCU/h)	*t*-test on different flow levels (uncongested regime) (PCU/h)							
			1000–2000	2000–3000	3000–4000	4000–5000	5000–6000	6000–7000	7000–8000	8000–9000
7	5850	4660	0.01	0	0.03	0.06	N.D.	N.D.	N.D.	N.D.
10.5	8990	7440	0.01	0	0.18	0.01	0	0	N.D.	N.D.
14	11,070	10,210	N.D.	0.43	0.37	0.11	0.05	0.02	0.01	0

From the t-tests, a significant difference is not observed at lower flow levels, wherein vehicles travel close to their desired speeds. However, as flow levels increase, a drop in speed is observed statistically, for similar flow levels. There is a significant difference observed, at all widths of mainline. Maximum pre-breakdown flow (i.e., maximum flow observed on the uncongested regime) is considered as capacity of the section. There is a slight drop in capacity levels for all mainline widths, as observed from speed-flow diagrams in Fig. 3, and values in Table 5.

6 Conclusion

This paper presents a thorough calibration and validation procedure of simulating a merging section in mixed traffic conditions using VISSIM. Further, it has attempted to estimate capacity of merging section and compare it with capacity of mid-block section.

Two field locations on the same highway are chosen for calibration and validation each, and section details, traffic details, macroscopic and microscopic traffic parameters are extracted from upstream and downstream sections on mainline and ramp. Total eight parameters are found to be sensitive, which include car-following and conflict area parameters. These parameters are calibrated within their range so that macroscopic properties (speed-flow diagrams) and microscopic properties (speed distribution of various vehicle types) of traffic at each section are significantly similar. A GA-based optimization technique inputs improved solution at every iteration to the replicated VISSIM network using a COM interface. Obtained calibrated parameters are used in another network representing the second field section and validated macroscopically and microscopically.

The calibrated VISSIM network is used to obtain merging section capacity. It is observed that the capacity of merging section for mainline width of 7, 10.5 and 14 m are 4660, 7440 and 10,210 vph, respectively. Although there is a slight reduction in capacity (7–20%), there is a significant reduction in speed. Since merging sections are bottlenecks on access-controlled roads, this paper presents corridor speeds and capacity necessary to be considered while network designing. Effect of variation of geometry, traffic composition, etc., needs to be studied in detail as a future scope.

Appendix

The appendix consists of figures for calibration and validation of macroscopic and microscopic parameters mentioned in the paper.

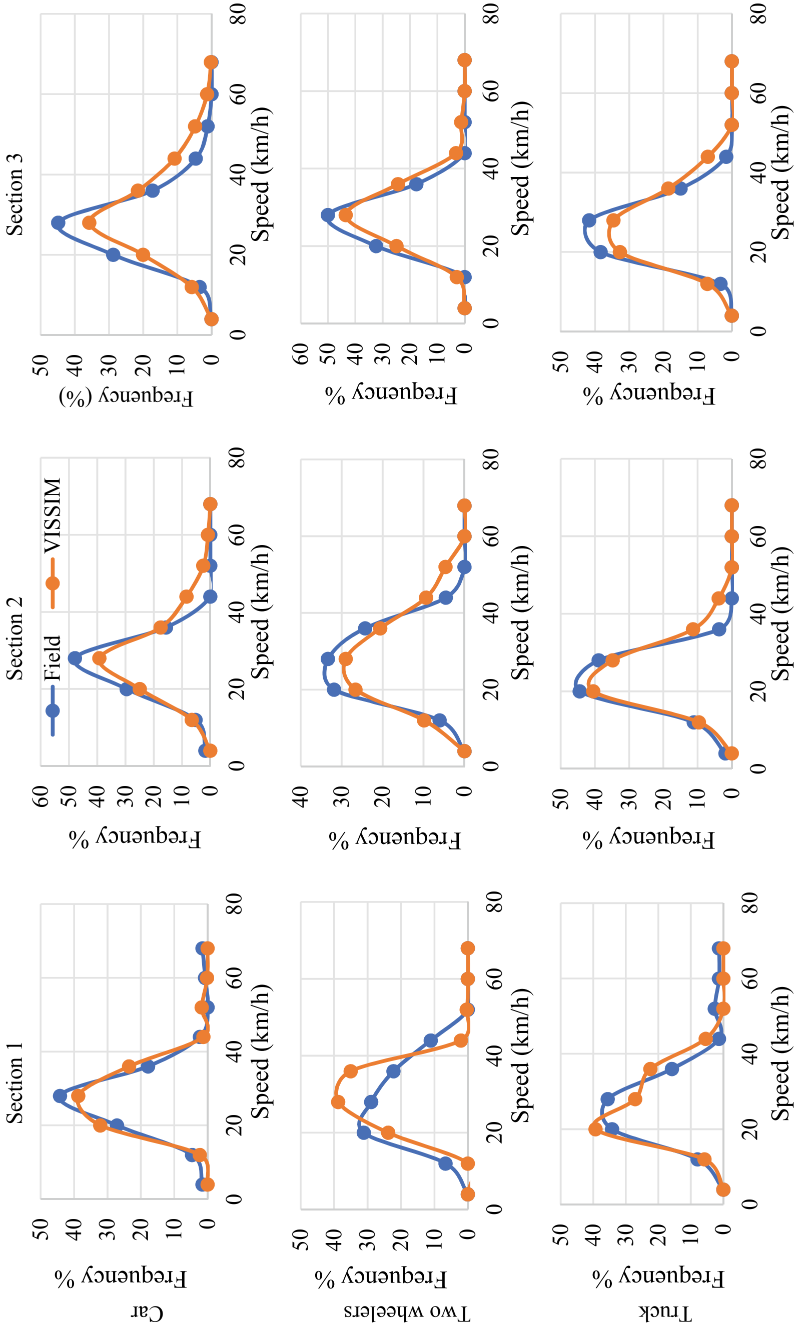


Fig. 4 Calibration section speed distribution diagram comparing field and VISSIM models

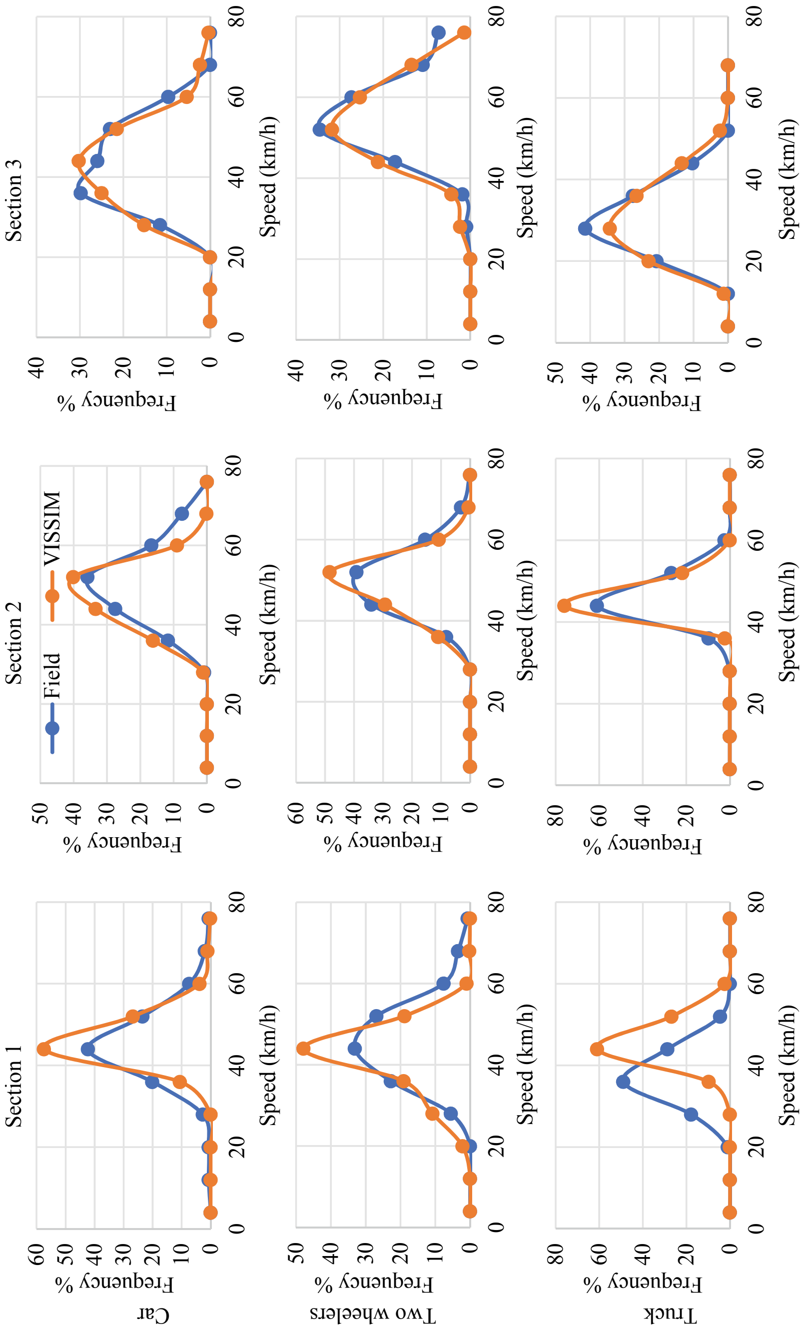


Fig. 5 Validation section speed distribution diagram comparing field and VISSIM models

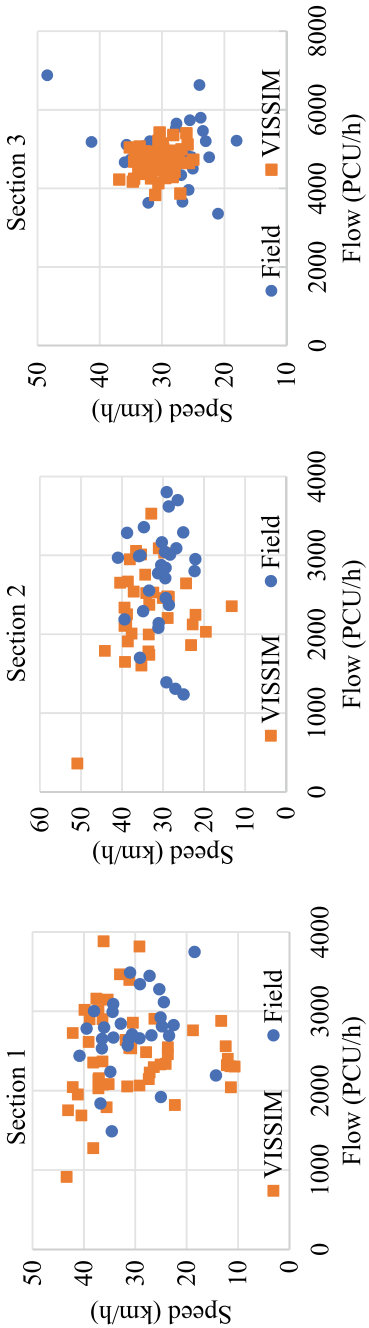


Fig. 6 Calibration section speed-flow diagrams comparing field and VISSIM models

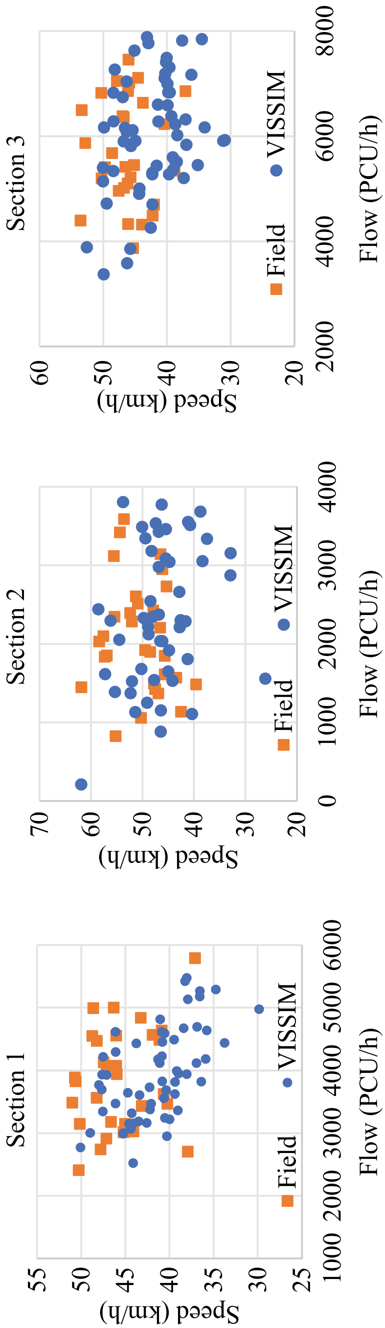


Fig. 7 Validation section speed-flow diagrams comparing field and VISSIM models

References

1 Drew D, LaMotte L, Wattleworth J, Buhr J (1967) Gap acceptance in the freeway merging process. Report 430-2. US Bureau of Public Roads
2 NCHRP report 600A human factor guidelines for road systems. Transportation Record Board, Washington, DC
3 Michael RM, Fazio J (1989) Driver behavior model of merging. Transp Res Rec 1213: 4–10
4 Cheng Y, Gao L, Zhao Y, Du F (2016) Drivers' visual characteristics when merging onto or exiting an urban expressway. PLoS ONE 11(9):e0162298. https://doi.org/10.1371/journal.pone.0162298
5 Beinum AV, Hovenga M, Knoop V, Farah H, Wegman F, Hoogendoorn S (2017) Macroscopic traffic flow changes around ramps. Transp A Transp Sci 14(7):598–614. https://doi.org/10.1080/23249935.2017.1415997
6 Kanagaraj V, Srinivasan K, Sivanandan R, Asaithambi G (2015) Study of unique merging behavior under mixed traffic conditions. Transp Res Part F 29:98–112
7 Budhkar AK, Maurya AK (2017) Characteristics of lateral vehicular interactions in heterogeneous traffic with weak lane discipline. J Mod Transp 25(2):74–89. https://doi.org/10.1007/s40534-0170130-1
8 Brewer KA, Buhr JH, Drew DR, Messer CJ (1969) Ramp capacity and service volume as related to freeway control. In: Proceedings of 48th annual meeting of transportation research board, Washington, DC
9 Weng J, Meng Q (2011) Modeling speed-flow relationship and merging behavior in work zone merging areas. Transp Res Part C 19:985–996
10 Sarvi M, Kuwahara M (2007) Microsimulation of freeway ramp merging processes under congested traffic conditions. IEEE Trans Intell Transp 8(3):470–479
11 Rudjanakanoknad J (2005) Increasing freeway merge capacity through onramp metering. PhD dissertation, Department of Civil and Environmental Engineering, UC Berkeley, CA
12 Wu N, Lemke K (2011) A new model for level of service of freeway merge, diverge, and weaving segments. In: 6th international symposium on highway capacity and quality of service, Stockholm
13 Highway Capacity Manual (2011) A guide for multimodal mobility analysis, volume 2: uninterrupted flow, 6th edn. Transportation Research Board, Washington, DC
14 Kondyli A, Elefteriadou L (2012) Driver behavior at freeway-ramp merging areas based on instrumented vehicle observations. Transp Lett 4(3):129–142. https://doi.org/10.3328/TL.2012.04.03.129-141
15 Mathew TV, Radhakrishnan P (2010) Calibration of microsimulation models for non-lane-based heterogeneous traffic at signalized intersections. J Urban Plann Dev ASCE 136(59)
16 Manjunatha P, Vortisch P, Mathew TV (2013) Methodology for calibration of VISSIM in mixed traffic. In: 92nd annual meeting of transportation research board, Washington, DC, 13–17 Jan 2013
17 Durrani U, Lee C, Maoh HF (2016) Calibrating the Wiedemann's vehicle-following model using mixed vehicle-pair interactions. Transp Res Part C Emerging Technol 67: 227–242. https://doi.org/10.1016/j.trc.2016.02.012
18 Mehar A, Chandra S, Velmurugan S (2014) Highway capacity through VISSIM calibrated for mixed traffic conditions. KSCE journal of Civil Engineering, 18(2):639–645
19 Bains MS, Balaji P, Arkatkar SS (2012) Modeling of traffic flow on Indian expressways using simulation technique. Procedia Soc Behav Sci
20 Arkatkar SS (2011) Effect of intercity road geometry on capacity under heterogeneous traffic conditions using microscopic simulation technique. Int J Earth Sci Eng 4(6):375–380
21 Siddharth SM, Ramadurai G (2013) Calibration of VISSIM for Indian heterogeneous traffic conditions. Procedia Soc Behav Sci 104:380–389

22 Li Z, DeAmico M et al (2013) Calibration of VISSIM roundabout model: a critical gap and follow-up headway approach. In: Transportation research board 92nd annual meeting, vol 131176, Washington, DC
23 Bokare PS (2013) Modeling of acceleration and deceleration behavior of vehicles. PhD dissertation, Indian Institute of Technology, Guwahati

Lateral Gap Maintaining Behaviour of Vehicles from Road Median

Geetimukta Mahapatra and Akhilesh Kumar Maurya

1 Introduction

In homogeneous traffic conditions, wherein, the lane discipline is strictly followed, the linear clearance maintained by the vehicles with respect to the vehicles in front may be sufficient to describe the vehicle manoeuvering process. In mixed traffic, the lane discipline is absent, and the vehicles travel abreast due to the variations in their size and non-standard widths; so it becomes essential to consider the clearances in the lateral direction in addition to that in linear direction for defining the flow logic. Road features/elements (like road edges conditions, divider conditions, etc.) have also an impact on the lateral and longitudinal placement of vehicles, based on whether drivers perceive a roadside element as an obstacle or not. Vehicles will move away (or maintains certain clearance or shy distance) from an object/road element which they perceive as an obstacle. A proper understanding of such clearance maintaining behaviour is necessary to define the vehicular interaction with the road features. Development of micro-simulation model requires a comprehensive understanding of microscopic and macroscopic parameters. Models of linear and lateral clearances are useful for finding influence areas, determining PCUs, safety analysis and development of geometrical design standards, etc.

In this present study, the impact of median type (width and height) on the lateral gap (shy distance) maintaining behaviour of vehicles is studied using a sensor based assembly. For lateral clearance measurement, only the vehicles travelling in median lanes (i.e. adjacent to the road median) is taken into account. The data is collected from the straight mid-block sections of different roads (four-lane divided highways) with different median facilities at free flow speed condition.

G. Mahapatra (✉) · A. K. Maurya
Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati, Assam, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_13

2 Background

In a country like India, the way a vehicle moves is pretty challenging to apprehend. The positioning of the vehicle has been studied for over the past few decades by various researchers globally, and some parameters have been found to be directly affecting it. Amongst them, some have been described in the following sections.

1. Impact of median or shoulder divider of the road on the lateral placement of vehicles.
2. Impact of traffic and stream parameters on the lateral placement of vehicles.

2.1 Impact of Median Divider/Shoulder on Lateral Placement of Vehicles

Function of the median is to separate opposing lanes to avoid head-on-collision. Similarly, the shoulder is required to prevent the road departure crashes for the out of control vehicles. Indian road design practises do not differentiate between the types of medians. Amongst highway engineers, there is quite a variance of opinion regarding the effectiveness of the different types of dividers. Similarly, edge line and centre line (in the form of a thick line or two lines) play a crucial role in the vehicle's position. Sagberg [27] observed a shift in the lateral position of about 30–38 cm for the road with new painting and 12–20 cm for the road with rumble strips. Zhou et al. [40] observed that average lateral placement tends to decrease with the increase in width of the inside shoulder. Similarly, the edge line, the outermost line in the roadway tends to affect the vision of the driver who is propelling along the highway. Woods [36] observed that vehicles tend to move away from the centre line and towards the edge when there is more lateral clearance between edge or traffic lane and the bridge curb. Middleton [22] stated that drivers had a tendency to drive closer to the centre of the road in the absence of edge line, irrespective of speed, vehicle type and at all levels of traffic volume. Armour [2] observed that the edge line had no effect on the lateral placement of the vehicles, however, shoulder type and its width have an effect on the lateral position of vehicles. Several studies have found that wider shoulders give the drivers a sense of security and space for correcting errors [10, 11, 13, 26, 30, 39]. However, narrow shoulder led drivers to steer away from the shoulder and drive closer to the centre of the road, thus increasing the likelihood of a head-on-collision in undivided roads [3, 6].

Shorpe [28] studied on six different types of median dividers in relation to their effect on the positioning of free moving passenger cars. In the past, various studies also indicated that the types of median barriers and median width have shown counterbalancing potential to improve safety of the multilane roadways [1, 14, 15, 38]. Elvik [8] suggests that the presence of median guardrail has the potential to increase crashes. Hong and Lee [16] found that the existence of medians does not only function as safety measures as prescribed in road designing but also susceptible to generate adverse effects. Another study by FHWA [9] also indicated that there is a relative effect of different median widths on accident types and accident severity. Siregar [29] stated that drivers behave differently for different types of medians. He found that drivers acknowledge the physical medians as hazards but do not consider the potential hazard of opposing traffic despite the least protective function of non-physical medians like line medians. The vertical dimension of medians determines the choice of safe distance, passenger cars and motorcycles tend to provide a bigger gap that trucks and buses [24, 35].

2.2 Impact of Operating Speed on Lateral Placement of Vehicles

The vehicle dynamic parameter, like operating speed, has a major impact on the lateral placement of vehicles. Van Driel et al. [34] concluded that traffic volume like opposing vehicles or vehicles in the same directions and the circumstances like day or night have no effect on speed and lateral position of vehicles. However, Dey et al. [7] observed that slow-moving vehicles move at the edge, whilst fast ones move towards the centre of the road. Bunker and Parajuli [5] observed that the size of the vehicle was inversely proportional to the spread of vehicles' lateral positioning. Further, Rasanen [25] revealed that due oncoming vehicles there was an overall shift of 15–20 cm towards the edge of the road, i.e. vehicles tend to move away from the centre line in the presence of opposing traffic. Hallmark [12] used an odds ratio of an adjacent lane to describe the relation between speed and lateral placement of vehicles. Balaji et al. [4] deduced that there is linear relation of speed and lateral placement for 3-wheelers, heavy and slow-moving vehicles, whereas quadratic relationship existed in case of 2-wheelers and cars. Stodart and Donnell [31] stated that there was no appreciable endogenic relationship observed between speed and lateral position differentials. Curve direction affects the lateral position of the vehicle, whereas adjacent curve radii affect vehicular speed. Authors like Middleton [22] and Mahapatra and Maurya [20] revealed that higher the speed of the vehicle, closer it drove to the centre of the road. He also found a linear relationship between lateral position and speed of vehicles. The relations clearly state that vehicles with higher speed will have greater or positive lateral placement values suggesting that a vehicle with higher speed tends to travel along the centre of the road, irrespective of the edge line condition.

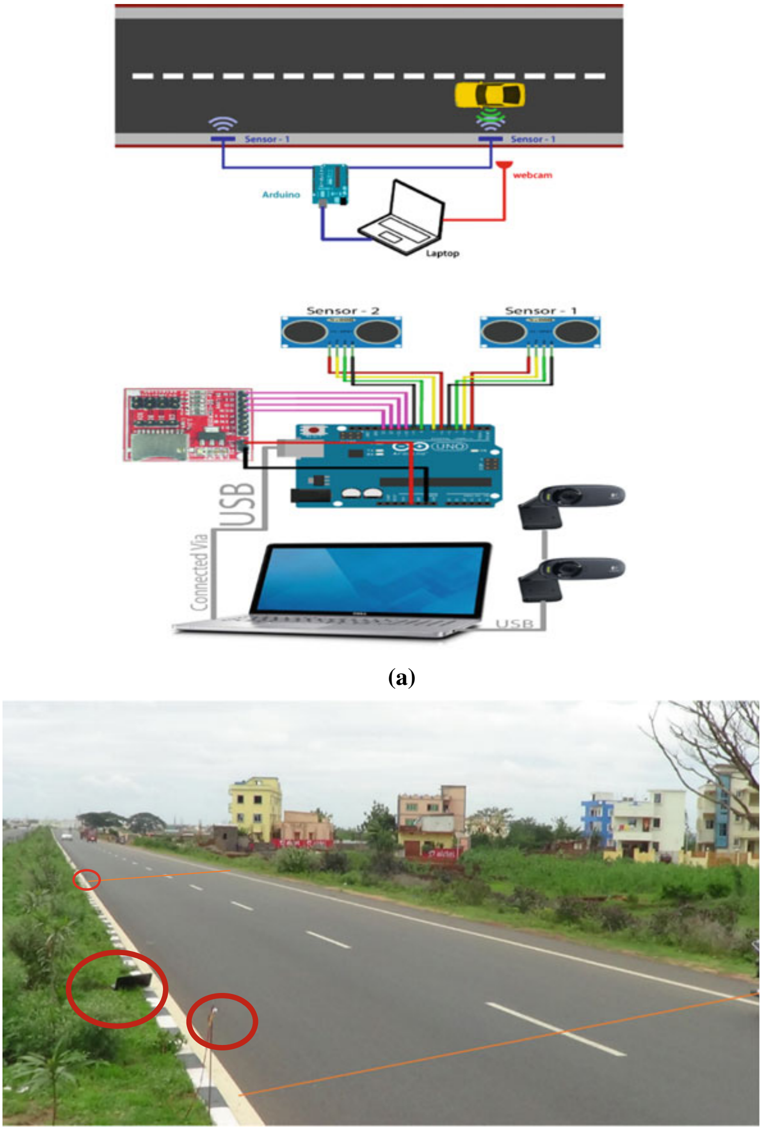
3 Motivation and Objective

The review of literature on the vehicles' lateral characteristics reveals that median and shoulder condition and their type have a significant impact on the lateral positioning of vehicles. Including the traffic parameters, the roadside features like road boundaries also affect the lateral behaviour of the traffic. Also, the relationship between speed and lateral position is important in understanding the Indian traffic behaviour. Numerous studies had been conducted using different methods and technologies to find the lateral position of the vehicle on the road. Most of the studies were limited to analysis of the lateral movement of the vehicle on curves. Very few studies have been carried out to see the impact of different type of medians on lateral positioning of vehicles. Studies related to the vehicle's lateral characteristics on a straight road with mixed and no-lane disciplined traffic were found important to understand the vehicle's lateral movement behaviour in a traffic stream for the realistic traffic simulation model. Amongst the few studies carried out in India were based on limited data and manual data analysis. Therefore, the accuracy of such results can always be challenged. Hence in our present study, the impact of road median type (width and height) on the lateral gap maintaining the behaviour of different vehicles travelling on the adjacent lane are studied.

4 Study Methodology and Data Collection

The researchers faced the problem of measuring the position of a vehicle on the road, whilst driving. Although there are numerous ways to measure the longitudinal distance the driven vehicle, the lateral position of the vehicle has been proven to be more difficult to measure [12, 32, 33, 37]. In India, there are very few studies that have been conducted on the lateral behaviour of the traffic. Dey et al. [7] studied on the lateral placement of vehicles under mixed traffic conditions. Again Balaji et al. [4], conducted a study on lateral placement and speed of vehicles on two-lane roads following a similar method as Dey et al. [7]. Data were collected by dividing the lane width into sections of 25 cm each with self-adhesive cloth tape, and these were numbered seriatim from pavement edge to the centre of the road. The placement of left rear wheel of a vehicle crossing the section was recorded by video recording and analyzed manually to get the vehicles' lateral position. Accuracy of such approach of vehicle's lateral position measurement is always poor and questionable. In our present study, a sensor based assembly has been developed to directly measure the lateral gap of each vehicle passing near the road edge. This developed assembly helps in measuring the, (i) lateral clearance maintained by vehicles from road elements, and (ii) speed of the vehicles. The operation is semi-automotive with very little human intervention [21].

Two ultrasonic sensors are placed along the side of the road at a known distance. As a vehicle passes in front of sensor 1, the sensor 1 gets activated and start receiving the reflected waves from the vehicle. Based on the travel time of emitted wave and reflected wave, lateral clearance of the vehicle from the sensor is calculated. A similar process is repeated in sensor-2. As sensors are places apart with a known distance, the travel time of the vehicle for the distance is determined from the time difference of actuation of both the sensors. Based on the measured travel time speed of the vehicle is calculated. Figure 1 depict the details of the sensor assembly and one of the data collection site after installing the instrument.



(a)

(b)

Fig. 1 Details of the sensor assembly and the data collection site **a** complete layout of sensor assembly with two cameras. **b** A selected site for data collection with sensors placed at the median of the road

The sensors work on sonar principle. They send out a burst of ultrasonic waves and wait for echoes to return to them. If an echo is received, the time lag (in microseconds) between the transmitted signal and received echo signal is obtained. This time lag value can be used to calculate the distance of the reflecting surface from the sensor using the following expression;

$$\text{Distance (in cm)} = \text{Microseconds}/58$$

where Speed of sound in air = 340 m/s.

Thus, 1 cm corresponds to approximately 29 μs. Since the ultrasonic signal travels back and forth, it must be halved to receive actual distance. Thus microseconds divided by 58 ($= 29 \times 2$) gives the distance of the object from the sensor in centimetre.

The Arduino Uno is a microcontroller board based on the ATmega328. It has 14 digital input/output pins (of which 6 can be used as PWM outputs), 6 analogue inputs, a 16 MHz ceramic resonator, a USB connection, a power jack, an ICSP header, and a reset button. A micro-SD card is used with the Arduino UNO board to record the data. Two web cameras have been connected to the laptop for taking the snapshots. Whenever a vehicle passes in front of the sensor 1, the first sensor gets activated and starts recording the distance data with respect to the time stamp. Then as soon as the sensor activates, the camera got activated and it starts taking the snapshot of the section. Whenever any non-zero values are coming in sensor, the camera gets activated. This instruction is given through a code written in MATLAB programme. The snapshots of each camera are being stored on the laptop with their time stamp. A similar procedure is followed for sensor-2 also. The complete working procedure of this setup is shown in the flow chart in Fig. 2.

This sensor assembly solves the problem of manual data collection from videos, which is a time-consuming process. The vehicle type identification is easier with this model. The camera is properly synchronized with the sensor. Hence, the error due to manual data synchronization can be reduced through this model. To counter the problem of limited battery backup, extra battery packs can be kept for the laptop model being used for data acquisition. Another solution is maybe keeping a 12-V lead-acid battery along with portable 200 W inverter. These items can be carried in a small vehicle to the data collection site.

To avoid unwanted data, a limit has been fixed through the programme for the number of data. The sensor will take only 5 data for each vehicle passing in front of it. Hence the corresponding camera will also take five number of snapshots corresponding to each sensor reading. Then by comparing the sensor reading and snapshot, it will be easy to identify the vehicle type.

The sensor assembly has been so designed that it measures the lateral clearances with an accuracy of ±5 cm. Thus, the developed assembly aids in measuring the following;

1. The lateral clearance maintained by vehicles from road elements.
2. Speed of different vehicles in the traffic mix.

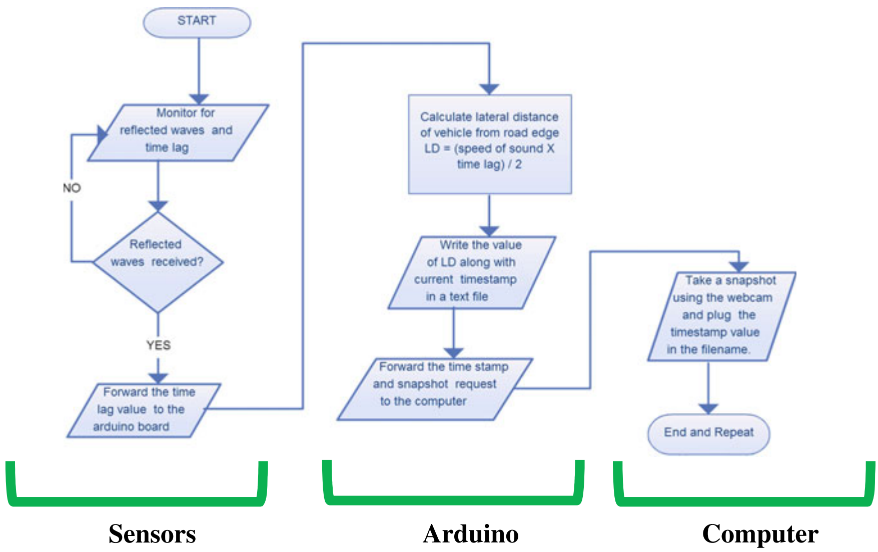


Fig. 2 Presents the complete working procedure of the sensor assembly

Since the sensor time stamp is used for speed calculation, there might be an error due to the time lag between consecutive vehicle detections. The data collection frequency of the sensors is 30–35 Hz. Therefore, the time gap between readings varies between 0.0286 and 0.033 s. The maximum error will correspond to the maximum time lag, i.e. 0.033 s. For reliability condition, let maximum time lag of 0.035 s. If the distance between the two sensors be D_0 (m) and the speed of the vehicle be v_0 (m/s), the vehicle should take time t_0 to cross this distance at the original speed. Hence, the time to cross the sensor, $t_0 = D_0/v_0$. Considering the error (δt) as 0.035 s, recorded speed will be $v' = D_0/(t_0 - \delta t)$.

Hence, the percentage of error (%) $= (v' - v_0) * 100/v_0$.

Thus, the error in measured speed will depend upon;

- Sensor distance (D_0)
- True speed (V_0) of the passing vehicle.

From Fig. 3, it can be observed that as the distance between sensors increases the error in the speed of vehicles decreases. For a distance of 20 m between the sensors, then 5% error is detected for a vehicle travelling at 100 km/h. Hence, in the present study, the 20 m distance has been used between the sensors, whilst collecting the data using the sensor assembly.

The sensor assembly is placed along the edge of the road with all the proper setup as mentioned above. The sensors are fixed at such a height where it can detect all type of vehicles (motorized two-wheelers, three-wheelers, car, bus and truck, etc.) in the mixed traffic stream. The height of placement of the sensor is based on the height of different vehicles, i.e. the seat height of two-wheelers and the chassis height of

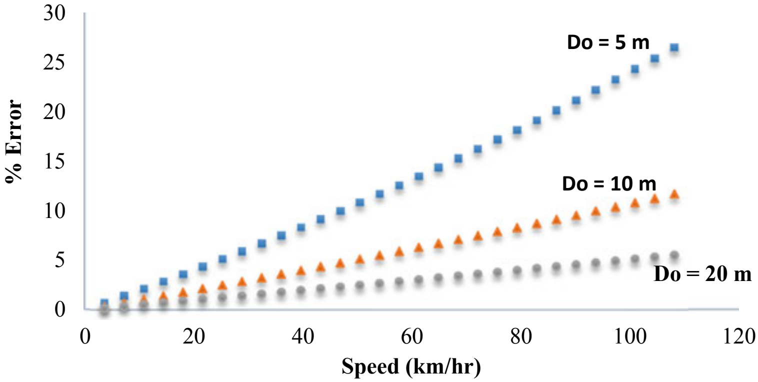


Fig. 3 Maximum percentage error which may occur in speed measurement using the proposed sensor assembly

different type of cars, buses and trucks. A height of 0.8 m from the carriage way is fixed for the sensor placement at the road boundary.

This assembly helps in large-scale data collection of the lateral clearances maintained by the vehicles from the road elements. Data was collected on straight mid-block sections of roads with different median facilities. The impact of the different lane positions (such as the Median Lane, ML and Shoulder Lane, SL) has been analysed for all sections on average travel speed. The lateral clearance of each vehicle type from the road edge of SL/ML is calculated.

Data have been collected from different cities on dry and sunny weather condition, during morning hours (9 a.m.–12 p.m., i.e. 4 h). The road surfaces were in good condition with proper lane markings. The data is collected from different median types of four-lane divided roads are divided into different groups with respect to their width and height of medians. The sites for data collection are selected according to their flow level (Similar range of flow per lane) (refer Table 1).

Speed, flow and vehicle composition of ML and SL of all highways are calculated from the field data to know the traffic condition of all three types of roads. Lane specific average speed for each vehicle type on different roads are computed. Statistical analysis is done to see the difference in the lane specific traffic characteristics (i.e. speed, time headway) of different roads. A MATLAB programme is used to analyze the data. The speed and lateral clearance of each passing by vehicle are calculated with respect to the time and vehicle type. Hence, the speed and lateral clearance of each vehicle with respect to its vehicle type are analyzed for ML (Median Lane) and SL (Shoulder Lane) of all the sections (Fig. 4).

It is observed that in the case of four-lane divided roads, median sided lane contains most of the cars and trucks. Hence, the models are developed only for cars and trucks. To see the minimum gap maintained by different vehicles from different medians, the lower 5% lateral gap data for different speed groups are extracted and modelled

Table 1 Site details for the lateral gap analysis

4 lane divided highway		Height	
	Width (m)	≤0.3 m	>0.3, ≤0.5 m
	≤0.8	No data available	NH-217 Flow = 310 PCU/h/Lane
	0.8–2.0	NH-37 Flow = 434 PCU/h/Lane	(Inner Ring Road) Delhi Flow = 1215 PCU/h/Lane
	2.0–3.2	NH-5 (BAM) Flow = 312 PCU/h/Lane	Bhubaneswar Flow = 473 PCU/h/Lane
	3.2–4.4	NH-203 Flow = 312 PCU/h/Lane	No data available
	≥4.4	NH-43 Flow = 324 PCU/h/Lane	Bhubaneswar Flow = 468 PCU/h/Lane

for cars and trucks. This indicates that 95% of vehicles maintain a larger gap than this.

5 Model Development and Validation

The regression models are fitted for a lateral gap of cars and trucks using each vehicle's operating speed and size of median as variables. The regression models for the lateral gap are fitted by taking two dummy variables (i.e. width and height of median) and operating speed of the vehicle as an independent variable. The simplest case of dummy coding is when the categorical variables have 'n' levels and are converted to '$n-1$' dichotomous variable. The size of the median are divided into different groups according to the design guidelines of India [17–19]. The width of the medians is divided into four groups. The dummy coding of the categorical variables, width and height of medians are presented in Table 2.

Following Eqs. 1 and 2 are the fitted models for minimum lateral gap maintaining the behaviour of cars and trucks, respectively.

$$\begin{aligned}\text{Gap}_{\text{cars}}\,(\text{m}) = 0.811 + (0.030 * V) - (0.204 * W_1) - (0.326 * W_2)\\ - (0.164 * W_3) + (0.132 * H)\end{aligned} \tag{1}$$

$$\begin{aligned}\text{Gap}_{\text{Trucks}}\,(\text{m}) = 0.925 + (0.013 * V) + (0.269 * W_1)\\ - (0.426 * W_2) - (0.032 * H)\end{aligned} \tag{2}$$

where Gap = Lateral gap maintained from the edge of the median (m), V = Operating speed of the vehicle, W_1, W_2 and W_3 are width groups of median according to design guidelines and H = Height group of median according to design guidelines. The residual plot and normal probability plots indicate a better fit of the predicted

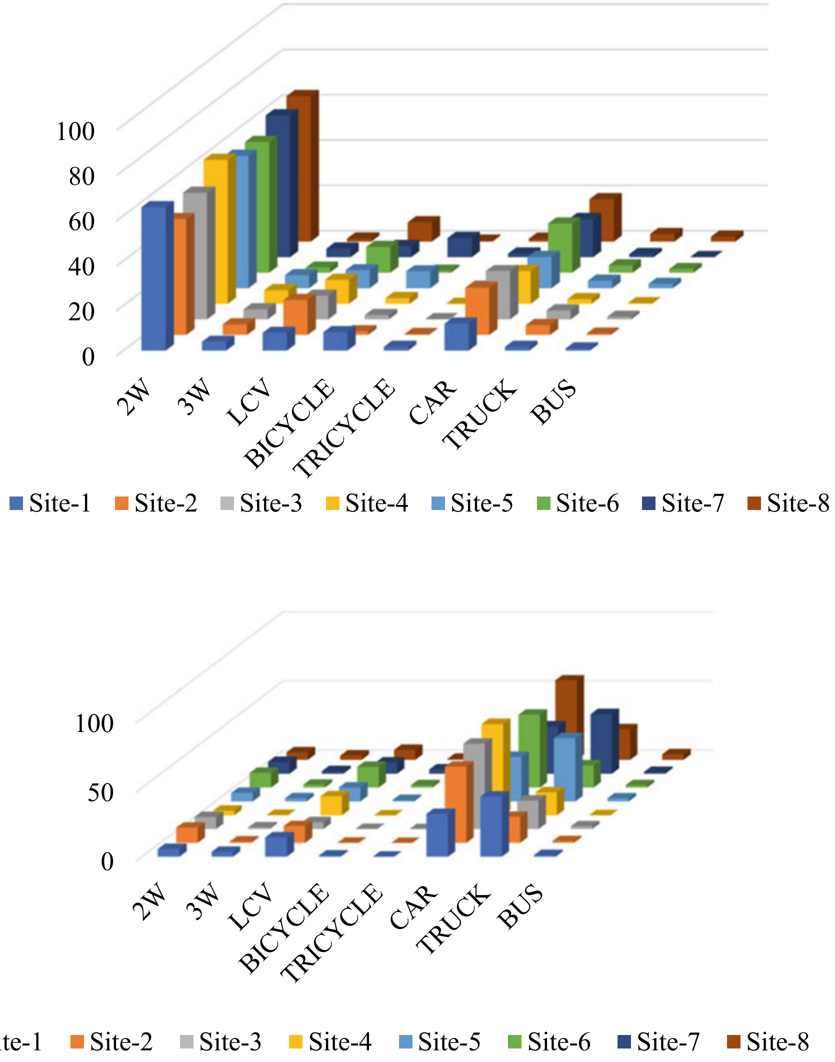


Fig. 4 Traffic composition of all different sites **a** shoulder Lane_SL, **b** median Lane_ML

model. Figure 5 presents the observed and predicted data of cars and trucks with their residual plot and the normal probability plots.

The models are validated with a new set of data collected from a different site. The MAPE values are obtained (23.9 for Cars and 22.32 for trucks) for the validation site also. From the MAPE values, it can be concluded a better fit of the model to the data as per the Lewis Scale of Measurement [23]. Figure 6 presents the details of the observed and predicted minimum lateral gap maintained by cars and trucks from the median.

Table 2 Dummy coding for different groups of height and width of medians

Width of median	Categories	$W1$	$W2$	$W3$
i. $W < 1.2$ m	1	1	0	0
ii. $W \geq 1.2$ m and <2.5 m	2	0	1	0
iii. $W \geq 2.5$ m and ≤ 4.5 m	3	0	0	1
iv. $W > 4.5$ m	4	0	0	0
Height: (barrier kerb)	*Categories*	H		
i. $H \geq 200$ mm and ≤ 350 mm	1	0		
ii. $H > 350$ mm and ≤ 800 mm	2	1		

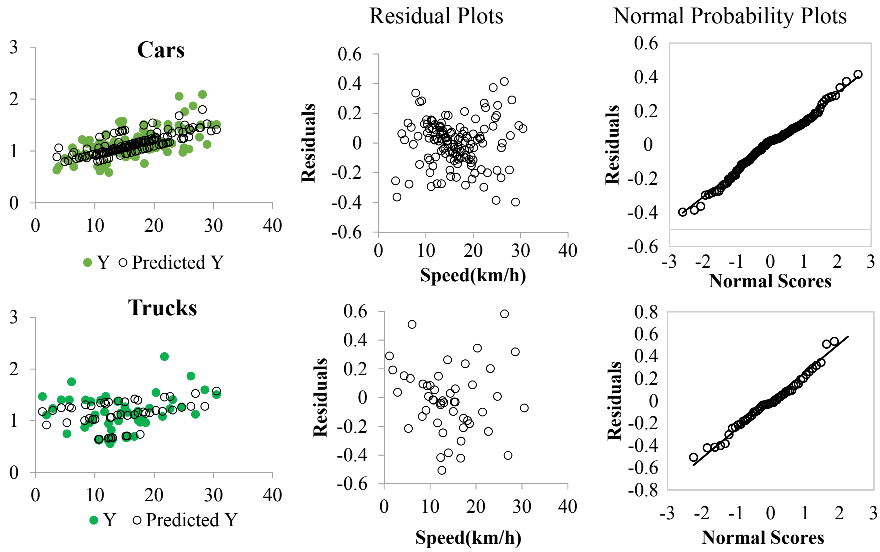


Fig. 5 Observed and predicted data with their residual plots and their normal probability plots

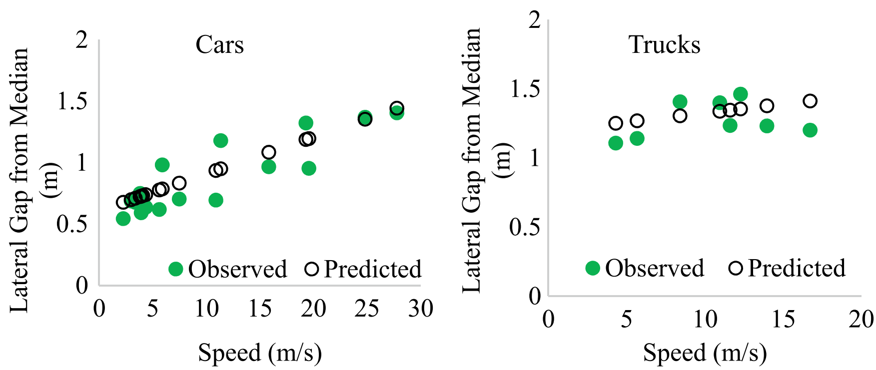


Fig. 6 Observed and predicted data of the lateral gap for the validation site

The minimum lateral gap maintained by cars and trucks from medians with different width and height is obtained and presented in Fig. 7. The impact of different width is presented by considering a fixed median height of 0.3 m (refer to Fig. 7a, c). Similarly, the impact of height is obtained and presented by keeping a fixed median width of 2.5 m (Fig. 7b, d).

From Fig. 7a, b, it can be observed that the lateral gap maintained by cars increases for medians with width, i.e. 2.5 and 4.5 m and a height of 0.3 m. There is no significant difference observed between the 2.5 and 4.5 m width medians with a standard height of 0.3 m. It is observed that for a median width of 1.2 m, cars feel safe to move

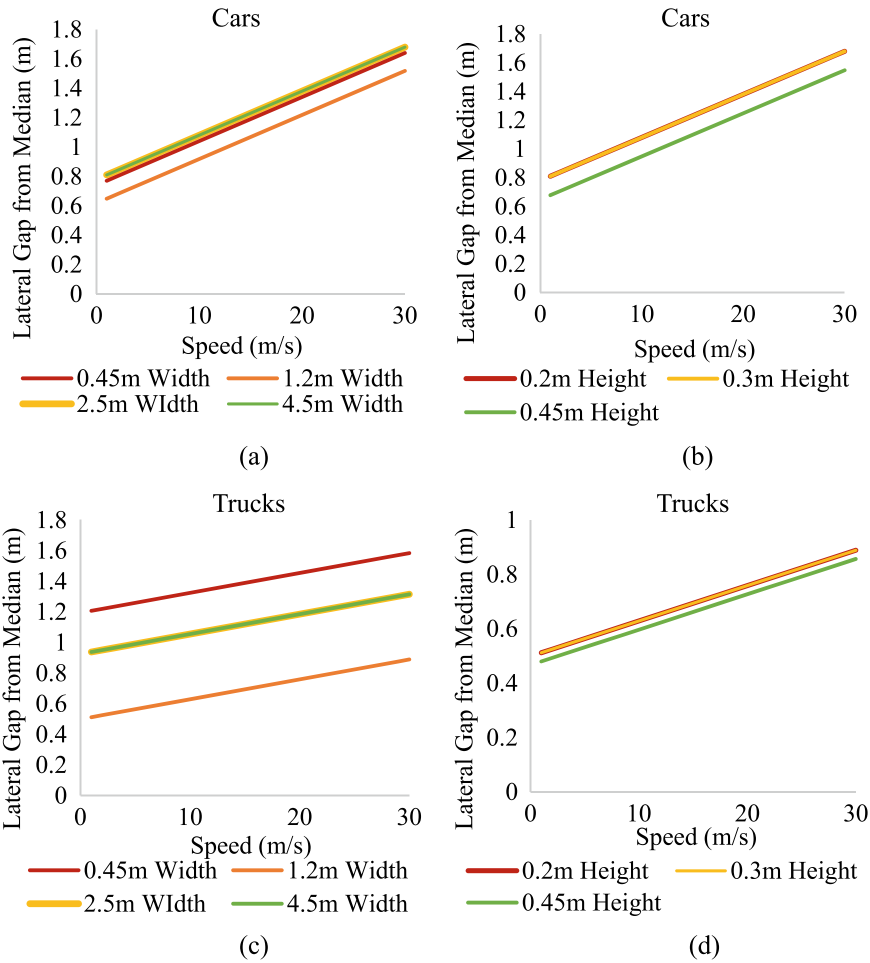


Fig. 7 Minimum lateral gap maintained by trucks and cars for a different combination of median width and height. **a** Cars with different median width, **b** cars with a different median height, **c** trucks with different median width, **d** trucks with different median height

closer to the median, which indicates that with the increase in width of medians for a standard height of 0.3 m, vehicles move closer to the median. However, in the case of 2.5 and 4.5 m width of medians vehicles shifts away from the median. From the video analysis it is observed that in the case of medians with larger widths, the plantations are observed. In such cases, though the height of median kerb is lesser, i.e. 0.3 m, riders perceive the height of median including the height of the plantation. So, they move away from the medians due to the safety concern. It is also observed that for a median height of 0.3 m, trucks maintain the higher gap for 0.45 m width of the median and it moves closer to the median for higher widths of 2.5 and 4.5 m. As seen in the case of cars, trucks also maintain a very closer gap for a median width of 1.2 m with a standard height of 0.3 m.

Similarly, the impact of median height is also studied for a standard median width of 2.5 m. It is observed that as the height of the median decreases cars move away from the median. In the case of trucks, it can be seen that the height of medians has a similar impact on trucks as in the case of cars. For a median height of 0.45 m, trucks maintain a slightly closer gap than 0.3 and 0.2 m median kerb height. It can also be observed that trucks move closer to the media than cars in all cases. This may be due to the tyre size of cars and trucks. The tyre size of trucks ($\approx$ 315 mm) is larger than the size of cars tyres ($\approx$ 205–235 mm). Due to the larger height of the wheel, the median height (i.e. up to 450 mm), does not have any significant impact on trucks, however, in case of cars due to the smaller wheel cars perceive the median as obstacles and moves away from the median. In the case of the median height of 0.45 m (450 mm), cars move closer to the median as the driver perceives the lesser risk of collision from the opposite traffic flow.

6 Concluding Remarks

In this part, the minimum lateral gap maintained by vehicles from the median are studied based on the type of median. It is found that the cars maintain a higher gap than trucks. The median height and width have a significant impact on the vehicles travelling near to the road edge. The minimum lateral gap maintained by vehicles also increases with the increase in operating speed of vehicles travelling along the median lane (ML). This may be due to the driver's safety perception. The drivers feel unsafe, whilst driving at the edge at higher speeds. Hence, they maintain a larger gap from the road edge. For the narrower width medians, i.e. width less than 0.5 m, both cars and trucks move away from the median. However, they feel safe to move closer to the median with a median width of 1.2 m width in all cases. Also, for larger width medians, i.e. 2.5 and 4.5 m, cars as well as trucks maintain a higher gap than the 1.2 m width medians.

Finding of this study can be used, whilst developing and to validate the simulation models developed for such traffic streams and the safety aspects of the road like the clearance from the edge of medians and shoulders.

This study was limited to the four-lane divided roads only. Also, the minimum lateral gap maintained by vehicles from the shoulder is not included in this study. Hence, in the future, the minimum lateral gap maintained by another different type of vehicles from both the road edges (median and shoulder) can also be studied for a different type of roads (roads with different widths).

References

1. Agent KR, Pigman JG (2008) Evaluation median barrier safety issues. Kentucky Transportation Center
2. Armour M (1985) Effect of road cross-section on vehicle lateral placement. Aust Road Res 15(1):30–40
3. Auberlet JM et al (2010) The impact of perceptual treatment on lateral control: a study using fixed-base and motion-base driving simulator. Accid Anal Prev 42(1):166–173
4. Balaji K, Bharadwaj MRK, Dey PP (2013) A study on lateral placement and speed of vehicles on two-lane roads. Indian Highw Indian Road Congr 41(9):49–54
5. Bunker JM, Parajuli A (2006) Examining lateral position of cars and heavy vehicles on a two-lane, two way motorway. Transp Eng Aust 10(2):129–139
6. Dewer RE, Olson PL (2001) Human factors in traffic safety. Lawyers & Judges Publishing Company
7. Dey PP, Chandra S, Gangopadhaya S (2006) Lateral distribution of mixed traffic on two-lane roads. J Transp Eng 132(7):597–600
8. Elvik R (1995) The safety value of guardrail and crash cushions: a meta-analysis of evidence from evaluation studies. Accid Anal Prev 27(4):523–549
9. FHWA (1993) The association of median width and highway accident rate. FHA, US Dept of Transportation
10. Finley MD, Funkhouser DS, Brewer MA (2008) Studies to determine the operational effects of shoulder and centerline rumble strips on two-lane undivided roadways. Texas Transportation Institute, The Texas A & M University System, Austin, TX
11. Hadi MA, Aruldhas J, Chow LF, Wattleworth JA (1993) Estimating safety effects of cross-section design for various highway types using negative binomial regression. Transp Res Rec J Transp Res Board 1500:169–177
12. Hallmark SL (2012) Relationship between speed and lateral position on curves. Part of MTC project (2009–03). Ames
13. Hauer E (2000) Shoulder width, shoulder paving and safety. Draft paper
14. Hauer E (2000) The median and safety. [Online]. Available at: www.trafficsafetyresearch.com. Accessed 26 Apr 2019
15. Holbrook LF, Kuo WH (1976) Median barriers and highway safety. Michigan Department of State Highways and Transportation Research, Michigan
16. Hong D, Lee Y (2005) Development of traffic accident prediction models by traffic and road characteristics in urban areas. In: Proceedings of the Eastern Asia society for transportation studies, Bangkok
17. IRC: 86 (1983) Geometric design standards for urban roads in plains, 1st edn. Indian Road Congress, New Delhi
18. IRC: SP 73 (2007) Manual of standarda & specifications for two laning of state highways on B.O.T. basis, 1st edn. Indian Road Congress, New Delhi
19. IRC: SP 84 (2014) Manual of specificantions and standards for four laning of highways through public private partnership, 1st edn. Indian Road Congress, New Delhi
20. Mahapatra G, Maurya AK (2015) Study on lateral placement and speed of vehicles under mixed traffic condition. Eastern Asia Society for Transportation Studies, Cebu City

21. Maurya AK et al (2014) Sensor based measurement of vehicle's lateral clearance with road edge, TPMDC 2014. IIT Bombay, India
22. Middleton G (1976) Marking of edge lines on narrow pavements. Aust Road Res 4(4):25–31
23. Moreno JJ, Pol AP, Abad AS, Blasco BC (2013) Using the R-MAPE index as a resistant measure of forecast accuracy. Psicothema 25(4):500–506
24. Porter R, Donnell E, Mahoney K (2004) Evaluation of effects of centerline rumble strips on lateral vehicle placement and speed. Transp Res Rec J Transp Res Board 1862:10–16
25. Rasanen M (2005) Effects of a rumble strip barrier line on lane keeping in a curve. Accid Anal Prev 37(3):575–581
26. Rosey F, Auberlet JM, Moisan O, Dupre G (2009) Impact of narrower lane width: comparison between fixed base simulator and real data. Transp Res Rec J Transp Res Board 2138:112–119
27. Sagberg F (2006) Effects of a painted median on lateral position and speed. In: Research into practice: 22nd ARRB conference, Canberra
28. Shorpe EB (1950) The effect of various types of median dividers on the lateral positioning of cars. Bureau of Highway Planning, New York State Department of Public Works, New York
29. Siregar ML (2015) Hazard-driven drivers' behaviours towards vehicle-to-median safe distances. Makara J Technol 19(1):5–10
30. Stamatiadis N et al (2009) Impact of shoulder width and median width on safety. National cooperative highway research program (NCHRP). Transportation Research Board
31. Stodart BP, Donnell ET (2008) Speed and lateral vehicle position models from controlled night time driving experiment. J Transp Eng 134(11):439–449
32. Tamiya N, Mandai H, Fukae T (1996) Optical spread spectrum radar for lateral detection in vehicles. In: IEEE 4th international symposium on spread spectrum techniques and applications proceedings, Mainz
33. Teshima T et al (2006) Vehicle lateral position estimation method based on matching of top-view images. In: 18th international conference on pattern recognition. ICPR 2006, Hong Kong
34. Van Driel CJ, Davidse RJ, van Maarseveen MF (2004) The effects of an edge line on speed and lateral position: a meta-analysis. Accid Anal Prev 36(4):671–682
35. Wong YD, Nicholson A (1993) Speed and lateral placement on horizontal curves. Road Transp Res 2(1):74–87
36. Woods WC (1972) A study on the lateral placement of automobiles due to restrictions on two-lane rural highways. Master's thesis, Missouri University of Science and Technology-ROLLA, Missouri
37. Yang JS (2012) Estimation of vehicle's lateral position via Lucas-Kanade optical flow method. WSEAS Trans Syst 11(8)
38. Zerger C, Deacon J (1987) Effect of lane width, shoulder width and shoulder type on highway safety. Transportation Research Board, National Research Council, Washington, DC
39. Zerger CV, Council FM (1995) Safety relationships associated with cross-sectional roadway elements. Transp Res Rec J Transp Res Board 1512:29–36
40. Zhou J, Shi ZK, Cao JL (2014) Nonlinear analysis of the optimal velocity difference model with reaction time delay. Phys A 396:77–87

Assessment of Impact of Roadside Bus-Stops on Capacity of Urban Divided Roads Using Microscopic Simulation Model

Pooja Raj, Gowri Asaithambi, and A. U. Ravi Shankar

1 Introduction

In developing countries like India, most of the roads in urban areas are found to be congested due to the presence of several types of vehicles with widely varying dynamic and static characteristics with the absence of lane discipline. Congestion leads to various problems like delay of travel, reduction in speeds of vehicles and capacity of roads. The major reason for congested roads in India is not only the mixed traffic and weak lane discipline, but also due to many activities that prevail on the sides of the road (e.g., bus-stops, parking, and street vendors) affecting the traffic movements and thereby affecting the performance of roads. Among these roadside activities, the presence of bus-stops has a significant influence on capacity of roads as the demand of public transport buses in urban areas of developing countries is relatively high. Moreover, most of the bus-stops in India are curb-side bus-stops due to the lack of sufficient space for construction of exclusive bus lanes or bus bays. Curb-side bus-stops are one of the major reasons for reduction in speed and capacity of urban roads. They obstruct the traffic flow as the buses stop in the same lane of travel creating a bottleneck condition. This results in reduction of road width at the location of bus-stops causing congestion and decline in capacity. Hence, this study is aimed to assess the impact of bus-stops on capacity of urban divided mid-block sections. Moreover, it is aimed to analyze the change in stream speed of vehicles and capacity of roads with and without bus-stop by varying proportion of buses.

P. Raj (✉) · A. U. Ravi Shankar
Department of Civil Engineering, National Institute of Technology Karnataka, Surathkal, Mangalore, Karnataka 575025, India

G. Asaithambi
Department of Civil and Environmental Engineering, Indian Institute of Technology Tirupati, Tirupati, Andhra Pradesh 517506, India
e-mail: gowri@iittp.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_14

The presence of several types of vehicles with varying static and dynamic characteristics, their complexities in vehicular interaction and non-lane discipline in mixed traffic conditions make it difficult to study the impedance of bus-stops on capacity of roads in urban areas using macroscopic speed-flow relationships. Most of the studies used empirical methods to investigate the effect of bus-stops and other side frictions on traffic flow characteristics in both homogeneous and mixed traffic conditions [1–6]. However, limited attempts have been made to study the influence of bus-stops on capacity of urban roads [7]. Out of these, very few researchers used microscopic simulation models to study the influence of bus-stops on capacity in mixed traffic conditions [8] where they examined the effect of dwell times on speed and traffic flow. Based on the review of literature, it is observed that various traffic simulation models and commercial software (e.g., VISSIM) have been developed for modeling homogeneous conditions [9, 10]. But, the scope of applicability of these models and software to mixed traffic was not clearly verified due to the presence of wide variety of vehicles traveling without lane discipline. Based on the above motivation, this study focused on developing a microscopic traffic simulation model to study the impact of bus-stops on capacity of urban divided mid-block sections under mixed traffic conditions.

2 Data Collection and Extraction

The study stretch selected for this study is a four-lane divided urban mid-block section (two lanes for each direction of traffic flow) in Bangalore city. In this study, traffic data were collected from one direction of the selected road (7 m wide). Data collection was performed on two sections of the same road where one section is free from side frictions (ideal section) and another with an on-street bus-stop. Data were collected using video-graphic technique on a weekday during morning peak and off-peak periods of traffic. Cameras were mounted on two separate elevated points to cover 70 m length of each section, i.e., ideal section and the section with bus-stop. Figure 1 shows the photographs and layouts of ideal section and section with bus-stop.

The types of vehicles considered in the study are motorized two-wheelers (TW), cars, three-wheelers (THW), buses, and heavy vehicles (HV). The peak hour for both the sections was identified as 8.40–9.40 a.m. with two-wheelers having a traffic composition accounted for the largest share of 53% and heavy vehicles (includes LCV and Trucks) with the least share of 1%. The off-peak flow was taken from 5.00 to 6.00 a.m. for both the locations. The traffic composition of the study sections is shown in Fig. 2.

The collected videos from both the sections were played in image processing software, Irfanview 4.38 [11] to extract the disaggregate data from videos at a rate of 30 frames per second. Gridlines with sufficient scale were plotted in AUTOCAD with obtained (X, Y) image coordinates and overlaid on video using a Video editor. Gridlines overlaid on the study sections were divided into lateral blocks of 1 m each

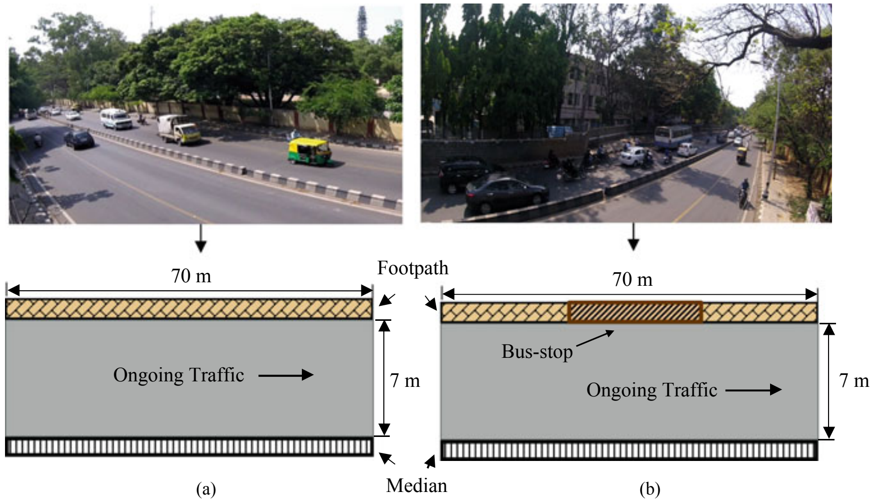


Fig. 1 Photographs and layouts of study section in Bangalore **a** ideal section and **b** section with bus-stop

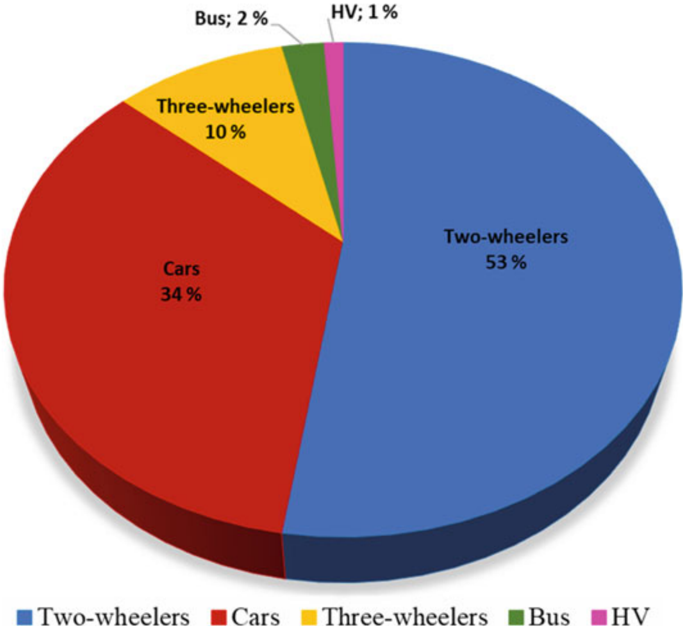


Fig. 2 Traffic composition of the study sections at Bangalore city

and longitudinal blocks of 5 m each. The required data extracted from ideal section are time-gaps, free flow speed, speed of vehicles, acceleration and deceleration, lateral positions, longitudinal, and lateral gaps of different types of vehicles. In the section with bus-stop, data extracted along with the above data are frequency of stopping and non-stopping buses, longitudinal and lateral position of bus-stop, bus dwelling time, acceleration and deceleration rate of buses, position at which bus start decelerating and position at which bus attains its normal speed.

3 Development of Simulation Model

A traffic simulation model for an urban divided mid-block section in mixed traffic conditions was developed based on microscopic approach. In this model, the entire road width is considered as a single unit instead of individual lanes to represent non-lane disciplined traffic. The vehicles are depicted as rectangular blocks with appropriate dimensions (adopted from [12]) occupying a specified area of road space. The interval scanning technique with fixed increment of time (scan interval) is used in the model. The scan interval for updating the vehicles is taken as one second in the model. The framework of simulation model for mid-block sections consists of mainly three major logics: vehicle generation, vehicle placement, and vehicle movement. The flow diagram illustrating the basic logical aspects of the developed simulation model is shown in Fig. 3.

3.1 Vehicle Generation

Vehicle generation is the primary step involved in mid-block traffic simulation. As traffic variables are stochastic in nature, simulation models require randomness to be incorporated to take care of stochasticity. This is easily done by creating a sequence of random numbers to generate vehicles. The vehicles are generated based on headway distribution. Distribution models of headway established for homogenous traffic conditions may not be applicable to mixed traffic conditions. In mixed traffic conditions, vehicles of smaller sizes like two-wheelers move parallelly even in a single lane and across the entire width of the road, thereby creating significant number of zero headways.

Headway values are observed from entire road width instead of individual lanes under mixed traffic conditions due to the absence of lane discipline. Here, the vehicles were generated using negative exponential distribution obtained based on the field data using Eq. (1).

$$H = -(1/\lambda)\ln R, \tag{1}$$

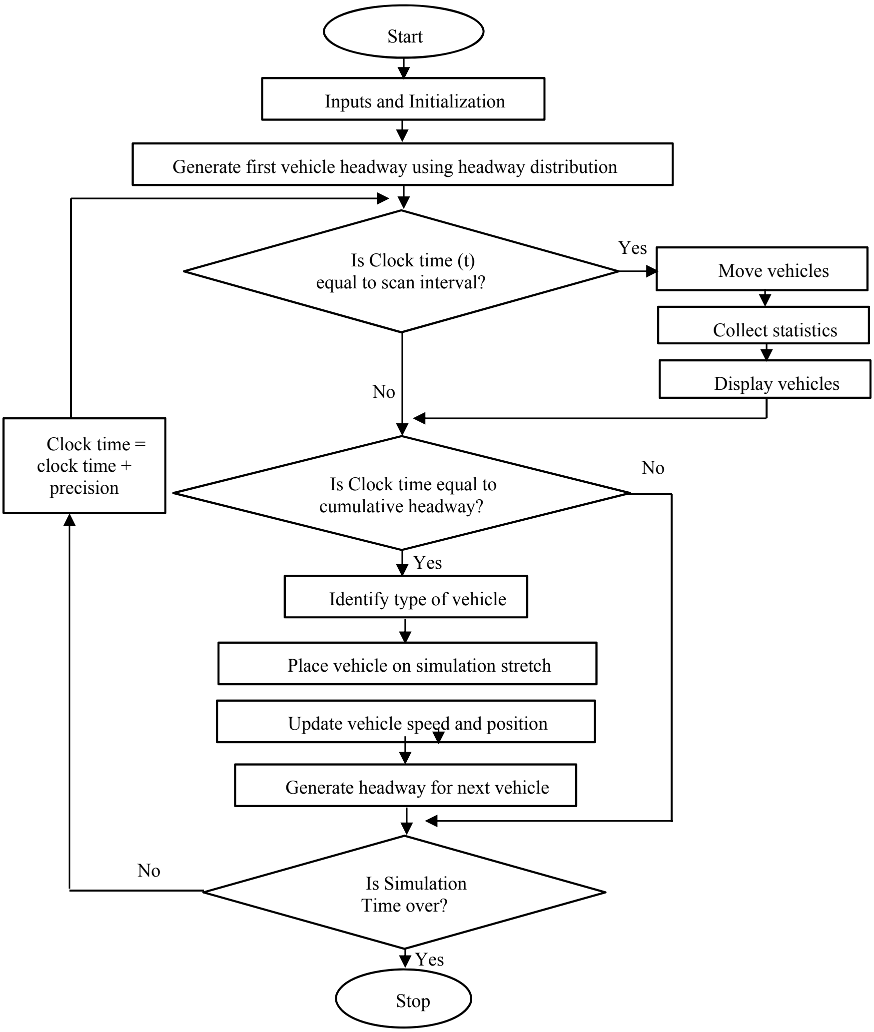


Fig. 3 General framework of simulation model

where, H = headway or time gap between arrival of successive vehicles (s), λ = mean time gap of the vehicles (vehicles per second) and R = random number in the range of 0–1. The headway range from the field was observed between 0 and 7 s. The first random number stream was used to calculate the headway using the range of headway. Thus, the vehicles were generated. Once a vehicle is generated, the types of vehicles are identified based on traffic composition. For this purpose, a set of random numbers generated is compared with the cumulative composition of each type of vehicle. Accordingly, the type of the vehicle was decided. Whenever a vehicle is generated, it is assigned with their static (length and width) and dynamic (acceleration, deceleration, etc.) characteristics along with its free flow speed. It was assumed that all vehicles enter the simulation road stretch with their free flow speed, and during the simulation process the vehicles will not be allowed to exceed their assigned free speeds. The free speeds of vehicles follow normal distribution based on the field data. Hence, the standard normal deviates are generated using Box–Muller transformation method using the following equation.

$$Z = \sqrt{-2\ln R_1 x}\cos(2\Pi R_2), \tag{2}$$

where, Z = standard normal deviate; R_1, R_2 = random numbers.

The free speed of each vehicle category was obtained using Eq. (3).

$$u_{\text{fi}} = \mu_i + \sigma_i Z, \tag{3}$$

where, u_{fi} = free flow speed of the vehicle type i; μ_i = mean free flow speed of vehicle type i; σ_i = standard deviation of speed of vehicle type i.

3.2 Vehicle Placement

In developing countries, vehicles occupy the entire space of road without any lane discipline. Thus, a suitable position is chosen for placing the vehicle across the width and length of the road depending upon available lateral and longitudinal gaps. The gaps of each vehicle were calculated based on their current speeds using second-degree polynomial relation obtained from the field data. Any generated vehicle is placed at the beginning of the simulation stretch considering a safe longitudinal gap available based on the free speed assigned to the vehicle, overall width of the vehicle and available lateral gaps.

When the clock time becomes equal to cumulative headway, vehicle placement logic was implemented. From the field data, it is observed that mostly vehicles travel near the median of the road as they can maintain their higher speeds near the median. Hence, the gaps available for vehicle placement were checked consecutively starting from the median of the road (right to left edge). If the arriving vehicle is the first vehicle, then checking for gaps was neglected as this vehicle is free from obstructions of other vehicles. If the available lateral and longitudinal gaps are sufficient for the next vehicle to get placed near the median, it was placed leaving a specified lateral gap from the right edge. If the gaps are deficient, vehicle tried to shift laterally according to its alignment and then gaps were checked for placement in the next position. Still if the gaps are inadequate, that particular vehicle was kept in queue and it was checked for placement in the next scan interval. During every scan interval, the vehicles in the queue are given preference over a newly generated vehicle for placement.

3.3 Vehicle Movement

Logics of vehicle movement was invoked so that the positions of all vehicles placed are updated sequentially from beginning to the exit end of the road stretch. The speeds and positions of vehicles were updated at each scan interval (i.e., 1 s). Each vehicle performed their longitudinal and lateral movements according to car following and lane changing logics. It was assumed that the vehicle can only accelerate either to its free speed or to the speed limit specified for the road stretch, whichever is minimum. The movable distance of each vehicle was calculated and the possibility of free movement was checked considering the influence of leader vehicle. If the vehicle is not free to move, then the vehicle changed its lane based on the available lateral and longitudinal gaps and also by considering the speed of its leader in new position to ensure that the vehicle can accelerate. Otherwise the vehicle tried to decelerate and follow the leader vehicle in its current position. Vehicle following models better describe the longitudinal behavior of the vehicle. The model describes the interactions of subject vehicle and leader vehicle. Two different longitudinal models were used in vehicle movement logics to identify the suitable car following model among these for mixed traffic conditions by evaluating their performances. In mixed traffic, roads in urban areas remain congested due to the presence of several vehicle types with weak lane discipline and hence, subject vehicle follows the leader closely and choose its speed and following distance in order to avoid rear-end collision. Thus, car following models i.e., safety distance models such as Gipps model and Intelligent Driver model (IDM) are considered reasonable and ideal in this study.

Gipps's car following model with vehicle dependent parameters was formulated on the basis of different conditions [13]. The formula for the estimating vehicle speed comprises of two sections: accelerating and decelerating section. The first part assures that the vehicle accelerates to its desired speed with an acceleration rate that initially increases with speed and then decreases to zero as the vehicle approaches its desired speed. If the subject vehicle (SV) is not impeded by any other influencing vehicles, the vehicle moved at its desired speed. When the movement of the vehicle is not influenced by the preceding vehicle, the updated longitudinal speed is given by:

$$G_a(t) = u_n(t) + 2.5a_n\tau(1 - u_n(t)/V_n)(0.025 + u_n(t)/V_n)^{1/2}. \tag{4}$$

If the SV is impeded by the preceding vehicle, its speed is dependent on the properties of the preceding vehicle. The SV is influenced by leader vehicle (LV) when the widths of both vehicles overlap each other. When the SV had lesser speed than LV, it accelerated and followed the leader. When the SV had greater speed than LV, SV initiated lateral shift or decelerated at its current position. In these cases, both acceleration part and deceleration part were calculated to obtain speed and the minimum speed is used as the updated speed. The updated speed is given by:

$$u_n(t + \tau) = \min\{G_a(t), G_d(t)\}, \tag{5}$$

where,

$$G_a(t) = u_n(t) + 2.5a_n\tau(1 - u_n(t)/V_n)(0.025 + u_n(t)/V_n)^{1/2}, \tag{6}$$

$$G_b(t) = b_n\tau + \left(b_n^2\tau^2 - b_n(2(x_{n-1}(t) - \alpha s_{n-1} - x_n(t)) - u_n(t)\tau - \left(u_{n-1}(t)^2\right)/b^\wedge)\right)^{1/2}, \tag{7}$$

where, $u_n(t)$ is the speed of SV "n" at time t, a_n is the maximum acceleration which the driver of SV "n" wishes to undertake, τ is the reaction time, V_n is the desired speed at which the driver of SV "n" wishes to travel, b_n is the maximum deceleration that the driver of SV "n" wishes to undertake, $x_{n-1}(t)$ is the location of the front of LV "$n-1$" at time t, $x_n(t)$ is the location of the front of SV "n" at time t, s_{n-1} is the effective size of leader "$n-1$," that is, the physical length plus a margin into which the following vehicle is not willing to intrude, α is the sensitivity factor and $b^\wedge$ is the maximum deceleration expected from the leader.

When the SV had greater speed than LV, it initiated lateral shift by checking the gaps on both the sides and ensured that the vehicle was able to accelerate in its new position. If it is possible, then the SV tried to shift either to its right or left side and then accelerates, otherwise the SV remained in its present position and decelerated following the leader vehicle. The position of subject vehicle n at time $t+\tau$ is updated using the following equation.

$$x_n(t+\tau) = x_n(t) + 0.5(u_n(t) + u_n(t+\tau))\tau, \tag{8}$$

where, $u_n(t)$ is the speed of subject vehicle "n" at time t; $x_n(t)$ is the location of the front of vehicle "n" at time t; τ is the reaction time of the driver.

Similarly, Intelligent Driver car following Model (IDM) [14] was also used to model longitudinal movement of vehicles in simulation model. IDM is a time-continuous car following model for the simulation of urban traffic. Similar to Gipps model, when the vehicle is not influenced by leader vehicle, the subject vehicle was free to move and accelerate to its desired speed, otherwise it either decelerated or shifted laterally. For vehicle "n," x_n denotes its position at time t, v_n is its speed and l_n give the length of the vehicle. To simplify notation, the net distance is defined as $s_n = x_{n-1} - x_n - l_{n-1}$, where "$n-1$" refers to the leader vehicle, and the speed difference, $\Delta v_n = v_n - v_{n-1}$. The acceleration or deceleration rate of the vehicle is updated using the following equation.

$$A = a_n\left(1 - \left(\frac{v_n}{V_n}\right)^\delta - \left(\frac{s^*(v_n, \Delta v_n)}{s_n}\right)^2\right), \tag{9}$$

where,

$$s^*(v_n, \Delta v_n) = s_0 + v_n T + \left(\frac{v_n \Delta v_n}{2\sqrt{a_n b_n}}\right), \tag{10}$$

where, a_n is the maximum acceleration which the driver of SV "*n*" wishes to undertake, V_n is the desired speed at which the driver of SV "*n*" wishes to travel, b_n is the maximum deceleration that the driver of SV "*n*" wishes to undertake (a positive integer), s_0 is the minimum spacing, T is the desired time headway and δ is the acceleration exponent. The position of vehicle is updated using equation of motion:

$$x_n(t+\tau) = x_n(t) + v_n(t+\tau)\tau + \left(\frac{1}{2}\right)A(t+\tau)\tau^2, \tag{11}$$

where, $v_n(t+\tau)$ is the speed of subject vehicle "*n*" at time $t+\tau$; $x_n(t)$ is the location of the front of vehicle "*n*" at time *t*; τ is the scan interval and $A(t+\tau)$ is the acceleration or deceleration rate of the vehicle "*n*" at time $t+\tau$. When there is no influence of leader and if the subject vehicle is free to accelerate then the equation used is given by:

$$A = a_n\left(1 - \left(\frac{v_n}{V_n}\right)^{\delta}\right). \tag{12}$$

The outputs obtained from the model are number of each type of vehicles placed, corresponding headway values, average speed of each type of vehicles, average traffic stream speed and lateral and longitudinal position of each type of vehicles.

3.4 Object Oriented Programming Concepts

All the three logics used for simulating a basic unidirectional traffic on divided mid-block section was implemented in MATLAB using Object Oriented Programming (OOP) concepts. The components of OOP such as objects, classes, inheritance, encapsulation, and polymorphism help the programmer to develop more intelligible, maintainable, and expandable codes [15].

Objects are the basic run-time entities in OOP. A class is defined as collection of similar objects. Classes are user defined data types. In this study, the developed code was divided into different virtual functions. Major tasks in the code are performed using these virtual functions while the overall approach remains to be object oriented with two super classes. Objects such as *tw, car, auto, bus, and hv* represent different vehicle types in this study. These objects are inherited from the *Vehicle* class. Super classes i.e., *Vehicle* and *Link* classes are associated with the main *Simulator* class.

Vehicle Class. The super class *Vehicle* encapsulates the characteristics of a vehicle with already defined set of values for each type of vehicle. Subclasses like *tw, car, auto, bus*, and *hv* are derived from super class *Vehicle.* This class is characterized by various properties which build the vehicle structure. This class consists of vehicle properties such as vehicle type, vehicle length, vehicle width, acceleration, deceleration, free flow speed, longitudinal speeds, minimum longitudinal gap, minimum time gap, etc.

Link Class. The *Link* class encapsulates the performance of traffic flow. The vehicular manoeuvers such as longitudinal and lateral movement were composed in this class. The major task of *Link* class is to implement the logics of the model i.e., vehicle generation, vehicle placement, and vehicle movement. This class also updated the position and speed of vehicles at every time step. The *Link* class comprises of several virtual functions such as *Generation (), Placeveh (), UpdateVeh (), RunSim ()*, and *SimOut ()*. These functions execute the required operations to simulate traffic flow. The required input includes various parameters such as scan interval, total simulation time, width and length of the road, composition of the traffic, headway distribution, and minimum longitudinal and lateral gaps. The input data were provided in the class using virtual function *Link ()*. The class *Link ()* takes care of the initialization process based on user input.

Simulator Class. *Simulator* class operates the different steps of the entire simulation process. The vehicle trajectories at each scan interval was stored in the developed code and the virtual function *SimOut ()* was applied to display the vehicles on the road stretch at that instant of time in the simulation. The main function was used to create objects of two classes (*Simulator* and *Link*). The simulation run took place in this main function.

Class Relationships. A class diagram to explain the various classes used in the model development and their relationship with each other is presented in Fig. 4. "The symbols used for representing the object relationship follow the Unified Modeling Language (UML)" [16]. In the developed model, objects of the *Link* and *Vehicle* classes are real-world objects while, objects of *Simulator* class are conceptual objects.

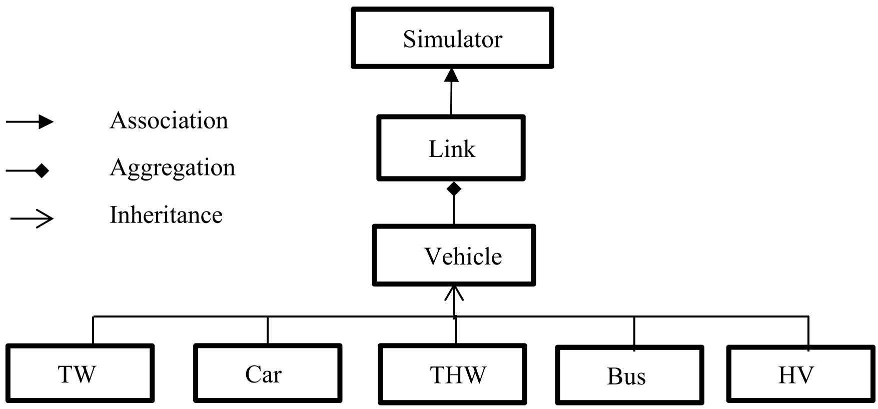


Fig. 4 Class diagram for unidirectional mixed traffic simulation

The classes *tw, car, auto, bus*, and *hv* are the subclasses derived from the super class *Vehicle*. Hence, the properties of *Vehicle* class are inherited by these subclasses. An object of *Link* class is an aggregation of objects of *Vehicle* class. Finally, the objects of class *Link* where all the logics were performed are associated with *Simulator* class.

4 Model Calibration and Validation

Calibration of the simulation model is the process of determining whether the simulation model is close enough to the real system. It is generally achieved through trial-and-error process which involves estimation of the model parameters, comparing the model to the actual behavior of traffic and using the discrepancies between the two to improve the results until the accuracy is judged to be acceptable. After developing the model, it was compared with the real-world system and was revised by making modifications to the inbuilt parameters if necessary, until the model accurately represents real traffic system. The simulation models developed using Gipps and IDM longitudinal movement models were calibrated separately to find a suitable model among the two that replicates the field conditions. The parameters such as reaction time (τ), sensitivity factor (α), deceleration of leader judged by subject vehicle ($\hat{b}$), acceleration rate (a_n), and deceleration rate (b_n) of subject vehicle are used to calibrate the model developed using logics of Gipps model. In the model developed using logics of IDM model used parameters like acceleration exponent (δ), desired or minimum time headway (T), and minimum spacing (s_0). Table 1 shows the calibrated parameters of both the models.

The developed models were also validated to check whether the models are capable of replicating the real traffic system. The data from the ideal section selected from Bangalore city for model development was used for validation. The model was validated by checking the error between the fields observed and simulated mean

Table 1 Calibrated parameters for Gipps and IDM model

Common parameters						
Car following model		Gipps			IDM	
Calibration parameters		τ (s)	α	$\hat{b}$ (m/s^2)	Acceleration exponent (δ)	
		1.0	1	−1.4	5	
Vehicle-specific parameters						
Calibration parameters		TW	Cars	THW	Buses	HV
Gipps	Acceleration rate (a_n) (m/s^2)	1.6	1.7	1.3	1.4	1.2
	Deceleration rate (b_n) (m/s^2)	−1.5	−1.4	−1.6	−1.4	−1.3
IDM	Desired time headway (T) (s)	0.5	0.5	0.5	1	1
	Minimum spacing (s_0) (m)	2	3	3	5	5

Table 2 Comparison of observed and simulated speeds of ideal section

Vehicle type		Mean speed values (m/s)		MAPE (%)
		Simulated	Observed	
Gipps	Traffic stream	13.4	12.7	5.51
	TW	14.3	13.2	8.33
	Cars	12.7	12.4	2.41
	THW	11.8	11.5	2.60
	Bus	11.8	11.6	0.01
	HV	11.2	10.6	5.66
IDM	Traffic stream	14.5	12.7	14.4
	TW	14.7	13.2	11.6
	Cars	13.1	12.4	5.7
	THW	12.8	11.5	12.0
	Bus	12.5	11.8	6.3
	HV	11.7	10.6	10.7

speed values of all vehicles (class-wise and aggregate). The comparison of observed and simulated mean speed values for different vehicle types for ideal section model is shown in Table 2. By calibrating and validating the models, the most accurate car following model among Gipps and IDM model for simulating four-lane divided mid-block section that replicates the field conditions was determined.

After calibrating and validating the simulation models, it was found that the mean absolute percentage error (MAPE) for each type of vehicle for both the car following models is less than 15% [11]. However, Gipps model gives lesser error for mean speed values for the entire traffic stream and also for different vehicle types compared to IDM model. Hence, the results indicate that Gipps car following model is better suitable for mixed traffic than IDM model.

5 Estimation of Capacity Reduction Due to Bus-Stops

5.1 Modification of the Model to Implement Bus-Stop

Gipps car following model was used to incorporate bus-stops in the developed simulation model as it is found to be more accurate than IDM model for mixed traffic. The developed model after calibrating and validating is capable of simulating the vehicular interactions associated on the same road stretch with an on-street bus-stop. Hence, the bus-stop was incorporated in the developed model by providing additional inputs such as frequency of stopping and non-stopping buses, longitudinal and lateral position of bus-stop, bus dwelling time, acceleration and deceleration rate of buses, the position at which bus start decelerating and the position at which bus attains its normal speed. Logics for incorporating bus-stop in the developed model are shown in Fig. 5.

Stopping buses and non-stopping buses were randomly selected with the help of a random number stream based on the composition of stopping buses and non-stopping buses. These buses were placed and moved on the simulation stretch based on the logics of the basic model developed. The buses going to stop near the bus-stop starts decelerating at a position before the bus-stop (obtained from the field) i.e., deceleration position and then stop near the bus-stop randomly within a range of

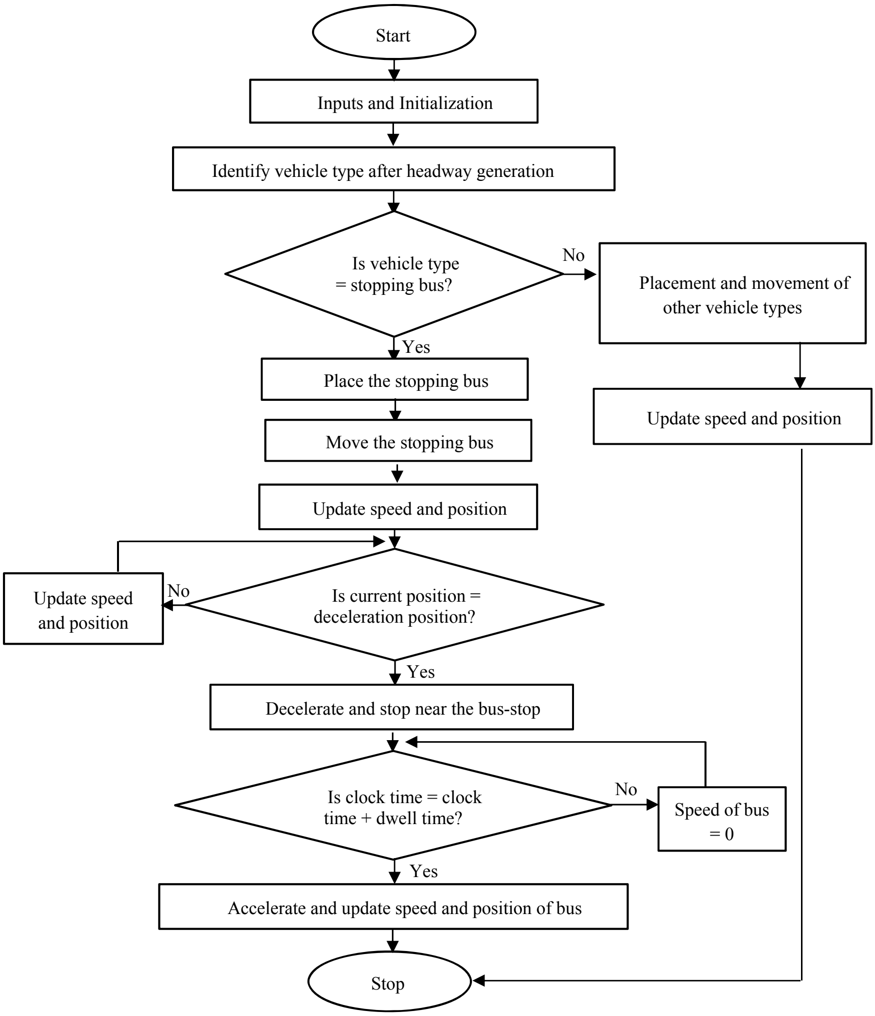


Fig. 5 Logics for implementation of bus-stop in the developed model

480–500 m in the simulation road stretch. Once, if the bus stops, it stopped for a dwelling time obtained from the field data and speed of the bus becomes zero. When clock time becomes equal to the sum of clock time at which bus stopped and dwell time, the bus tried to accelerate and update its speed and position. Dwell times of buses were randomly generated with normal distribution of mean 4.9 s and standard deviation 3.2 s.

These logics were also implemented in the model using OOP concepts. Classes like *stoppingbus* and *nonsbus* are derived from the super class *Vehicle* instead of *bus* class. A new class *Busstop* is associated with *Link* class with virtual functions such as *buspos(), decpos (), and normspd ()* to perform the logics involved.

5.2 Validation of Bus-Stop Model and Estimation of Capacity Reduction

Validation of on-street bus-stop model was done by considering the data obtained from bus-stop section selected near the ideal section. The average speeds of vehicles travelling across the bus-stop were obtained as simulation output. A comparison of observed and simulated average speeds of various types of vehicles has shown satisfactory results. Table 3 gives the comparison of mean speed values of simulated and observed data from bus-stop model. MAPE values for mean speeds of each type of vehicle and traffic stream is found to be less than 15%, hence replicating the field conditions.

Moreover, the change in speed values were observed due to the influence of bus-stop from the simulation results. The reduction in mean speed values of traffic stream in bus-stop section was found to be 22.5% with that of ideal section. Reduction in mean speed values of different types of vehicles is also observed. Results indicate that buses have maximum reduction in mean speed values (52.5%) as they decelerate and stop near the bus-stops. Due to the influence of bus-stop, other vehicles such as TW, cars, THW, and HV have a reduction of 21.8%, 20.0%, 20.9% and 15.3% in mean speed values, respectively.

Table 3 Comparison of simulated and observed speeds of bus-stop section

Vehicle type	Mean speed values (m/s)		MAPE (%)
	Simulated	Observed	
Traffic stream	10.3	10.0	3.3
TW	11.2	10.7	4.9
Cars	10.2	9.5	6.8
THW	9.3	9.2	1.4
Bus	5.6	6.2	8.6
HV	9.5	8.7	9.5

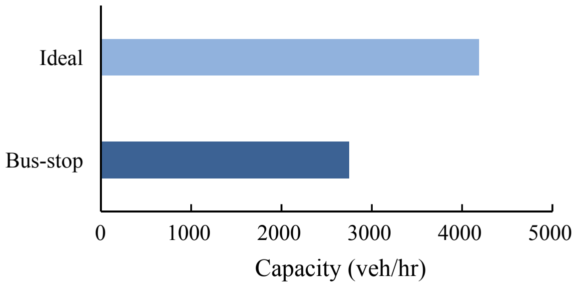


Fig. 6 Comparison of capacity (veh/h) of ideal section and bus-stop section

The bus-stop model was then used to estimate the capacity reduction due to the influence of buses stopping on the roadside. The capacity for both ideal section and bus-stop section was obtained from the corresponding simulation models developed i.e., basic model for ideal section and bus-stop model. The capacity of ideal section is obtained as 4208 veh/h (3008 PCU/h) which is close to the capacity of four-lane divided urban road given as 2700 PCU/h (as per Indo-Highway Capacity Manual). The capacity of the section with bus-stop is obtained as 2747 veh/h (1892 PCU/h). It was observed that there is reduction of 34.4% in capacity due to the presence of bus-stop on the road section. From the obtained results, it is clear that if an ideal section is incorporated with an on-street bus-stop without any exclusive lanes for the buses to stop, then the speeds of other vehicles get affected thus causing a decline in the existing capacity of the particular road. Figure 6 shows the comparison of capacity (veh/h) of ideal section and bus-stop section.

5.3 Effect of Change in Proportion of Buses on Capacity and Speed

In the model, the given composition of buses from the observed traffic flow was 2% with 1.7% of stopping buses. The capacity in ideal section (without bus-stop) obtained with this composition of buses is 4208 veh/h and the capacity in the section with bus-stop is 2747 veh/h. Thus, it was found that there is reduction of 34.4% in capacity of urban divided roads with 2% of buses out of which 1.7% are stopping buses. To better understand the change in capacity values with change in proportion of buses (%) and also, to find the change in reduction of capacity due to the influence of bus-stop with change in proportion, the percentage composition of buses was modified from 2 to 1%, 4%, 6%, 8%, 10%, 12%, 14%, 16%, 18%, and 20%, respectively, in both ideal section model and bus-stop model. Accordingly, modifications were made in the composition of other vehicle types with corresponding change in percentage composition of buses. Table 4 gives a comparison of capacity (veh/h) and mean stream speed (m/s) values of ideal and bus-stop section with change in proportion of buses.

Table 4 Capacity and stream speeds for ideal and bus-stop sections for various proportions of buses

Proportion of buses (%)	Capacity (veh/h)			Mean traffic stream speed (m/s)		
	Ideal section	Bus-stop section	% Reduction	Ideal section	Bus-stop section	% Reduction
1	4372	3347	23.4	13.4	11.5	14.7
2 (observed)	4208	2747	34.7	13.4	10.3	22.5
4	3974	2516	36.7	13.2	9.5	28.0
6	3895	2248	42.3	13.2	8.1	38.6
8	3728	2135	42.7	13.1	7.8	40.3
10	3658	1874	48.7	12.5	7.5	40.5
12	3490	1624	51.0	11.7	7.0	41.1
14	3398	1590	53.2	11.6	6.7	42.7
16	3318	1546	53.4	11.3	6.5	43.0
18	3192	1411	55.8	10.8	6.2	43.1
20	3148	1211	61.5	10.7	5.7	46.7

With the change in composition of buses, the duration of temporary bottleneck condition also changes there by leading to a difference in traffic flow levels and stream speed with composition of buses. From the analysis, it was found that the capacity and stream speed of vehicles showed a decline in their values for every 2% increase in the bus composition. Moreover, the percentage reduction of capacity and stream speed comparing ideal section and bus-stop section had an increase with increase in bus composition. As the bus composition increased up to 20%, lower levels of traffic flow for both ideal section (3148 veh/h) and bus-stop section (1211 veh/h) were observed. The mean stream speeds of vehicles in both ideal section and bus-stop section with increase in bus composition to 20% was also found to be lower having observed values as 10.7 m/s and 5.7 m/s, respectively. However, when there is a decrease in bus composition to 1% from the field observed value (2%), an increase in capacity (Ideal section: 4372 veh/h and Bus-stop section: 3347 veh/h) and stream speed values (Ideal section: 13.4 m/s and Bus-stop section: 11.5 m/s) were observed. It was also found that there is a decrease in percentage reduction of capacity and stream speeds, when comparing the values of ideal section and bus-stop section, with decrease in percentage composition of buses. Hence, it is clear that more the number of buses in the traffic stream, lower is the traffic flow levels and traffic stream speeds as buses occupy most of the road space making the road width deficient for other vehicles to travel.

6 Summary and Conclusions

The presence of bus-stops on urban road networks often leads to congestion and deterioration in the quality of traffic flow. Thus, it is imperative to study the influence of bus-stops on capacity of urban divided mid-block sections. For this purpose, a microscopic traffic simulation model was developed for urban divided mid-block section under mixed traffic condition. Two different car following models, Gipps and IDM models were used to model the mixed traffic flow. Model validation results showed that Gipps model gives accurate results compared to IDM model. Hence, Gipps car following model was used in the simulation model to incorporate bus-stop in order to study the influence of bus-stops on speed and capacity. From the simulation model, it was found that there is a reduction of 22.5% in mean speed of traffic stream in bus-stop section when compared to that of ideal section. Also, the capacity obtained for ideal section is 4208 veh/h, and the capacity of the section with bus-stop is 2747 veh/h. Hence, the results indicate a reduction of 34.4% in capacity due to the influence of bus-stop. The variation in proportion of buses on capacity and speeds was also studied. It was observed that the duration of temporary bottleneck condition changes there by leading to a difference in traffic flow levels and stream speed with the change in composition of buses. The results indicate that more the number of buses in the traffic stream, lower is the traffic flow levels and traffic stream speeds as buses occupy most of the road space making the road width deficient for other vehicles to travel.

Further research can be performed to study the impact of bus-stops on dynamic PCU values of different vehicle types. Also, the effect of various factors such as dwell time of buses, influence length of bus-stop, etc., on capacity and dynamic PCU values of different vehicle types can be examined. This study can be further extended to study the impact of other roadside activities like pedestrians, parking, encroachments, etc., on capacity and PCU values of different vehicle types. The developed simulation can be applied to mid-block sections of any length. As the object oriented programming concept is used to develop the program in this study, further modifications can be made in the model to incorporate intersections by expanding the code. Bus-stops in urban roads of India are mostly found without bus bays which seriously affect the capacity. The amount of reduction in capacity due to presence of bus-stops are not well established in the available codes. This study finds interesting applications in developing standards related to capacity estimation and reduction due to side frictions in mixed traffic conditions. For this purpose, correction factors can be obtained to estimate the capacity of roads with side frictions.

References

1. Rao AM, Velmurugan S, Lakshmi KMVN (2017) Evaluation of influence of roadside frictions on the capacity of roads in Delhi, India. Transp Res Procedia 25:4771–4782

2. Pallavi G, Mehar A, Neeraja D (2016) Effect of side friction on the traffic flow behaviour of multilane divided urban road at mid-block sections. In: Proceedings of 12th transportation planning and implementation methodologies for developing countries, IIT Bombay, Dec 2016
3. Parmar R, Maru A, Chhabra RS (2016) Macroscopic analysis of side friction impact on speed of urban roads-case study of Jawahar Marg, Indore. In: Proceedings of 12th transportation planning and implementation methodologies for developing countries, IIT Bombay, Dec 2016
4. Chiguma ML (2007) Analysis of side friction impacts on urban roads: case study Dares-Salaam. Doctoral dissertation, KTH University, Sweden
5. Salini S, George S, Ashalatha R (2016) Effect of side frictions on traffic characteristics of urban arterials. Transp Res Procedia 17:636–643
6. Patel CR, Joshi GJ (2014) Mixed traffic speed–flow behaviour under influence of road side friction and non-motorized vehicles: a comparative study of arterial roads in India. Int J Civ Environ Struct Constr Archit Eng 8(11):1144–1150
7. Chand S, Chandra S (2017) Improper stopping of buses at curb side bus stops: reasons and implications. Transp Dev Econ 3(1):5
8. Koshy RZ, Arasan VT (2005) Influence of bus-stops on flow characteristics of mixed traffic. J Transp Eng 131(8):640–643
9. Yang Q, Koutsopoulos HN (1996) A microscopic traffic simulator for evaluation of dynamic traffic management systems. Transp Res Part C Emerg Technol 4(3):113–129
10. Xiong C, Zhu Z, He X, Chen X, Zhu S, Mahapatra S, Chang G, Zhang L (2015) Developing a 24-hour large-scale microscopic traffic simulation model for the before-and-after study of a new tolled freeway in the Washington, DC–Baltimore Region. J Transp Eng ASCE 141(6):05015001
11. Gowri A, Sivanandan R (2015) Evaluation of right-turn lanes at signalized intersection in non-lane-based heterogeneous traffic using microscopic simulation model. Transp Lett 7(2):61–72
12. Chandra S, Kumar U (2003) Effect of lane width on capacity under mixed traffic condition in India. J Transp Eng ASCE 129(2):155–160
13. Gipps PG (1981) A behavioural car following model for computer simulation. Transp Res Part B Methodol 15(2):105–111
14. Treiber M, Hennecke A, Helbing D (2000) Congested traffic states in empirical observations and microscopic simulations. Phys Rev E 62(2):1805–1824
15. Gowri A, Venkatesan K, Sivanandan R (2009) Object-oriented methodology for intersection simulation model under heterogeneous traffic conditions. J Adv Eng Softw 40(10):1000–1010
16. Bahrami A (1999) Object oriented systems development using unified modelling language. Irwin/McGraw-Hill, Singapore

State-of-Art on Data Mining and Microscopic Simulation Techniques for Evaluation of Urban Uncontrolled Intersections

Suprabeet Datta, Siddhartha Rokade, and Sarvesh P. S. Rajput

1 Introduction

Intersections are traffic facilities which are created after two or three roads having sufficient right-of-way conjoint and intersect at angles acuter or obtuse than 90°. Intersections are broadly unsignalized or signalized. Further, intersections can be narrowly classified as signal-controlled, sign-controlled, legal/police-controlled and uncontrolled based on minor and major road traffic manoeuvre control strategies. For an uncontrolled type of traffic intersection, the approaches are neither intimated with regulatory stop/yield signs on either of the intersecting roads (i.e. major and minor roads), and they are not regulated by legal restrictions (like traffic police enforcements) present in most of the developed/developing nations [1]. Performance evaluation represents evaluation and determination of road-user behaviour, operational, safety, geometric and environmental attributes of a traffic facility. This includes capacity assessment, delay analysis, level of service analysis, safety and user risk assessment with crash prediction modelling and control/priority assessment under different base and unstandardized roadway, traffic and environmental conditions with applicability of reliability. Capacity estimation is conventionally done using empirical and deterministic approaches [2] as suggested by global traffic engineering manuals of different countries like the United States Highway Capacity Manual, the Indian Highway Capacity Manual, the Indonesian Highway Capacity Manual, etc. Procedures enlisted in these manuals require extensive video/manual data be collected on a larger scale. For example, if a proper understanding of the complex heterogeneous traffic on Indian roads is to be achieved, real-life observations will have to be conducted on all types of roads in India on a complete range of vehicle type and composition in the traffic and a full coverage of traffic volumes and speeds.

S. Datta (✉) · S. Rokade · S. P. S. Rajput
Department of Civil Engineering, MANIT, Bhopal, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_15

Such a procedure is extremely tedious, time consuming and expensive. The empirical approach requires that data be collected on a large scale. For example, if a proper understanding of the complex heterogeneous traffic on Indian roads is to be achieved, real-life observations will have to be conducted on all types of roads in India on a complete range of vehicle type and composition in the traffic and a full coverage of traffic volumes and speeds. Such a procedure is extremely tedious, time consuming and expensive. At the end of the effort the building up of suitable relationships will be often fraught with uncertainties, and it often happens that a reliable relationship does not emerge. Traffic Engineers are, therefore, taking recourse to a Simulation Modelling. Microscopic simulation is a numerical technique for conducting virtual replica of traffic facilities on a digital platform which may include stochastic macroscopic or microscopic characteristics involving creation of simple mathematical models to describe the behaviour of a transportation system over extended periods of real time.

This paper presents a review on two aspects: Primary aspect deals with application of soft computing and non-parametric machine learning techniques in the microscopic traffic flow simulation framework; Secondary aspect deals on usage of simulation packages like VISSIM, AIMSUN or HCS in assessing performance evaluation of uncontrolled intersections. The authors used the following scientific databases for the literature investigation through our institute's digital journal and literature search archive of our central library (exclusively available for institute intranet users):

a. ABI Inform Complete (ProQuest)
b. ASCE Library
c. EBSCO Academic Search Complete
d. Ei Compendex
e. IEEE Xplore Digital Library
f. Index Copernicus Digital Search Engine
g. ScienceDirect
h. Scientific Electronic Library Online (SciELO)
i. Scopus
j. SpringerLink
k. Taylor & Francis Online
l. Web of Science.

A total of 59 studies related to the broad area was found to be relevant which mostly covers European, American and Asian nations. Both heterogeneity and homogeneity of traffic behaviour with focus on driver attitude in developing nations have been focused upon while performing the literature search. A parochial perspective was chosen (without being too much critical) for the literature search as a criterion.

2 Methodology Followed for the Literature Investigation

A stand-alone literature review was conducted using a systematic and rigorous standard which is often termed as a systematic literature review (SLR) which rules out the demerits of narrative review methodologies. Five stages of "SLR" as recommended by Denyer and Tranfield [3] were followed. Stages depicted in the smart art (Fig. 2) have been followed for selecting and evaluating the studies. It should be noted that most of the articles are double-blind folded published journal articles and conference proceedings from publishers of international repute. Selected keywords were: Uncontrolled intersections, microscopic simulation, heterogeneous traffic flow, homogeneous traffic flow, capacity, delay, level of service, performance evaluation, cellular automata, artificial neural networks, support vector machines, decision trees, fuzzy logic, gap acceptance, random forest models, game theory-based models, surrogate safety measures, object oriented simulation models, etc.

Some of the reviews are narrative in nature and therefore the outcomes of these reviews are reported inherently without any illustrative improvements or modifications. Therefore, both narrative and "SLR" procedure was used in representing the research gaps and findings in a structure manner in this paper. Table 1 classifies the literature investigation conducted for preparation of this review with research lines and the research sublines.

Table 2 gives a detail about the literature presented in this review grouped by various sublines identified from past 15 years. It is important to note here that a single article may be referenced under more than one research subline if only the main findings are related to various sublines. The research lines and sublines are formulated after rigorous discussions and previous inventories of the research objectives among the authors.

The number of articles reviewed for each research line and subline includes full length papers (original research, reviews or case studies) which are conjunctive in describing the ideal structured process of literature reviews followed according to the overall methodology of the "SLR" represented in Fig. 2. Each and every research line and subline have at least one article based on the utility of the subline decided

Fig. 1 Four-legged and three-legged T-type uncontrolled intersections in India with drivers not-following lane discipline and priority decisions being violated on both major and minor approaches

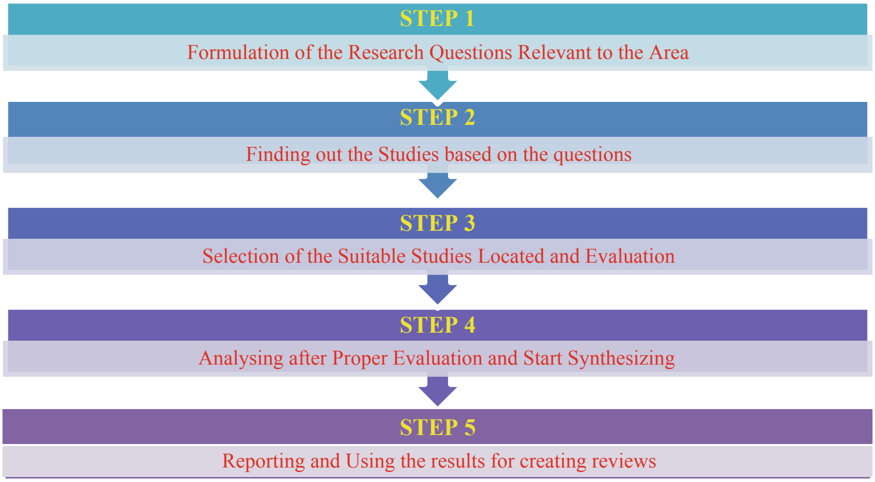


Fig. 2 Overall methodology of the "SLR" followed for the state-of-art review

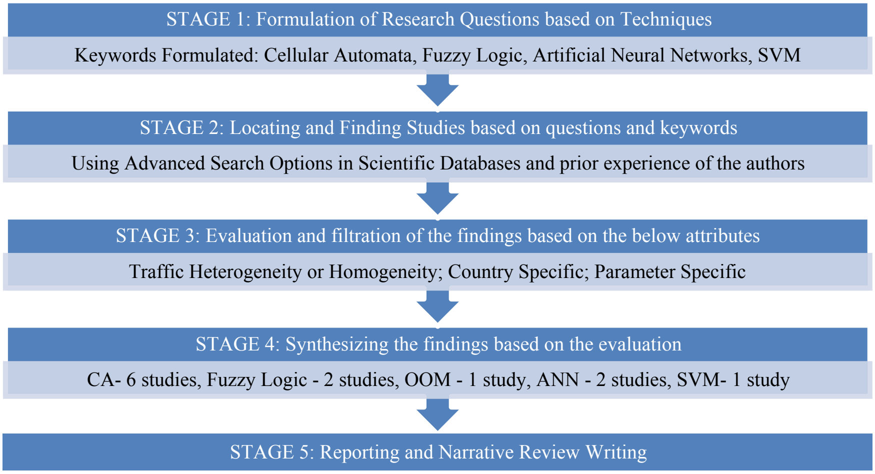


Fig. 3 Methodology followed for the review of soft computing and non-parametric estimation of uncontrolled intersection capacity literature

from the study. Overall performance evaluation of uncontrolled intersections broadly includes: (a) capacity assessment; (b) delay analysis; (c) crash prediction modelling; (d) level of service or comfortability and level of safety at intersecting roads; (e) gap acceptance behaviour and driver attitude analysis; (f) calibration and validation of fundamental car-following rules, priority rules, vehicular movement analysis, lane changing rules, lateral driving rules in microscopic simulation environments.

Table 1 State-of-art classification with research lines and their sublines

Sr. No.	Literature classification	Research lines	Research sublines
1.	**Class-I**: Performance evaluation of uncontrolled intersections utilizing soft computing, machine learning and data mining techniques including safety evaluation using surrogate measures	**Line 1**: Using cellular automata based simulation models	***Subline 1a***: Assessing measures of effectiveness (MOEs) like control delay, queue lengths and levels of service ***Subline 1b***: Capacity assessment based on calibrated and field observed values ***Subline 1c***: Formulation of complex traffic rules under varying geometric conditions
		Line 2: Using ANN/ANFIS models for validation and modelling	***Subline 2a***: Assessing gap acceptance behaviour based on driver behaviour ***Subline 2b***: Capacity assessment ***Subline 2c***: For service/control delay assessment based on field estimated values with model calibration and validation
		Line 3: Using binary logit/multiple parameter regression models and their comparison with ANN/ANFIS	***Subline 3a***: Comparative analysis of statistical techniques and machine learning algorithm-based capacity estimation ***Subline 3b***: Gap acceptance driver behaviour analysis
		Line 4: Using fuzzy logic/game theory-based concepts	***Subline 4a***: Object oriented programming with C++ code generations based on fuzzy rules formulations ***Subline 4b***: Chicken game theory-based algorithms for controlling autonomous vehicle movements at uncontrolled intersections using Cooperative Adaptive Cruise Control (CACC) systems

(continued)

Table 1 (continued)

Sr. No.	Literature classification	Research lines	Research sublines
		Line 5: Using support vector machines (SVMs), surrogate safety measures (SSMs), decision trees, random forest models	***Subline 5a***: Gap acceptance driver behaviour ***Subline 5b***: Capacity assessment ***Subline 5c***: Control delay assessment ***Subline 5d***: Crash probability assessment for minor and major roads
2.	**Class-II**: Performance evaluation of uncontrolled intersections utilizing microscopic traffic flow simulation considering heterogeneous and homogeneous traffic flow conditions	**Line 1'**: Under traffic homogeneity	***Subline 1a'***: Calibration of car-following parameters ***Subline 1b'***: Calibration based upon probabilistic/reliability-based variations in geometric attributes of intersections ***Subline 1c'***: Capacity assessment ***Subline 1d'***: Service/control delay and queue lengths assessment ***Subline 1e'***: Gap acceptance behaviour calibration
		Line 2': Under traffic heterogeneity	***Subline 2a'***: VISSIM calibration for capacity assessment ***Subline 2b'***: AIMSUN calibration for delay assessment ***Subline 2c'***: Web-based software application for delay and queue length determination and comparison with outputs obtained from machine learning and data mining estimations

Source Prepared by authors

3 State-of-Art on Soft Computing and Non-parametric Machine Learning/Data Mining Techniques for Performance Evaluation

Most of the capacity estimation procedures followed officially by developing countries are either empirical or deterministic is sense. Empirical methods need a lot of

Table 2 Fruition of the number of research articles by their research lines and sublines

Date	Research line 1			Research line 2			Research line 3		Research line 4		Research line 5				Research line 1’					Research line 2’		
Research sublines	1a	1b	1c	2a	2b	2c	3a	3b	4a	4b	5a	5b	5c	5d	a	b	c	d	e	2’a	2’b	2’c
1997–2006	1	0	0	0	1	1	0	1	0	1	1	0	0	1	1	0	1	0	1	0	1	0
2007–2012	2	1	0	1	0	1	1	1	1	0	1	0	1	1	1	0	1	0	1	1	0	1
2013–2019[a]	2	1	1	0	3	1	2	1	1	1	2	0	1	2	1	1	1	2	1	3	0	1

Source Prepared by authors
[a]Publications till February 2019

data collection on a larger scale. For example, if proper understanding of heterogeneous traffic flow behaviour is to be achieved, then many real-life observations is required in terms of vehicle composition, flow, speed, nature of manoeuvrability, geometric characteristics, vehicular characteristics, road-user characteristics, climatic considerations, situational environments, etc. Such kind of a procedure is time consuming, expensive and tedious. The other alternative for estimation of capacity is through microscopic analysis using numerical soft computing techniques such as Cellular Automata (CA), Fuzzy Logic (FLs), Artificial Neural Networks (ANN), Artificial Neuro Fuzzy Inference Systems (ANFIS), Support Vector Machines (SVM), Decision Trees (DTs), Random Forests (RFs), Object oriented simulation techniques, uncontrolled intersections, microscopic traffic flow simulation, capacity, delay, level of service (LOS), surrogate safety measures (SSMs), game theory, gap acceptance, additive conflict flow technique, heterogeneous, mixed, homogeneous, etc. The detailed list of machine learning and soft computing techniques being used over the past few years are summarized in a detailed manner in Table 1. The vast majority of the examination systems of unsignalized intersections depend on stochastic models; i.e. gap acceptance hypothesis. As indicated by GAP which was first universally introduced by the United States, development limits of Minor Street are probabilistically decided dependent on organized position of clashing significant street developments and exponential/Gaussian conveyance of minor street critical gaps and follow-up-times as recommended by Cvitanik. Before development of the HCM gap acceptance capacity estimation procedure, most part of the United States utilized the German and Swedish capacity estimation procedures for analysing the stop and yield-controlled multi-way unsignalized intersections. HCM used the reserve capacity and overall control delay as primary measures for analysing level of service (A through F). Another major drawback of the HCM procedure is over-estimation of critical gaps by 2–6% that actual field values. But after further analysis by the Transportation Research Board (TRB) conducted in the year of 1987 in cities like California, Florida and Chicago, TRB included vehicle delays for both major and minor movements for calculation of potential unsignalized intersection capacity. Also, the HCM did not give a comprehensive capacity estimation procedure for all-way stop-controlled intersections (AWSC). This was later sufficed by the HCM which provided relatively well structured methodologies for both two-way stop-controlled (TWSC) and all-way stop-controlled (AWSC) intersections. Although reserve capacity was still being used as a measure along with motorized and non-motorized vehicle delays for assessing unsignalized intersection level of service (LOS). A large portion of the investigation strategies of unsignalized intersections depend on stochastic models; i.e. gap acceptance hypothesis. In this hypothesis, it is accepted that a minor stream vehicle can enter an intersection, when the time interim to the following showing up higher need vehicle is bigger than a critical gap, and a sheltered time interim (follow-up time) has gone since the flight of the previous minor stream vehicle. The potential limit according to Transportation Research Board (TRB) of the United States in their recent version of the HCM characterizes potential capacity as that "perfect limit" for a particular subject vehicle development which fulfils the entirety of the accompanying conditions:

1. Traffic on significant roadway doesn't obstruct the traffic on minor roadway.
2. Traffic from close by intersections doesn't reinforcement into the intersection viable.
3. A different path is accommodated restrictive utilization of every minor road development viable.
4. No different developments (either vehicular or pedestrian) obstruct the subject development in thought.
5. Impedance on potential limit must be viewed as when traffic gets blocked in a high-need development.

When the clashing volume, critical gap and catch up time are known for a given development its potential capacity can be assessed utilizing gap acceptance models. The idea of potential capacity expect that every single accessible gap are utilized by the subject development i.e.; there are no higher need vehicular or person on foot developments and holding on to utilize a portion of the gaps it likewise accept that every development works out of a selective path. The potential capacity is processed utilizing the recipe given by Seigloch:

$$c_{\mathrm{px}} = v_{\mathrm{cx}} \times \frac{\mathrm{e}^{-v_{\mathrm{cx}} \cdot t_{\mathrm{cx}}/3600}}{1 - \mathrm{e}^{-v_{\mathrm{cx}} \cdot t_{\mathrm{fx}}/3600}} \tag{1}$$

where, c_{px} is the potential capacity of minor movement 'x' (veh/h), v_{cx} is the conflicting flow-rate for movement 'x' (veh/h), t_{cx} is the critical gap for minor movement 'x', and t_{fx} is the follow-up time for movement 'x'. According to the Indo-HCM, Capacity (c_x) for any movement 'x' at unsignalized intersections can be computed based to the gap acceptance model presented in Eq. (2). Capacity of a movement can be deduced from the estimated values of critical gap, follow-up time and conflicting flow-rates.

$$c_x = a \times v_{\mathrm{cx}} \times \frac{\mathrm{e}^{-v_{\mathrm{cx}} \cdot (t_{\mathrm{cx}} - b)/3600}}{1 - \mathrm{e}^{-v_{\mathrm{cx}} \cdot t_{\mathrm{fx}}/3600}} \tag{2}$$

where, c_x = capacity of movement 'x' (in PCU/h), $v_{\mathrm{c,x}}$ = conflicting flow-rate corresponding to movement 'x' (PCU/h), $t_{\mathrm{c,x}}$ = critical gap of standard passenger cars for movement 'x' (s), $t_{\mathrm{f,x}}$ = follow-up time for movement 'x' (s), and 'a' and 'b' = adjustment factors based on intersection geometry. The value of adjustment factor 'b' for right turn from major at intersections having two-lane major streets is considered to be negative, while the values of all other factors are positive. As such, the negative value of adjustment factor to account for the aggressive behaviour of minor street right turners in the above typology of intersection. Hebert studied variations of split volume (left-turning proportions, right turning proportions), average headway and effect of number of lanes on four-way stop-controlled intersection capacities. The author reported left-turning proportion have no effect on capacity whereas with one per cent increase in right turning vehicle proportion, capacities increase by 0.2%. Average headway which was obtained using the gap acceptance technique given by Wegmann for three right angled intersections came out to be same for both left and through vehicle capacity but significantly different for right turning vehicle capacity estimation. If 100/0 split volume is allowed, then under congested conditions a discharge rate of one vehicle for every 4.05 s can be expected. Modifying the split ratios to 50/50, capacity per lane increases to one vehicle for every 8.08 s. Khattak and Jovanis compared the stochastic gap acceptance (followed by United States) and the deterministic (i.e. empirical regression-based as followed by United Kingdom) based on theory and methodology, validity, policy, sensitivity, simplicity, data requirements and compatibility to the United States traffic flow conditions for evaluating uncontrolled and sign and yield-controlled intersections. It was found that in all the aspects considering the driver behaviour of the United States overall, the gap acceptance procedure is indeed relevant and can be included in further revisions of the US highway capacity manual for unsignalized intersection capacity analysis. Kyte and Marek studied effects of non-standard conditions like number of approach lanes, pedestrians and heavy vehicles on capacity estimates for 23 numbers of single lane approach all-way stop-controlled intersections collected from sites like Idaho, Oregon and Washington D.C of the United States. A methodology for operational performance is proposed by the authors who suggested that major and minor flow splits, heavy vehicle percentages and presence/absence of pedestrians indeed affect intersection movement capacity. Fisk generalized the Tanner's capacity equation for minor road vehicles to several lanes for non-priority movements at an isolated uncontrolled intersection by utilizing bunching of major road traffic and by incorporating dynamic critical gaps for different lanes according to Eq. (3):

$$C = q_1/\mathrm{e}^{q_1\tau}\left(1 - \mathrm{e}^{-q_1\beta_1}\right) \tag{3}$$

where, q_1 = major road flow; β_1 = service time for major road traffic to pass through the intersection; τ = dynamic critical gap for several lanes. Brilon and Brilon and Groβman introduced a new German guideline for capacity estimation at stop-controlled, priority-controlled and uncontrolled intersections based on gap acceptance concepts developed by Brilon and Dunne and Buckley for minor road capacity estimations based on delays and service times of major roads and the highway capacity manual. According to this new procedure, average delays (measure of traffic flow quality) for both major and minor roads are given importance in validating capacity estimation at unsignalized intersections. The authors also tested the results of the new procedure with the KNOSIMO-simulation tool in a series of realistic case studies from several parts of Germany. Kyte developed a theoretical framework for capacity estimation for all-way-stop-controlled (AWSC) intersections based on validation of twenty selected intersections to identify factors affecting gap acceptance capacity. Around 7000 departure headways were collected under different behaviour conditions for the validation. The results indicate that number of approach lanes, subject-approach lane volume split (%), proportion of left and right turns, percentage of heavy vehicles and flow-rate split (%) affect capacity at AWSC intersections. Ruskin and Wang in their experimental analysis for TWSC performance evaluation found that Cellular Automata (CA) models can overcome the shortcomings of the HCM gap acceptance capacity model. Therefore, the authors developed

CA models of 4 nos. four-legged TWSC intersections and found out that the capacity of minor streams depended on flow-rates of major streams and also on flow-rate ratio [FRR] (i.e. flow-rate of near lane/flow-rate of far lane) for a particular movement. Capacity of minor streams decreases with decrease in left-turning ratio [LTR] (i.e. flow-rate of major stream left turns/flow-rate of major stream) and with decrease in FRR, which demarcates that increase in flow-rate of near lanes. The CA model developed by the authors was able to describe the stochastic interaction between individual vehicles and can be used during situations for which headway distributions fails to describe traffic flow during occasions of non-priority. Li and Jiang set up a capacity model of uncontrolled intersections (in which the major stream mixed traffic flows cross m major lanes and the traffic flow headways fit the M3 distribution). The intersection composed of both heavy and light vehicles. The model is essentially an extension of minor-lane capacity theory first proposed by Troutbeck and Cowan which is $C_n = \frac{\alpha v_{\mathrm{p}} \mathrm{e}^{-\lambda(t_{\mathrm{c}}-\Delta)}}{1-\mathrm{e}^{-\lambda t_{\mathrm{f}}}}$, where, α, λ and Δ are the calibrated parameters and v_{p}, t_{c} and t_{f} are potential demand volume, critical gaps and follow-up-times for minor queuing, respectively. The model was tested and validated for Chinese traffic conditions and was found to be suitable (with MAPE values < 5%). Luttinen adjusted the movement capacities of lower rank streams for both higher rank queues after modifying the lower rank stream gap distributions and under lower flow-rates during the free flight times of higher position streams. The subsequent development limits were seen as higher than the HCM values, explicitly at low significant stream flow-rates. Chodur conducted studies in various cities of Poland in order to evaluate the adequacy of the HCM unsignalized intersection capacity parameters. HCM considers critical gaps and follow-up-times as prime calibration parameters, the authors examined the factors affecting these parameters under Polish conditions. The critical gap values for both major and minor left turns were found to be significantly different from the HCM values. Same was reflected for follow-up-time values. City size and number of lanes on the major roads are some of the factors affecting follow-up-time and critical gaps. For better fitting the empirical capacities of the selected two-way-stop/yield-controlled intersections (TWSC/TWYC), an additional scale parameter for modification of the critical gap values used in capacity calculation was developed. Akcelik reviewed some well-known analytical capacity estimation models for minor streams that use bunched exponential and simple negative exponential distribution of gaps in the opposing stream. The authors also found through realistic case studies that default parameters of HCM and Akcelik can be calibrated for intersection geometry and traffic characteristics like number of lanes, movement type, heavy vehicle per cent, grade, major road approach speed, restricted sight distance and delay time experienced by subject vehicles for which capacity is to be assessed. The authors compared two types of bunching models i.e. the Traditional-M1 and the Akcelik-M3D [SIDRA INTERSECTION]. The comparisons show that there is minor difference between the models for low major flows but differences in capacity estimates increase with high conflicting stream flows. Xu et al. modelled capacity reliability of minor streets for at-level controlled intersections for execution assessment. First-request reliability-based technique was embraced to change the capacity reliability file. Through an itemized LR, the creators dispersed the major and minor volumes as typical arbitrary factors. At last, the intersection execution was assessed utilizing capacity reliability, affectability and save capacity through a numerical model. Three kinds of gap conveyances specifically, exponential; moved exponential and Cowan's M3 were considered. Some of the major findings are:

1. Minor street capacity reliability is most elevated when significant street gaps follow exponential conveyance, second most elevated for Cowan's M3 and least for moved exponential regardless of default volume levels.
2. Volume and capacity reliability is most elevated when significant street progress follows exponential conveyance.
3. Minor street capacity reliability diminishes with increment sought after volumes for both major and minor streets.

Binti and Al-Masaeid studied the effects of volume on driver's critical gap acceptance at a TWSC. The authors pointed that HCM critical gap estimation model neglects the account for the effects due to increasing traffic volume. Authors also stated that the HCM procedure does not take into consideration a driver's typical behavioural characteristics due to time-in-queue and volume on the major street and also stated that the HCM critical gap is based on the average driver. The results concluded that HCS underestimates delay while it overestimates critical gaps. This study demonstrates that as volumes increases at a TWSC intersection, the critical gap accepted by the average driver decreases. Review of the data shows that an average decrease of 0.007 s of critical gap will occur with a one vehicle increase in volume. Critical gap was assessed by a portion of the current exact systems like slack, Harders, twofold Logit, adjusted Raff and Hewitt strategies at unsignalized T-intersections by Ashalatha and Chandra [4]. The critical gap variety by these techniques features the ineptitude of the current strategies to address the blended traffic conditions. Along these lines, the creators figured a substitute system for critical gap estimation utilizing freeing conduct from vehicles related to gap acceptance information. Amin and Maurya [5] and Maurya et al. studied the crossing behaviour of drivers at uncontrolled intersections in India through gap acceptance technique. The authors used ten methods namely the Raff's method, Lag, Maximum Likelihood, Harders, Macroscopic Probability Equilibrium, Greenshield's, Logit, acceptance curve and clearing behaviour methods for estimating critical gaps for through movements from a minor road. Some of the conclusive remarks as per the authors are: The critical gap values obtained from all the nine methods were quite low compared to that estimated using the clearing behaviour method (developed for Indian mixed traffic conditions) which can also work well under homogeneous, under-saturated and over-saturated traffic conditions. The preciseness of capacity estimation depends upon the most realistic evaluation of critical gaps which was obtained using the clearing behaviour technique. From the analysis, driver's gender, age, speed of oncoming vehicles, waiting time and number of rejections were affecting two-wheeler gap acceptance. However, vehicle occupancy and waiting time did not affect three-wheeler gap acceptance. The authors also recommended conducting studies on two-stage gap acceptance at four-legged uncontrolled intersections in which limited research has been devoted. Tian et al. and Zhou et al. attempted to identify factors which affect gap acceptance of drivers turning left at unsignalized intersections. Technique employed for data collection was videography (five cameras were installed) at each of the six unsignalized intersections on both four and two-lane roads in the city of Connecticut, United States. The presence of left turn lanes (LTL) and both high and low speed limits were included in the data sets. Driver gap acceptance (i.e. accepted/rejected) was considered as a paired choice and corresponded Logit models were utilized to evaluate the likelihood of tolerating a gap. A portion of the discoveries from the eight models created in the examination are:

1. The displaying results demonstrated that nearness of LTL and the quantity of paths on the significant street has no huge impacts on the driver's likelihood of tolerating a given gap.
2. The impacts of speed limit (high > 45 kmph or low < 45 kmph) were excessively immaterial.
3. Female drivers are more moderate than male drivers in tolerating gaps which is shown by chances ratio of 0.55.
4. Age doesn't influence driver's gap acceptance.
5. The model with both the quantity of dismissed gaps and the mean interim of the dismissed gaps has the best model fit and was chosen as the gap acceptance model.
6. Likelihood of tolerating a gap raises as the two estimations of the variables increments.

Prasad et al. and Troutbeck and Brilon calculated capacity of three semi-urban unsignalized intersections on 4-lane roads in the city of Vishakhapatnam using the conflict technique. The conflict system depends on the scientific detailing of cooperation and effect between flows at an intersection. The authors also used the modified Tanner's formula for finding capacity of major and minor road right and left-turning movements as per the following Eq. (4):

$$C_{\mathrm{p}} = \frac{q_{\mathrm{M}}\left(1 - \lambda t_{\mathrm{p}}\right)\mathrm{e}^{-\lambda\left(t_{\mathrm{c}} - t_{\mathrm{p}}\right)}}{1 - \mathrm{e}^{-\lambda t_{\mathrm{f}}}} \tag{4}$$

where, $\lambda = q_{\mathrm{M}}/3600$ (veh/s)

t_{p} = minimum headway in the major traffic stream

t_{c} = critical gap in seconds

q_{M} = number of major stream headways

t_{f} = follow-up-time.

Some of the conclusions are:

1. The modified Tanner's model was appropriate that the HCM technique.
2. Conflict technique is quite simple in identifying the conflicting traffic flow in the HCM procedure.
3. After analysis the capacity as per the conflict technique, it was recommended by the authors that the intersections selected should be signalized.

Mohan and Chandra [1] and Asaithambi and Anuroop estimated critical gaps considering different vehicle types by different methods namely Maximum Likelihood Method (MLM), Clearing Behaviour Method (CBM), Ashworth Method, Greenshield's Method, Harder's Method and Raff's Method. In order to collect gap information accurately, authors devised a new reference line for realistic measurement of gaps. The study reiterated the fact that clearing behaviour method (CBM) of critical gap estimation under mixed traffic conditions in India. Parameswaran and Asaithambi estimated and compared capacity of two uncontrolled T-intersections in Mangalore and Calicut using the gap acceptance procedure (GAP) and the additive conflict flow technique (ACF). The authors used Chandra's method for dynamic PCU estimation in this regard. They estimated critical gaps for priority movements (minor RT, major RT and major LT) using Raff, Greenshield's, Ashworth's and Clearing Behaviour method. The authors compared the capacities obtained by using HCM field data, gap acceptance procedure (GAP) and Additive conflict flow (ACF) technique for all the three priority movements for both the cities. For both the cities, the ACF technique overestimated capacity for minor left turns at the T-intersections. The gap acceptance capacities were also found to be lower compared to those estimated using the HCM field data values. Rao et al. estimated critical gaps at three uncontrolled TWSC T-intersections in the city of Vishakhapatnam using Raff's, Harders, Wu and a newly proposed method by IIT Roorkee. The authors also estimated and compared capacity of major and minor right and left turns using the HCM technique for critical gaps estimated using the above stated methods. In addition, the authors compared control delay for the selected major and minor road movements along with a critical gap comparison using HCM TWSC control delay estimation technique. They also evaluated and compared the Levels of service (LOS) for both morning and afternoon peak hours using HCM level of service (LOS) estimation procedure. For determining the capacity, an occupation/service time of 3.6 s has been adapted in case of AWSCs. Earlier studies for single–lane AWSC intersections found that turning movements did not affect the occupation time significantly. Harders, Maurya and Amin, Ashalatha and Chandra [4] and Mohan and Chandra [1] estimated critical gaps and later then translated them in estimation of capacity at four Indian uncontrolled intersections using the Maximum Likelihood method (MLM), Harder's Method, Modified Raff's method, Logit (weighted linear regression) method and the Ashworth's method. The authors used a new concept of "Occupancy Time" (i.e. the time spent by a subject movement or vehicle type in occupying the intersection influence area) and used its theoretical background along with cumulative distributions of accepted gaps and lags to estimate critical gaps for the four intersections. The validation of this proposed method was demonstrated by the authors by comparing field capacity of non-priority movement at one of the intersections with the theoretical obtained capacity value. The Indian highway capacity manual (Indo-HCM 2018) in its first and recent publication has included a Chap. 6 on critical gap and capacity analysis of unsignalized/uncontrolled intersections exclusively considering Indian mixed traffic behaviour. In this manual for the first time, the occupancy time method (OTM) has been conceived for the calculation of critical gaps. Unlike other methods, OTM incorporates actual driver behaviour observed on unsignalized intersections largely. As such, OTM accounts for the actual clearing pattern of the conflict area and the traffic interaction that occurs within this region. Thereafter, the capacity for various movements observed at an unsignalized intersection is carried out through a series of steps which will be described in this section. The German capacity estimation procedure uses an array of curves for relating basic capacity with the conflicting volumes of major street traffic while the procedures use only a single set of curves for minor road traffic. The German procedure on the other had lends more attention towards consideration of major street vehicle travel speed for capacity estimation while HCM takes such consideration casually. Indo-HCM methodology estimates primarily capacity and LOS of three movements i.e. right turn from Major Street, right turn from Minor Street and through movement on Minor Street occurring on three-legged unsignalized intersections. Using the critical gaps for passenger cars (s), conflicting flow (PCUs/h) and assuming follow-up-time as 60% of critical gap, the capacity (in PCUs/h) for individual movements is estimated.

4 State-of-Art on Microscopic Traffic Flow Simulation Techniques for Performance Evaluation

Empirical, analytical or deterministic analysis for performance evaluation (capacity, delay and level of service analysis) demands extensive collection of geometric, traffic, road environmental, crash related and other important data which is time consuming, tedious, non-economic and skill-extensive. The word ‘traffic simulation’ can be broken down into ‘traffic’ and ‘simulation’. ‘Traffic’ represents the operation of united transportation modes under a definitive geographical boundary area. ‘Simulation’ is a way to recreate a previously formulated mathematical/physical identity by recreating evolutions over time periods with observing respects to certain performance parameters that can be used for ascertaining a real-world phenomenon. So, before simulating any phenomenon, a proper and scientifically proven mathematical model is always needed that will support the simulation process. Often it happens that all the models are not accurate and may be only a few can represent the correct natural physical conditions of the real-world phenomenon taking place on the spacio-temporal road spaces. According to George Box, ‘*All models are wrong, but some are useful*’. This is why, ‘Validation’, is an essential rule to be followed after ‘calibration’ of any simulation model to obtain absolute accuracy. Traffic simulation can be ‘*macroscopic*’, ‘*microscopic*’ and ‘*mesoscopic*’ in nature. The objective of macroscopic simulation is mainly spatio-temporal representation of three ‘macro’ elements, which is speed (x, t), flow-rate (x, t) and density (x, t) in terms of differential equations. Mesoscopic simulation is a combination of both “macro” and ‘micro’ aspects. When each individual vehicle unit in a microscopic model follows the simplification of a macro model and average speed of the route becomes as its measure of effectiveness, then it being governed as per mesoscopic ideology.

'*Microscopic simulation*' governs the movement of individual vehicle units based on their operational traceability and on the rules of lane changes [6]. Some of the time-step multi-modal microscopic traffic flow simulation systems like CORSIM (*Corridor simulation*), AIMSUN (within the simulation environment GETRAM-*Generic Environment for Traffic Analysis and Modelling*), MATSim (*Multi-agent transport simulation toolkit*), SUMO (*Simulation of Urban Mobility*), SiMTraM (*Simulation of mixed traffic mobility*), PARAMICS (*Parallel microscopic simulation of road traffic*), HCS (*Highway capacity software*), PTV-VISSIM (*Visual Simulation in English*) have been widely used for performance evaluation of uncontrolled intersections by most of the transportation professionals. The expert's opinion (from different European, middle-eastern, American, Australian and African origins) has proclaimed VISSIM to obtain consistent and precisely allied convergent results even under traffic heterogeneity. Creation and customization of complex priority rules (through easy positioning of conflict pointers), dynamic vehicle routing decisions, varying driver behaviour attitudes, varying compositions of traffic heterogeneity, smooth and effortless simulation runs and designs, variant types of outputs and analysis makes this multi-modal marvel to vary much favourable for most of the researchers in developing Asian countries like India, Bangladesh, Nepal, Indonesia, Malaysia, Tehran, etc. In this paper, performance evaluation is broad term used for capacity assessment, service/control/stopped delay analysis, level of service analysis, level-of-safety analysis, crash predictions at minor and major roads of intersections, travel time comparison. Some of the primary uses of micro-simulation are:

1. Evaluation of traffic congestion for different traffic facilities.
2. Expressing and designing complex geometric configurations.
3. Studying impacts from generating traffic and design improvements that are beyond limitations of other tools.
4. Micro-simulation can be classified based on some broad scientific fields of operation in traffic flow theory, operations and engineering.
5. Microscopic modelling of traffic itself based on traffic flow theory.
6. Computational physical relation developments in either regional or municipal divisions.
7. Agent-based or microscopic demand behaviour modelling which combines models from evolutionary programming, game theories, sociology and other multi-agent systems [7] which include Cellular Automata and Gravity-approximation based physical models.
8. Co-evolutionary algorithms that can explain complex adaptive systems based on some special interaction scenarios occurring due to collective decision-making processes.

Tan and Tufo presented a detailed methodology for capacity analysis procedure for two-way yield or stop-controlled unsignalized intersections in Switzerland. According to the authors, the objective of capacity analysis is to estimate available capacity and capacity used as a percentage of actual capacity which is the main intersection operation indices. According to the authors, the capacity of an intersection entry or intersection entry lane is defined as the maximum inflow of the entry or the lane concerned while the capacity used is the ratio of the traffic volume to the corresponding capacity. This procedure was based on determination of critical gaps and conflict traffic volume at yield-controlled and right-side priority-controlled intersections. The authors also calculated the traffic disturbance at entry lanes of the intersections as a function of theoretical capacity and real traffic volume. Some of the main factors to be considered while enhancing capacity of unsignalized intersections are:

1. Number of conflicting streams
2. Conflict Area
3. Total Conflicting Traffic Volume
4. Conflict Angle and its type
5. Storage space for traffic queue (non-priority movements)
6. Levels of priority.

Following are the tentative results from capacity estimation procedures as per the program developed by the authors:

1. Entry capacity (vehicles per hour)
2. Reservation of entry capacity (vehicles per hour)
3. Entry capacity used (%)
4. Entry capacity used at conflict points (%)
5. Average queue length (in terms of number of vehicles)
6. Average delay (seconds per vehicle)
7. Variation in the entry queue (vehicles in every 5-min intervals)
8. Variation of entry delay (seconds per vehicle in 5-min intervals).

Wu et al. [8] examined traffic flow at unsignalized T-intersections in which three info bearings of vehicles and two right turning and one remaining turning utilizing the cellular automata models defined by Nagel and Schreckenberg, Esser and Schreckenberg [7] CA traffic model. The creators additionally examined associations between vehicles on various paths and impacts of traffic flow conditions of various streets on capacity of T-formed intersection framework. The outcomes demonstrate that this model can be applied to genuine traffic examination and traffic conjecture. To put it plainly, the model thinks about the variety of vehicle speed and uses transient (time) gaps rather than unique gaps to figure out which vehicle is permitted to pass the intersection. Illustration of the T-intersection and the traffic is appeared in the figure beneath. The paths are separated to CA cells. Each vehicle takes a solitary cell. The lengths of path 1–5 and 2–4 both $L = 1999$. The lengths of path 3 and 6 are $L3 = L6 = 500$. The intersection situates at the centre purpose of path 1–5 ($b = 1000$). The conflicts at the defining moment are taken care of with observing standards:

1. The conflict of vehicles on path 1 with the left-turning vehicles on path 2.
2. The conflict of straight guiding vehicle on path 1 with the right turning vehicles on path 3.
3. The variety of intersection capacity is dictated by the net augmentation or decrement of transition on all the three paths.

4. The expansion of information transition on path 2 abatement the motion on path 1 yet builds the motion of path 3.
5. The increase of input flux on lane 3 will decrease the flux on lane 1 and lane 2.

Fellendorf and Vortisch [9] and Tian et al. explained along with visualizations and applications of a microscopic, conduct based multi-reason traffic simulation programming named as VISSIM. The investigation displayed a survey on the history, run of the mill applications and is trailed by demonstrating standards introducing the general engineering of the test system. The creators additionally exhibited a few systems to calibrate the traffic flow models with comments of interfacing VISSIM with different instruments. VISSIM is a microscopic, discrete traffic simulation framework demonstrating motorway traffic just as urban traffic operations which depend on a few numerical car-following and path changing or horizontal driving models. The option to proceed for non-signal-ensured conflicting developments is demonstrated with priority rules. Priority rules are utilized to display uncontrolled intersections where traffic needs to offer approach to traffic on left, uncontrolled intersections where traffic on the ending street must offer approach to traffic on the proceeding with street and TWSC/AWSC intersections. The priority controls in VISSIM comprises of a stop line showing a sitting tight situation for vehicles of minor developments. At stop line the minor vehicle will check if a vehicle of the significant development is inside the headway territory. The headway region is characterized as a portion beginning marginally before the two developments combine. In VISSIM, this position can be set physically. Also, the minor vehicle checks if a significant vehicle will arrive at the conflict marker inside the base gap time if going with its present speed. Holik et al. [6] developed a universal simulation model of three T-intersections based on gap acceptance theory using a web-based software Witness. Four levels of priority based on movement ranks (Rank 1, 2, 3 and 4) as per HCM were fed in the simulation module. This web-based application for input data enables to set almost all attribute values. Secondly, using the application does not require a user licence. The script is based on UNIX systems and each user gets his/her login information. The web application consists of following bookmarks:

1. Input data—Traffic intensities, basic parameters, direction parameters and lane settings.
2. Simulation—A data consistency test runs and the simulation run is generated.
3. Output data—Consists of two sub-pages (Queue Lengths and Waiting Times).
4. Settings—Enables to edit basic information about the user (email, password).
5. Help—Enables to provide information regarding software usage.
6. Sign out—Enables user to sign out of the web application.

5 Conclusion

A total 59 (30 from developed nations having homogenous traffic flow conditions and 29 from developing nations having heterogeneous traffic flow conditions) number of previous literatures have been investigated for preparing this state-of-art review. The literature was classified as for Class-I: {*machine learning; data mining; soft computing including safety assessment using surrogate measures*} techniques for performance evaluation and Class-II: {*microscopic traffic flow simulation-based*} performance evaluation for uncontrolled intersections operating under both heterogeneous and homogenous traffic flow conditions. The first classification had 5 research lines with 14 sublines and the later one had two research lines with six sublines. Performance evaluation is broad term conglomerating operational analysis, control environmental assessment, safety evaluation and many more intricacies that are normally missed out while talking about uncontrolled intersections. Capacity, delay, level of service, queue lengths and travel time are the major parameters which are often focused upon while assessing uncontrolled intersections. Other than these, agent-based uncontrolled intersection priority models, object oriented delay models, traffic assignment-based delay models, game theory-based uncontrolled driver behaviour control concepts and some more are of quite interest when performance evaluation is thought of. Gap acceptance studies are performed frequently either through simulations, analytically or through deterministic procedures but there are certain other deterministic techniques which have been identified from this review that are presently gaining priority. Modified classical Cellular Automata (CA) models (like NaSc or Nagel-Rickert's) can be used effectively to define uncontrolled intersection movements and trajectories. Artificial Neural Networks (ANN) and Adaptive Neuro Fuzzy Inference (ANFIS) can be used effectively for checking and validating different methods for performance evaluation of uncontrolled intersections with relatively high degree of accuracy. Crash data prediction using surrogate safety measures (PET, speed, deceleration rates and conflict probability) and incorporation of these parameters in microscopic simulation frameworks is always a step ahead in governing performance criteria. Support Vector Machines (SVM), decision trees and random forest models are gaining priority over statistical techniques (like Binary logit; regression-based) estimations for identifying driver decision making. Microscopic simulation packages like VISSIM, FLOWSIM, AIMSUN, SiMTraM, PARAMICS, SUMO and web-based models like Witness are some of the most dominant tools which have been used for safety, operational and environmental evaluation of uncontrolled intersections under traffic heterogeneity. Calibration and validation of the driver behaviour models (car-following, lane changing, lateral driving) is an important factor while simulating and should be aided with machine learning techniques or by using proper field estimated values. Out of the micro-simulation models, PTV-VISSIM which a time-step multi-modal framework has been used widely for calibration and validation of performance and safety models under heterogeneous traffic flow conditions rather other prevalently used tools like AIMSUN or HCS. VISSIM uses the Weidmann car-following driver behaviour model which is psychophysical framework and considers the user's behaviour attributes and therefore can be calibrated dynamically using neural networks or fuzzy principals. Microscopic simulation parameter sensitivity analysis either statistically (using ANOVA) or by using data mining tools like neural networks, genetic algorithm is gaining importance in the recent times. The studies depicted in this paper have been conducted under both heterogeneous and homogenous vehicle compositions with differing geometric environments (for standardized and non-standardized situations) with varying roadside and environmental conditions. Therefore, the tools or techniques described here have not been widely utilized in assessing capacity, delay and level of safety at uncontrolled intersections in a generalized traffic flow situation arising in mid-sized cities or rural areas of Indian urbanized land uses. Other than computer simulations, optimal automatic calibrated driver simulators can also be utilized for studying the road-user behavioural effects on capacity and LOS analysis for further performance investigations of uncontrolled intersections under various traffic flow conditions. Some of the studies from homogeneous traffic prevailing countries can even be replicated for heterogeneous conditions and can be compared using the tools portrayed in this study. Keeping in mind these statements, the authors would like to conduct experimental investigations using some of the concepts, methods and soft computing tools as a part of their future work towards uncontrolled intersection performance evaluation under Indian mixed traffic conditions.

References

1. Mohan M, Chandra S (2018) Three methods of PCU estimation at unsignalized intersections. Transp Lett 10(2):68–74
2. Khattak JA, Jovanis PP (1999) Capacity and delay estimation for priority unsignalized intersections: conceptual and empirical issues. Transp Res Rec (TRR) 128(7):129–137
3. Denyer D, Tranfield D (2006) Using qualitative research synthesis to build an actionable knowledge base. Manag Decis 44(2):213–227
4. Ashalatha R, Chandra S (2011) Service delay analysis at TWSC intersections through simulation. KSCE J Civ Eng 15(2):413–425
5. Amin HJ, Maurya AK (2015) Modeling the gap acceptance behavior of drivers of two-wheelers at unsignalized intersections in case of heterogeneous traffic using ANFIS. In: 3rd international conference on advanced computing, networking and informatics. Smart innovation, systems and technologies. Springer, New Delhi, pp 533–543
6. Holik J, Dorda M, Teichmann D, Graf V (2016) Universal simulation model for unsignalized intersection capacity analysis. In: 17th international carpathian control conference (ICCC). IEEE, Czech Republic, pp 236–241
7. Esser J, Schreckenberg M (1997) Microscopic simulation of urban traffic based on cellular automata. Int J Mod Phys C 08(5):1025–1036
8. Wu QS, Li P, Hu I, Jiang R (2005) Study of traffic flow at an unsignalized T-shaped intersection by cellular automata model. Eur Phys J B 48(2):265–269
9. Fellendorf M, Vortisch P (2010) Microscopic traffic flow simulator VISSIM. In: Barcelo J (ed) Fundamentals of traffic flow simulation, chap 2 (145). Springer, Heidelberg, pp 71–72

Assessing Spatial Accessibility to Amenities, Facilities, and Services from Public Transit Using Non-motorized Transport: A Case of Nagpur City

Pavan Teja Yenisetty, Jenil John, and Pankaj Bahadure

1 Introduction

Theories of new urbanism and smart growth usually point at accessibility as one of their most significant principles [1]. As per transportation research board, mobility and accessibility have been identified as very crucial issues. Therefore, the integration of transportation and land use planning is repeatedly emphasized. Researchers have called for a paradigm shift from "auto mobility-oriented planning" to "accessibility-oriented planning" [2]. In most of Indian cities, accessibility criteria are neglected while preparing for transportation plans [3]. The plans emphasize more on mobility planning and motorized transport only. Mobility is an important factor in urban areas, but an approach based on mobility creates sprawl [4, 5]. Particularly, when it relates to lower-income households, their daily lives are largely dependent on non-motorized transport. Even after investing enormous capital and infrastructure for public transit, NMT is often used as a significant choice that influences overall accessibility and affects connectivity in the last mile [5, 6]. Planning practice in Indian cities involves land use planning as a different activity without considering its impact on transport [7], and transport plans are drawn up separately from land use plans [8]. These planning approaches resulted in the inefficient use of available resources. So, Government of India (GoI) promoted integrated land use and transportation planning in all Indian cities and provided guidance through National Urban Transportation Policy (NUTP) [9]. NMT is an essential element of urban transport whose two major modes, i.e., walking and cycling play a dominant role in terms of mobility in both the developing and developed countries [10]. Asian cities because of their compact, dense, and high mixed land use characteristics are well known for a high share of non-motorized trips both in low and high-income cities [4]. Several cities in

P. T. Yenisetty (✉) · J. John · P. Bahadure
Department of Architecture and Planning, Visvesvaraya National Institute of Technology, Nagpur 440010, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_16

Japan, China, Vietnam, India, Bangladesh, Indonesia, and Nepal still rely heavily on these modes as their share of trips ranges from 25 to 80% [2]. It is understood that there are a significant number of trips made by foot for reaching services in Indian cities [8]. So, focusing on this issue will help in improving NMT infrastructure and integrating of various services and facilities to reach public transit in a better way. Even policies in India now emphasize the possibility of maximizing people and activities by providing better linkage to public transport (bus stops) to all kinds of service [9]. They even target to improving better cycling and walking infrastructure and increasing in network coverage of public transport in the city [10]. Cycling can be limited for shorter lengths, but it is a very flexible in trip timing and destination whereas public transport can cover longer distances with high capacity but it is not flexible like NMT as it has fixed timings and destinations. So, there are strong achievements to be made out of combining the strengths of both NMT and public transport from the perspective of an individual traveler to overall transport system [11].

1.1 Relevance of the Study

Few developing nations use tools and instruments to measure accessibility to services, jobs at walkable distance from public transport (SNAMUTS) in Australia, spatial network analysis for multimodal urban transport systems (SNAPTA) [12], etc., are tools which contribute to the identification of barriers and create accessibility index to integrate NMT with public transport. Every journey begins or finishes with a walking share, i.e., NMT. In India, walking and biking mode shares are relatively much higher. But supporting infrastructure and environment are not created for them which declines the mode share from the past decades. In the current scenario, government started implementing sustainable practices in transportation due to the impacts increasing motorization rate, traffic, and parking congestion which result in environmental pollution, drastic climate change, and health patterns. NMT is the best sustainable mode because they provide mobility and support to other modes, resulting in increased accessibility. Improving walking environments can increase transit accessibility, since most of the transit trips consist of walking portion [11]. In Indian context, there are few cases of measuring accessibility using NMT in neighborhood level [13]. Mostly, issues are mainly concerned about aspects of traveling mode, time, and problems related to infrastructure (pavements and foot paths) [14, 15]. But the studies addressing on connectivity of AFS with the public transportation is not focused. So, the aim of this paper is to introduce simple measure for accessing amenities, services, and facilities (AFS) from public transport at macro-level and micro-level using NMT and addressed issues of connectivity and infrastructure.

2 Measures and Indicators

Traditionally, there are various approaches to measure accessibility related to NMT and public transport like cumulative measure, gravity measure, etc. [16–18]. Studies categorized these approaches into five types which include components of distance, travel time, cost, and movement over space and selection of mode [19]. This paper adopts distance measure as this measure estimates the distance between two places, i.e., from AFS to public transit [20]. Both gravity and cumulative measures are not selected because this study do not deal with trip attraction and the number of opportunities available in a given zone [21, 22].

There are different types of distance calculation measures such as (1) Euclidean distance (e.g., through buffering), (2) Manhattan distance, (3) network distance (closest path), and (4) Minkowski distance method [21]. Studies show Euclidean distance and network distance are suitable in context to accessibility measures [23]. Researchers have used walking distances, i.e., 400 and 800 m, and even the World Bank indicates that public transit facilities for less than 400 m should be provided to encourage pedestrian access [23–25]. In case of India, urban and regional development plans formulation and implementation (URDPFI) guidelines [24] consider 400 m walk and 600 m radius access to urban facilities and public transport. In this paper, spatial accessibility to amenities, facilities, and services to public transit using non-motorized transport from the various literatures [25, 26], parameters, and their indicators [27, 28] are identified for measuring accessibility in a macro-level as well as micro-level [29, 30] as shown in Table 1. Distance measures are carried out for every 400 m as shown in Fig. 1.

3 Study Area and Data Sources

For this study, Nagpur city is selected that lies precisely at the center of the country (Fig. 2). The total area under development plan 2000–2011 of Nagpur is 235.21 km^2 and consists of the area under Nagpur Municipal Corporation (NMC). The city area is divided into 136 wards; population density of the city is 10,873 persons/km^2 as per 2011 census [19]. Only 12% of Nagpur's commuters use public transport, while 35% use two-wheelers and a significantly high proportion of individuals uses NMT. Study reveals that Nagpur's share of non-motorized mode (walk, bicycle, and cycle rickshaw) accounts for 58% and share of motorized modes is 42%. Almost 35% of worker's travel takes about 15 min. The average travel distance in the city is 4.6 km [31]. Now presently, metro rail project is under implementation. As per mobility plan, city targets to achieve 30% of mode share using public transportation [32] and mentions to improve NMT to support PT. For this study, data from various sources are collected and mapped in Google Map API as shown in Table 2.

Table 1 Selected measures and parameters

	Parameters	Indicators	Measures
Macro-level accessibility	Land cover	Builtup	The average percentage of builtup in grids
		Number of opportunities amenities, services, and facilities (AFS)	Number of opportunities present
	Network	Road network	Length in meters
	Mobility	Availability of public transit	Number of bus stops/stations
	Walkability/bicycling	Proximity	Distance between public transit from AFS (length in meters)
Micro-level accessibility	Land use	Land use mix entropy	Percentage of land use by area
		Number of opportunities amenities, services, and facilities (AFS)	Number of opportunities present
	Network	Road network	Length in meters
		Road density	Total road length/total area
	Mobility	Availability of public transit	Number of bus stops/stations
	Walkability/bicycling	Proximity	Distance between public transit from AFS (length in meters)
	Density	Built density	Ratio of number of built units to total area
		Residential density	Ratio of number of dwelling units to total area
		Commercial density	Ratio of number of commercial buildings to total area
	Infrastructure	Pedestrian/cycle pathways	Length in meters
	Safety	User willingness	Score

Selection of divergent opportunities will make it possible to emphasize accessibility plans [33, 34]. So, this study adopts various AFS in the macro- and micro-analysis as shown in Table 3.

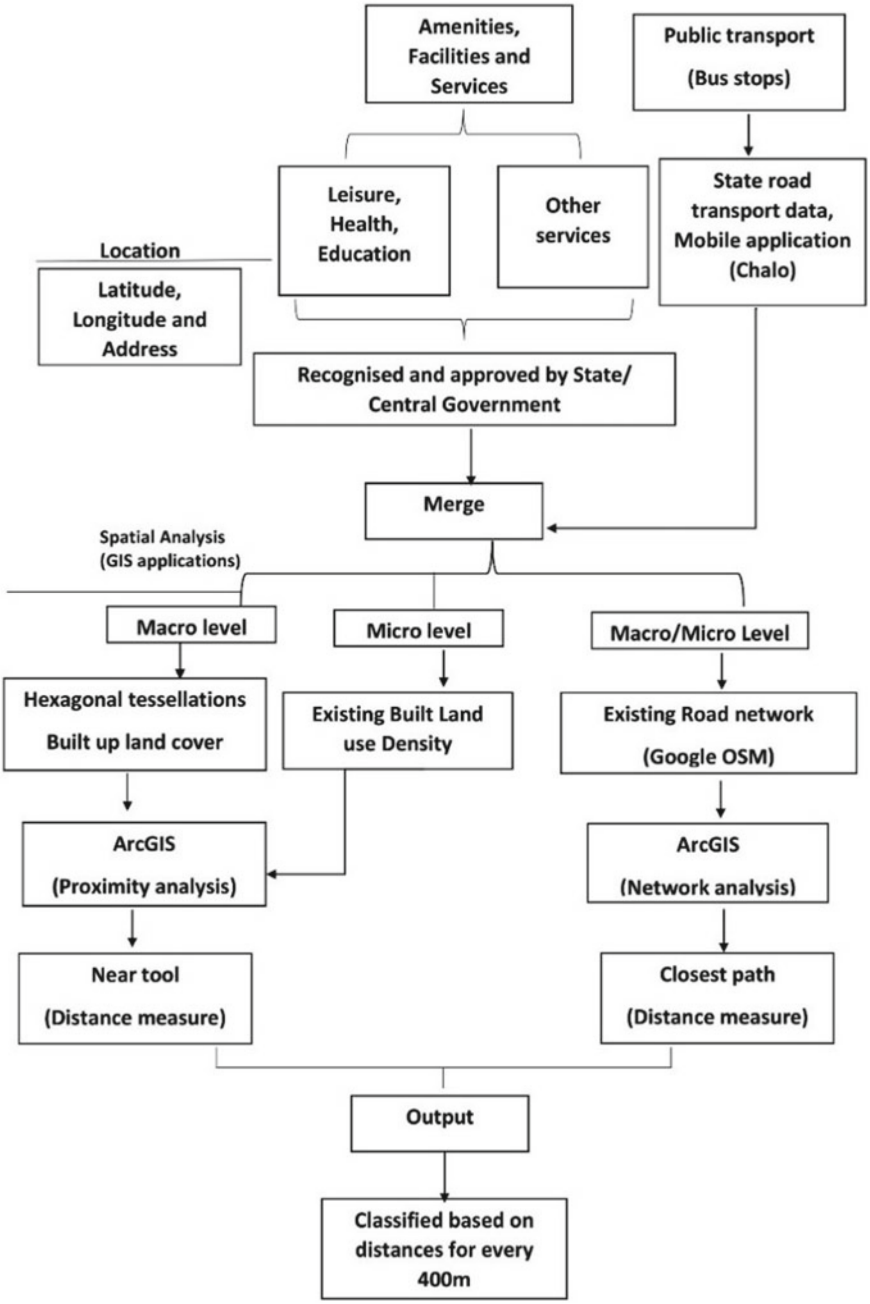


Fig. 1 Method adopted in GIS

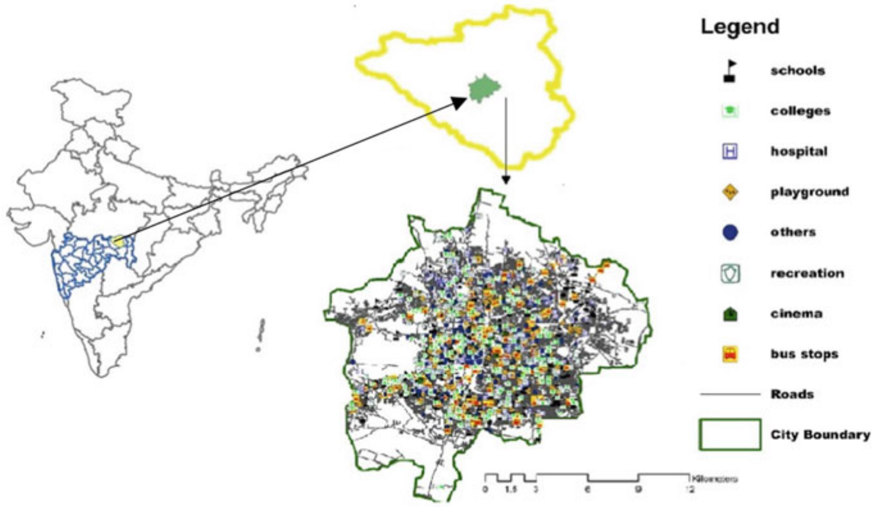


Fig. 2 Location of Nagpur city

Table 2 Data types and its source

S. no.	Data type	Source (Nagpur)
1	City base map	Nagpur improvement trust
2	Boundary limits	Prepared by the author from Nagpur municipal corporation (NMC) Website
3	Bus stop location	Prepared by using the information available from operator city bus service, head office (star bus) NMC, Chalo mobile application
4	List of hospitals	Prepared by using information from NMC health office (Google Map API)
5	List of education Institutes	Prepared by using information from Nagpur divisional education board (Google Map API)
6	List of parks/gardens and other facilities (police station, post office, community halls)	Information collected from land use maps (city development plan), (Google Map API)
7	GIS map	Maharashtra remote sensing application center
8	Land cover map	Extracted from USGS LandsatLook
9	Metro station location	Nagpur metro rail corporation

Table 3 Identified number of AFS in Nagpur city

Sector	Identified places (mapping)		Identified in Google API
Amenities	Leisure	Recreational Playgrounds Cinema hall	134 76 19
Facilities	Health	Hospitals Clinics Dispensary	275
	Education	Primary/secondary schools	178
		Colleges	198
Services	Other services	Post offices Police station Community halls	139

4 Data Analysis

4.1 *Macro-level Accessibility Analysis*

This study uses a grid-based measuring technique in spatial levels for Euclidean measurement (near distance) as grid assessment does not follow the macro-level boundary profile of roads and terrains [36]. The urban area is converted into a hexagonal grid of side 200 m covering 103,923 m^2 (10.39 ha) with 400 m between the hexagonal centers. The <lat, lon> of these AFS are entered in Google API. Initial distance was taken from the bus stop/ metro station to the closest grid and based on the distance a specific color was assigned. The color coded grid is overlapped with the existing built land cover extracted from Landsat image for Nagpur scene ID LC81440452018325LGN00, path 144/45 having spatial resolution of 30 m as shown in Fig. 3. It is designated as how much builtup parcel of land is in high accessible region and how many services and facilities are presented in that range (Fig. 4).

In this study, the levels of accessibility indices for macro-level assessment are consider for 400–2400 m and more than 2400 m of range are recognized as unreserved area. These levels of distance range are developed based on literature taking the minimum walking distance. For example, the distance to the closest public transit from each recreation point is measured. As per the distance, the symbol and color are designated to each and every point as shown in Fig. 5. This process is further repeated to all ASF.

For calculating distance from the network, one-way constraints were ignored, while at intersections, U-turns were permitted because mode was not considered. Based on distances between 400 and 2400 m, different color codes were used as shown in Fig. 6. Since the network data from open street map (OSM) are not accurate, basic checks and edits are being carried out to OSM's road network. Main roads for trucks that are not suitable for sidewalks or footpaths are removed. Each access point of the AFS is also verified by physical survey.

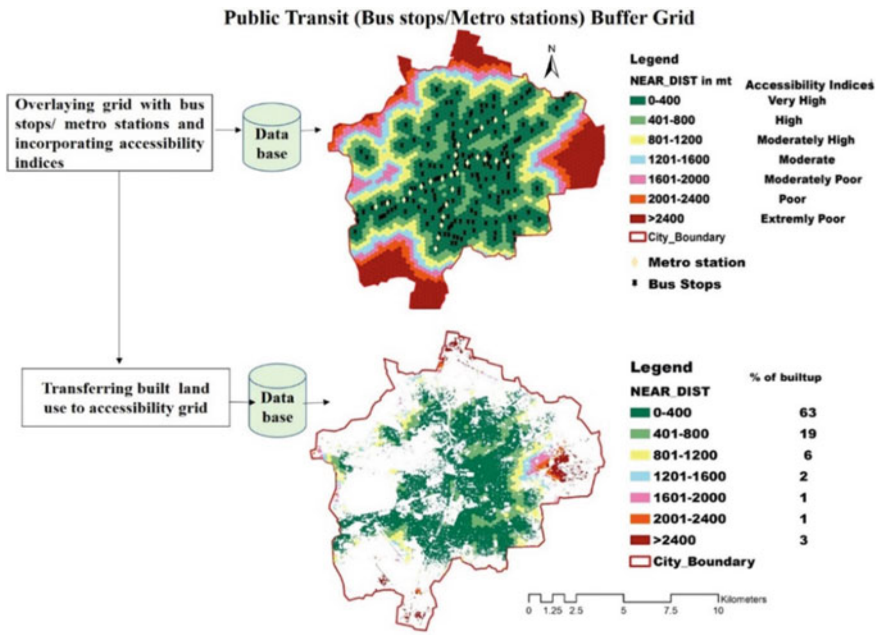


Fig. 3 Data preprocessing

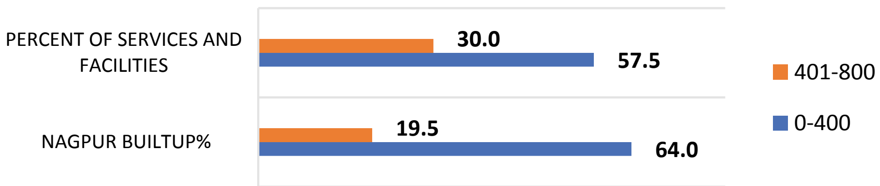


Fig. 4 Nagpur builtup % to facilities and service %

It is difficult to compare the outcomes of accessibility measures as large data are involved and account in aggregation errors [35]. So while taking the location, we choose the closest access points to overcome the negative errors [23]. To understand the difference of near and network distance, descriptive statistics were carried out in statistical package for social sciences (SPSS) to observe the difference in distance for near and network of each AFS in Nagpur (Table 4).

Careful observation of the area reveals that the hospitals and cinema halls are having less difference between near and network distance. Therefore, the immediate access is highly possible for public transit without many barriers which is a good scenario. But the maximum range limit of colleges is more than 3000 m. This means few colleges are located very far away from the public transit.

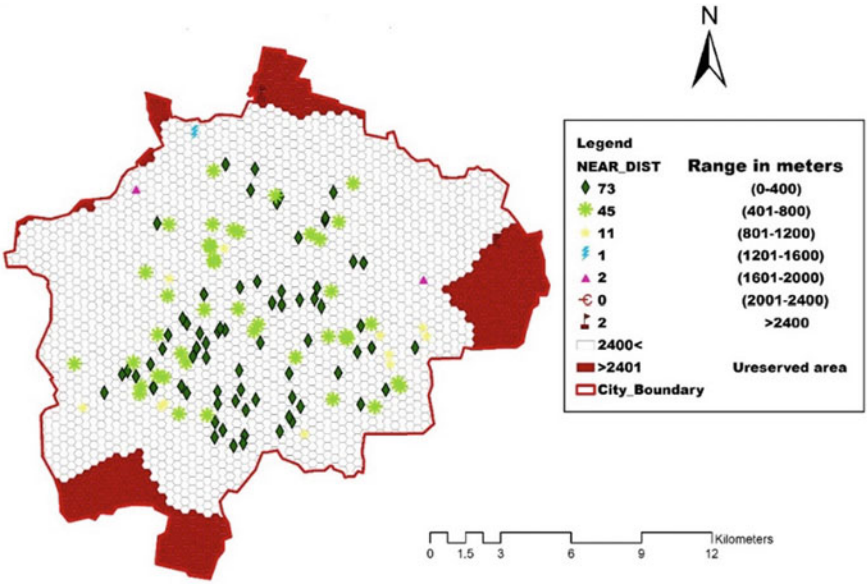


Fig. 5 Near (Euclidean) distance to recreation from public transit (city level, Nagpur)

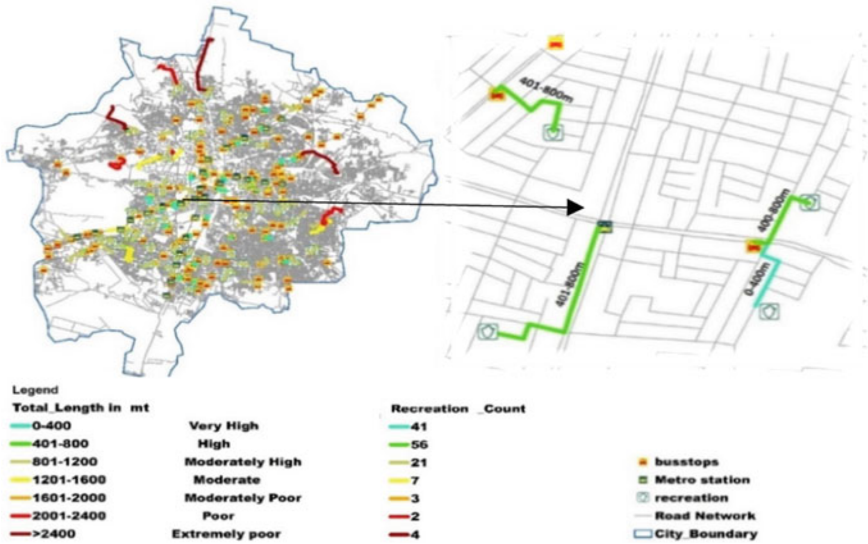


Fig. 6 Nagpur network distance to recreation from public transit (city level)

Table 4 Descriptive statistics to compare the difference of near and network distance for all AFS

AFS Nagpur	Min	Max	Range	Mean	Std. Dev.
	All distance is in meters				
Recreational	3.7	1500	1496	258.5	271
Schools	1.2	1451	1450	252.8	281
Colleges	3.5	3630	3626	255.0	351
Hospitals	2.4	1466	1463	173.8	207
Other Services	2.6	1047	1045	169.6	183
Playgrounds	7.6	866	858	241.6	206
Cinema halls	3	352	348	139	100

As the study was conducted within the municipal boundary limits, the primary identification percentage of AFS is accessible at different levels for each sector (Table 5). Nearly, 55% of AFS are within 0–400 m from public transit when measured by near distance. But the distance through network is doubled (Fig. 7). This observation confines the scope for really identifying the local barriers and network issues.

Table 5 Near and network distance percentage of AFS for each sector

AFS	0–400 m		401–800 m		801–1200 m		1201–1600 m	
	Near	Network	Near	Network	Near	Network	Near	Network
	Percentage accessible							
Recreational	54.4	30.6	33.5	41.7	8.21	15.67	0.75	5.22
Schools	56.1	36.5	30.3	38.2	7.3	12.92	3.37	5.06
Hospitals	55.5	31.8	33.3	36.3	7.58	21.72	3.54	3.03
Colleges	60.7	41.4	30.1	40.3	3.27	10.91	2.55	4
Other Services	63.3	43.8	25.9	34.5	6.47	13.67	2.88	2.88
Playgrounds	52	26.3	34.2	47.3	9.21	13.16	1.32	9.21
Cinema halls	56.5	39.1	26.09	34.78	-	8.7	-	-

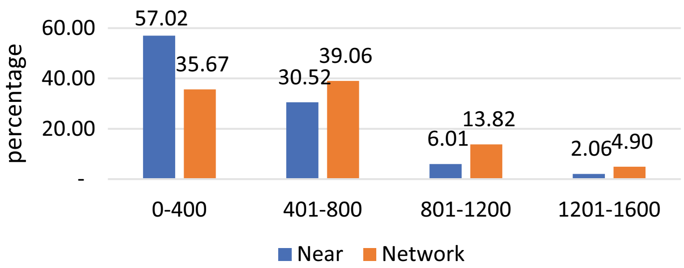


Fig. 7 Overall percentage of AFS for near and network (macro-level)

Especially the access to colleges and playground, the connectivity to public transit is very poor compared to other AFS.

4.2 Micro-level Accessibility Analysis

Criteria for selecting the areas for micro-level accessibility analysis are the presence of public transit stations, mixed-use character, and nodes which have large catchment areas. According to these criteria, two public transit nodes are selected, namely Shankar Nagar from the west and Itwari telephone exchange from the east (Fig. 8). Buffer area of 400 and 800 m is considered around the Shankar Nagar square and Itwari telephone exchange metro stations for micro-level accessibility analysis. Potential destinations like schools, colleges, market, shops, public semipublic buildings, parks/ playgrounds, hospitals, post offices, community hall available in these areas are mapped through built use survey (Fig. 9). An extensive household survey is conducted to know the existing NMT mode share, their trip purpose, and willingness to use NMT. It is identified that Itwari telephone exchange has higher residential, commercial, and mixed-use compared to Shankar Nagar square area, but recreational areas and public, semipublic buildings are less in telephone exchange (Table 6).

From various literature, several parameters are identified for micro-level accessibility analysis which is already mentioned in Table 1.

Land use mix entropy

Measuring land use mix is essential for assessing accessibility to AFS using NMT. A higher mix of land use is suitable for pedestrian and cyclist as AFS will be nearby. Land use mix can be identified from the entropy index developed by Cervero [36], which takes into consideration the relative proportion of two or more kinds of land use within the region. The entropy index ranges between 0 and 1, where closer to

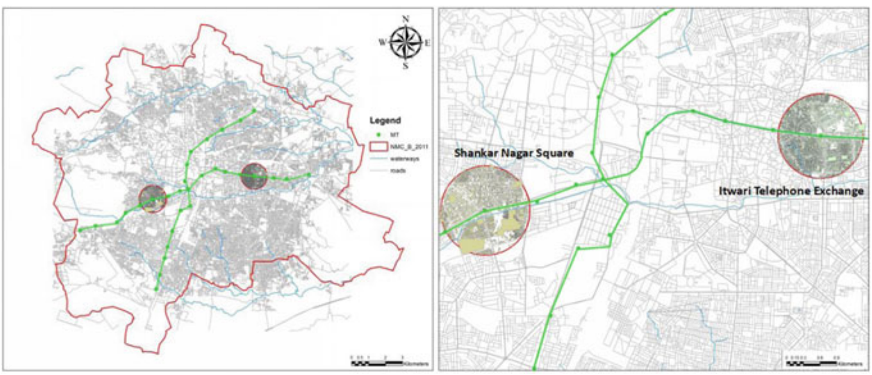


Fig. 8 Selected two different metro station and its 800 m buffer area

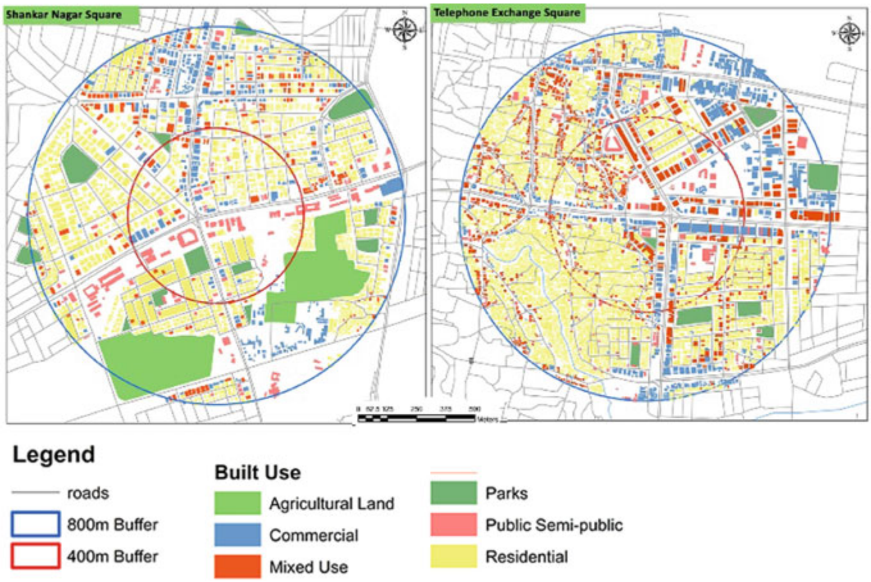


Fig. 9 Built use survey of Shankar Nagar Chowk (left) and Itwari telephone exchange (right)

Table 6 Comparison of built use percentage of both area

Land use category	Built use %	
	Shankar Nagar Chowk	Telephone exchange
Residential	43	50.6
Commercial	11	19.2
Mixed use	7	19.7
Public and semipublic	7	5.1
Recreational	7	5.4
Agriculture, water bodies, and special areas	25	–
Total developed area	100	100

value 1 is observed as a high mix. In Eq. 1, let P_i be the proportion of each form of land use i in the region and let j more than or equal to 2 be the number of kinds of land use i.

$$\text{Entropy Index} = (-1) \times \sum_{i=1}^{j} [P_i \times \ln(Pi)/\ln(j)] \tag{1}$$

From the entropy index calculation of both Shankar Nagar and Itwari telephone exchange in Table 7, it is observed that both areas have higher land use mix and they have the potential for non-motorized transport.

Table 7 Land use mix entropy index

	Land use mix entropy index	
	400 m	800 m
Shankar Nagar Chowk	0.76	0.78
Telephone exchange	0.77	0.80

Density

Increased density and clustering of activities tend to improve accessibility. Shorter travel distances can encourage NMT, which in turn increase the accessibility to AFS. Built density, especially residential and commercial density, and road density are calculated. It is observed that densities in the 400 and 800 m buffer of Itwari telephone exchange are comparatively higher than Shankar Nagar inferred in Fig. 10. This shows that both pedestrians and cyclists have more potential in Itwari telephone exchange than in Shankar Nagar, which increases the accessibility to AFS from public transit using NMT.

Figure 11 shows the road density for the selected area, and it is found that road density is higher in Itwari telephone exchange compared to Shankar Nagar. But road density in the 800 m buffer of Itwari telephone exchange metro station is observed to be lesser than in its 400 m buffer. This shows that the road network in the 800 m buffer has to be improvised in order to increase accessibility to AFS from public transport using NMT.

Proximity (near) and network analysis

All the amenities, services, and facilities along with the public transit stops and existing road network are identified and mapped. Proximity analysis and network analysis using Euclidean distance are carried out for both the selected areas (Fig. 12). From this analysis, it is found that two "recreation" facilities in Shankar Nagar and one "other" facility.

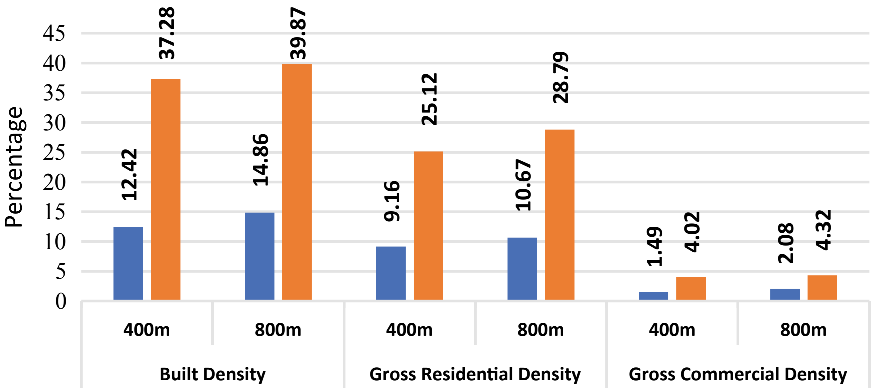


Fig. 10 Various densities

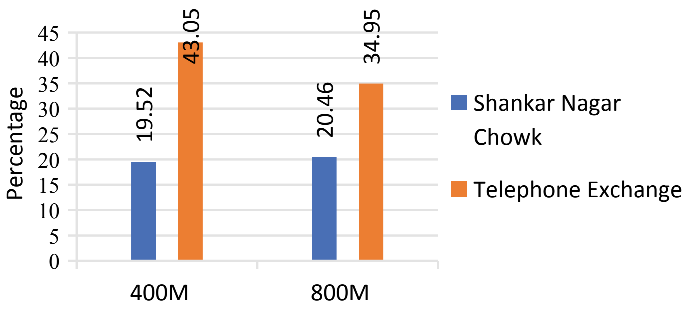


Fig. 11 Road density

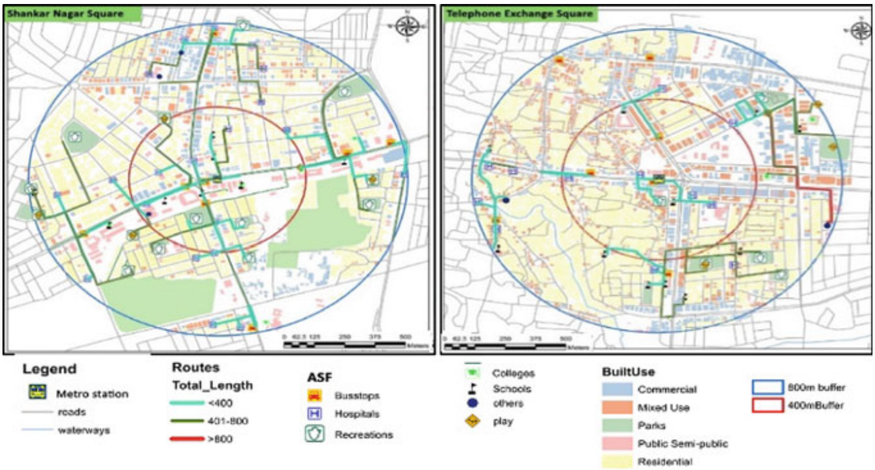


Fig. 12 Near and network analysis carried out in both area

in Itwari telephone exchange require more than 10 min walking to access from public transit. Otherwise, the network is fine, since all other AFS lies under 800 m distance from public transit (Fig. 13).

Household Survey

Household survey results give the understanding of infrastructure and safety parameters for micro-level accessibility (mentioned in Table 1). Extensive 100 household surveys are carried out and found that middle age and younger age population are higher in both the buffer areas around the selected metro stations. It is observed that Shankar Nagar is dominated by high- and middle-income groups, whereas Itwari telephone exchange is dominated by middle- and lower-income groups.

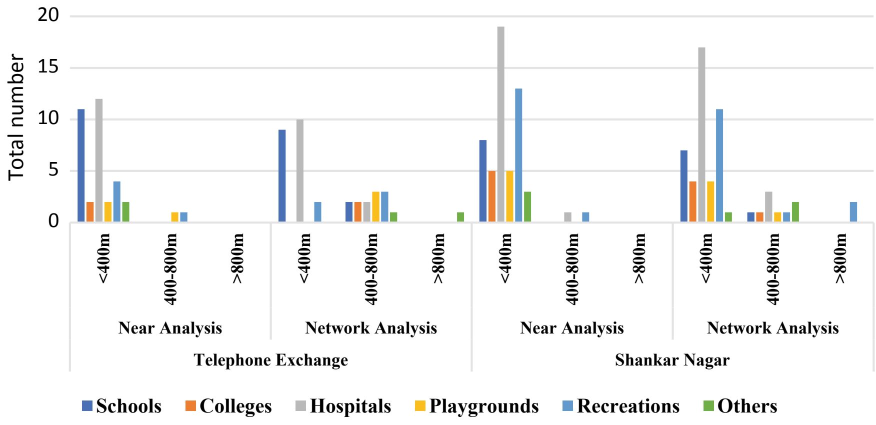


Fig. 13 Near and network distance analysis in micro-level

Existing Mode Share

Existing mode share at both the locations (Fig. 14) shows that they use both walking and cycling for shopping, reaching facilities, services, and leisure purpose. Mode share of NMT is comparatively higher in Shankar Nagar than in telephone exchange, due to the availability of footpaths and better infrastructure in the Shankar Nagar. But for education purpose, the share of cycling is higher in Itwari telephone exchange

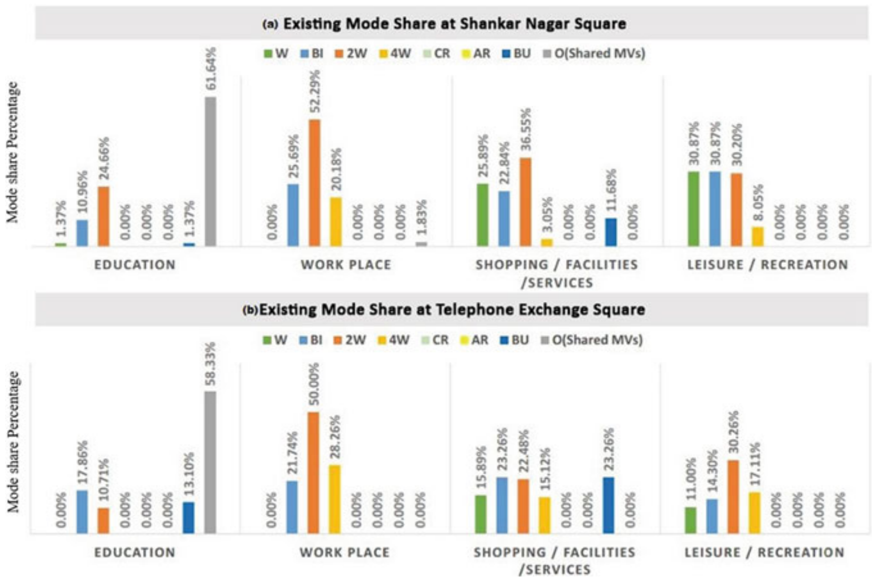


Fig. 14 Existing mode share at **a** Shankar Nagar square and at **b** Itwari telephone exchange

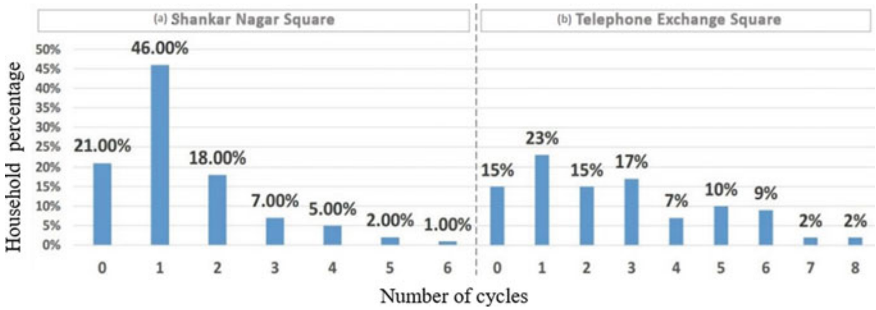


Fig. 15 Cycle ownership at **a** Shankar Nagar and **b** Itwari telephone exchange

than in Shankar Nagar which can be related to their income wherein they prefer NMT for going to school, and preferred schools are also nearby.

Cycle ownership

Cycle ownership is higher in Itwari telephone exchange than in Shankar Nagar square. 46% of households have at least one cycle in Shankar Nagar square, but in Itwari telephone exchange, households are having eight cycles per house (Fig. 15). In Itwari telephone exchange, people have less affordability to buy a motor vehicle, so they use bicycles for their daily works and to reach other AFS.

Problems faced by NMT users

In both the areas, NMT users are facing a lot of problems such as unsafe, high speed of motor vehicles, high traffic, negative attitude toward NMT, encroachments, improper design of footpath, lack of amenities, bicycle theft, poor signage, insufficient road crossings and segregation, narrow footpath, and lack of NMT lane which force them to reduce the use of NMT use (Fig. 16).

Narrow footpaths, lack of amenities, and encroachment are major problems faced by pedestrians at both the areas, whereas for cyclists, lack of NMV lane, encroachment on footpaths, lack of amenities, insufficient segregation, and bicycle theft are the significant problems present at both the areas. Lack of shade-giving trees and negative attitude toward the cyclists is the other vital issues in Shankar Nagar faced by NMT users. Improper design of footpath is the other major issue faced by pedestrians in telephone exchange (Fig. 17).

Satisfaction Level

According to satisfaction levels at Shankar Nagar (Fig. 18), majority of the households feel dissatisfied on safety for NMT as well as its infrastructure in the area, whereas in telephone exchange (Fig. 19), majority of people are extremely dissatisfied for the safety of cycling and infrastructure for them in their area. Also, they are disappointed for pedestrian's safety and infrastructure provided in their area.

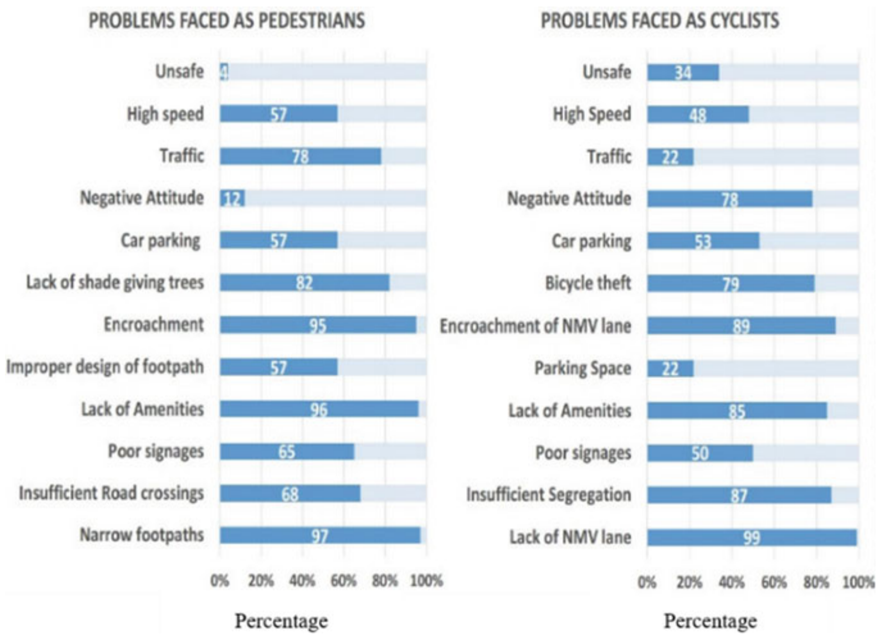


Fig. 16 Problems faced by pedestrians and cyclists at Shankar Nagar

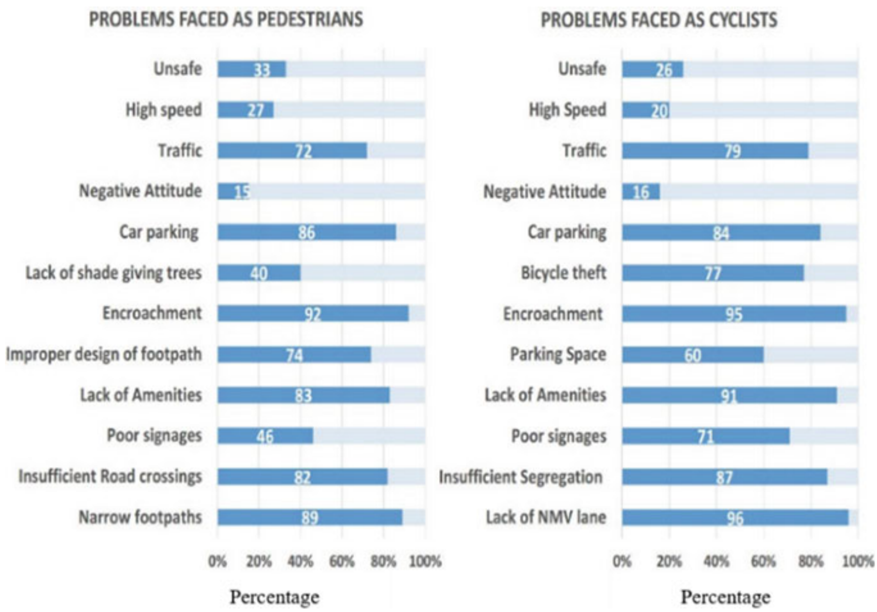


Fig. 17 Problems faced by pedestrians and cyclists at telephone exchange

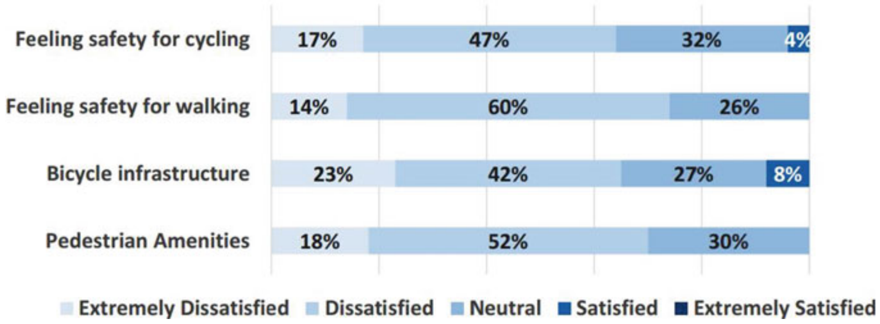


Fig. 18 Satisfaction level at Shankar Nagar

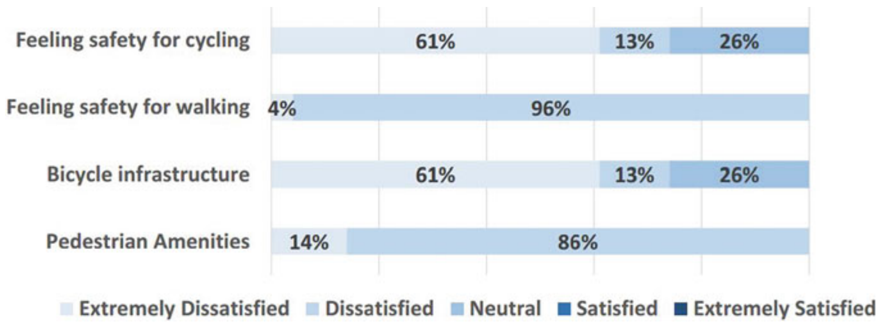


Fig. 19 Satisfaction level at telephone exchange

Willingness to use NMT

Households in Itwari telephone exchange are more willing to use NMT than in Shankar Nagar as inferred in Fig. 20. But households in both the areas demand proper infrastructure, better safety, and convenience for them to use NMT.

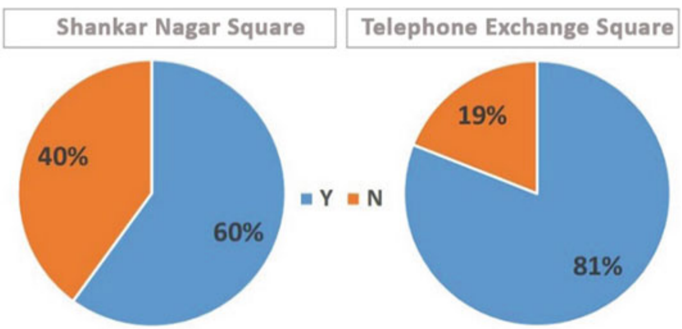


Fig. 20 Willingness to use NMT

5 Discussion and Conclusion

The macro-level assessment is assessed for the range of (0-2400 m) in the entire city. The near distance assessment reveals that 57.5% of AFS are available in the 64% of built area and can be accessed within 400 m distance from PT. But this does not correspond with the real scenario while considering the network distance from PT to reach the AFS. Nowadays, usage of network tools in ArcGIS is more common as retrieving the data from OSM has become easier. This encourages studies to compare the network distance and near distance [23]. In macro-level assessment, the findings show that percentage of AFS reaching through the network distance from PT within 0–800 m range is always half when measured by network as compared to near. If most of the AFS are not provided within 400 m network distance from PT stops, then it causes individuals to use motorized vehicles instead of walking or cycling [37]. Statistical studies performed show that almost 90% of AFS can be accessed from public transit within a range of 1200 m.

In micro-assessment, it is observed that builtup density, residential density, commercial density, and road density are 40% higher in Itwari telephone exchange than Shankar Nagar [38]. These higher densities show that there are more people within this area using AFS has sufficient road availability. In this study, it is found that increased density and clustering of activities improve accessibility. Shorter travel distances can enhance walking and cycling. Network analysis shows that some of the AFS in both the areas are not within 800 m walkable distance and it will be beneficial if the network is improved in such areas. From the user survey, it is observed that walking and cycling are unsafe and inconvenient due to the lack of traffic segregation and incomplete street section. Therefore, connectivity between vehicular roads and pedestrian pathways should be enhanced to allow more safe and convenient direct travel between locations. It should include distinct shortcuts for NMT wherever it is applicable. In the user study, it is observed that Itwari telephone exchange has more cycle ownership as compared to Shankar Nagar. Yet, motor vehicles are used in both areas for certain journeys that could otherwise be covered by NMT. The lack of infrastructure and planning is the major issues in discouraging individuals from using NMT. Although densities and mixed use are higher, still accessibility is discouraged if there are no adequate network and infrastructure.

Measuring NMT-based accessibility of the AFS from public transit using both near and network distance measures is a new method introduced in this paper for Indian cities. The previous accessibility measures are only concentrated to walk scores which use proximity distance that is not accurate in the real world. So, comparing Euclidean distance with actual network will be more beneficial. Measuring near and network distance on both macro-level and micro-level give better understanding of accessibility performance of city. The difference in these methods will not only help in identifying barriers for accessibility, but also help in recognizing shortest routes in the network. So, it improves infrastructure facilities for NMT to reach AFS as well as connecting public transit in a better way. Distance measurement in studies is aimed at investigating the accessibility of only specific type of facility often used in different developed countries [23, 39]. But in this study, we adapted different AFS for measuring distance to PT in Nagpur city, India which will improve in understanding and implementing policies. Application of both macro- and micro-assessment method in urban areas is a viable option to draw comprehensive conclusion on city's overall performance in accessing AFS. This study uses Google Map API where this kind of data can support urban local bodies of developing nations to gather information for planning future urban activities and services at city level.

References

1. Clifton KJ, Livi Smith AD, Rodriguez D (2007) The development and testing of an audit for the pedestrian environment. Landsc Urban Plan 80(1–2):95–110
2. United Nations Human Settlements Programme (2013) Planning and design for sustainable urban mobility
3. Murthy K (2011) Urban policies and the right to the city in India: rights, responsibilities and citizenship. In: Urban policies right to city India, pp 129–139
4. Ieda H (2010) Sustainable urban transport in an Asian context. Library for sustainable urban regeneration series, vol 9, p 86. Springer, Berlin
5. Wagner A (2018) Public transport integration and transit alliances, Module 3f. GIZ, Sustainable Urban Transport Project SUTP, 2018. Retrieved from https://www.sutp.org/files/contents/documents/News/2018_March/3f_Transit_Alliances_SUTP_TUMI_GIZ_final.pdf
6. Verma A (2018) Sustainable transport measures for liveable Indian cities. In: Indo-Norway project CLIMATRANS, Indian Institute of Sciences, Bangalore
7. Beltr G (2013) Promoting inclusive urban development in indian cities urban planning and land management for promoting inclusive cities. Giovana Beltrão Canada, Asian Development Bank
8. Agarwal OP, Rathi S, Kalra K, Gupta M, Pal S, Lakshmi A, Bhattacharya S, Mishra A (2014) Review of urban transport in India. Centre for Study of Science Technology & Policy, Institute of Urban Transport, India
9. Singal B (2014) National urban transport policy. Institute of Urban Transport (India) (IUT), Reterieved from www.iutindia.org
10. Leather J, Fabian H, Gota S, Mejia A (2011) Walkability and pedestrian facilities in Asian Cities State and issues. Asian Dev Bank Sustain Dev Work Pap Ser 17:69
11. Litman T (2015) Evaluating accessibility for transportation planning measuring people's ability to reach desired goods and activities, Jan 2015, p 57
12. Hull A, Karou S (2012) Spatial network analysis of public transport accessibility (SNAPTA). In: Silva C, Bertolini L (eds) Accessibility instruments for planning practice. COST Office, pp 183–188
13. Papa E, te Brömmelstroet M, Silva C, Hull A (2016) Accessibility instruments for planning practice: a review of European experiences. J Transp Land Use 9(3):57–75
14. Manoj M, Verma A (2016) Effect of built environment measures on trip distance and mode choice decision of non-workers from a city of a developing country, India. Transp Res Part D Transp Environ 46:351–364
15. Papa E, Silva C, Te Brömmelstroet M, Hull A (2015) Accessibility instruments for planning practice: a review of European experiences. J Transp Land Use 3:1–20
16. Tiwari G, Jain D (2012) Accessibility and safety indicators for all road users: case study Delhi BRT. J Transp Geogr 22:90–91
17. IIHS (2015) Challenges and recommendations: IIHS RF Paper on Urban Transport, p 42
18. Scheurer J, Curtis C (2007) Accessibility measures: overview and practical applications. Impacts Transit Led Dev New Rail Corridor 4:1–53

19. Saghapour T, Moridpour S, Thompson RG (2016) Public transport accessibility in metropolitan areas: a new approach incorporating population density. J Transp Geogr 54:273–285
20. Apparicio P, Abdelmajid M, Riva M, Shearmur R (2008) Comparing alternative approaches to measuring the geographical accessibility of urban health services: distance types and aggregation-error issues. Int J Health Geogr 7
21. Pirie GH (1979) Measuring accessibility: a review and proposal. Environ Plan A 11(3):299–312
22. Verma A, Mehvish VH (2015) Urban transport policies in india in context to climate change: an international perspective, pp 1–20
23. Azzopardi J (2018) Effect of distance measures and feature representations on distance-based accessibility measures. Lund University
24. MoUD (2015) Urban and regional development plans formulation and implementation (URDPFI) guidelines. Minist Urban Dev I:447
25. Zuidgeest M et al (2013) GIS for multi-modal accessibility to jobs for the urban poor in Ahmedabad. Thirteen WCTR 1–20
26. De Sousa Vale D (2008) Sustainable urban form, accessibility and travel: the relationship between polycentric urban development and commuting in Lisbon
27. Suber BA (2017) Public transportation accessibility impacts of the Tucson Modern Streetcar, May 2017
28. Perra VM, Sdoukopoulos A, Pitsiava-Latinopoulou M (2017) Evaluation of sustainable urban mobility in the city of Thessaloniki. Transp Res Procedia 24(2016):329–336
29. Agampatian R (2014) Using GIS to measure walkability: a case study in New York city, April 2014
30. Jafari F, Ghorbani R (2013) Urban density and social sustainable development on neighborhoods case study: Tabriz, Iran, vol 3, no. 8, pp 457–467
31. Wilbur Smith Associates (2008) Study on traffic and transportation policies and strategies in urban areas in India
32. Urban Mass Transit company Limited (2018) Comprehensive mobility plan, Nagpur. Retrieved from http://www.metrorailnagpur.com
33. Gutman J, Tomer A (2016) Developing a common narrative on urban accessibility: an urban planning perspective. In: Brookings Moving to Access, pp 1–43
34. Apparicio P, Séguin A-M, Naud D (2008) The quality of the urban environment around public housing buildings in Montréal: an objective approach based on GIS and multivariate statistical analysis. Soc Indic Res 86(3):355–380
35. Higgs G, Langford M, Norman P (2015) Accessibility to sport facilities in Wales: a GIS-based analysis of socio-economic variations in provision. Geoforum 62:105–120
36. Cervero R (1988) Land-use mixing and suburban mobility. Transp Q 42(3):22
37. La Rosa D (2014) Accessibility to greenspaces: GIS based indicators for sustainable planning in a dense urban context. Ecol Indic 42:122–134
38. Ministry of Housing and Urban Affairs (2001) National Mission on Sustainable Habitat
39. Higgs G, Jones S, Langford M, Heley J (2018) Assessing the impacts of changing public service provision on geographical accessibility: an examination of public library provision in Pembrokeshire, South Wales. Environ Plan C Polit Sp 36(3):548–568

Study of Driver's Behavior for Lateral Moving Vehicles

T. Dilipan, R. Das Vivek, and K. Parthan

1 Introduction

India possesses the world's second-largest road network accommodating about 1% of the world's vehicular population. Despite having this vast road infrastructure, the safety aspect incommodes the road users as adhered to the fact we contribute about 20% of the world fatalities. On investigating the causes for the accidents, it was explored that apart from over-speed, DUI (driving under influence) and inadequate infrastructural amenities play a significant rate of accidents occurred due to lane indiscipline. As per the report on road accidents in India 2016, published by Transport Research wing under Ministry of Road Transport and Highways, Government of India, in the year 2016 over 12,257 of 1,50,785 road accident deaths happen due to overtaking and when changing lanes. This number is greater than the combined road accidents deaths due to speed breakers and potholes which are accounted at 5720 fatalities. Lane ensures that there are enough gaps maintained between vehicles, a major uplifting problem with Indian traffic is vehicles are driven recklessly close to each other leaving little to no margin of error. Apart from safety the gap maintained between the vehicles also renders the corridor for emergency vehicles (ambulances) to drive through. In case of an emergency, vehicle has to move past all the occurring traffic; all motorist have to do is move over to the side to make the required room. To curb this issue, the government and other authorities have made lane indiscipline as a punishable offensive act as per Indian penal code under section 66 r/w 192 of motor vehicle act; even after enforcing it, if we look for a most uncontemplated traffic safety rule in India, it would be lane indiscipline. The lack of lane discipline

T. Dilipan (✉) · R. Das Vivek
Department of CTM, DSCE, Bangalore, Karnataka, India

K. Parthan
Department of Civil Engineering, BMSCE, Bangalore, Karnataka, India
e-mail: parthank.civ@bmsce.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_17

causes traffic to interact haphazardly with the encountering adjacent vehicles. If we scrutinize our existing traffic pattern, the two major factors identified influencing the lane indiscipline are traffic composition and driver's behavior. Our traffic is composed of a heterogeneous class of vehicles. There are very little amendments that can be made to alter our traffic composition, and on the other hand, the driver's behavior can be improvised on getting a thorough insight into their behavioral attributes. In the previous researches across the world, many eminent studies were made in concern with the study of vehicle's lateral movement behavior with respect to driver behavior. Now this study concentrates on the driver's behavior observing their lateral deviation, lateral acceleration and the minimum gap maintained while negotiation between different classes of vehicles.

2 Literature Review

Gunay found that there are many factors contributing to the weak lane discipline such as driving attitudes, poor road surface, poorly maintained lane markings and non-existence of studs on lane lines which leads to loss of safety, difficult traffic management, inapplicability of conventional lane-based models. Gunay developed a car-following model with particular reference to weak discipline of lane-based driving which was based on the discomfort caused by lateral friction between vehicles. Choudhury et al. [1] formulated a lane changing model enabling the drivers to jointly consider mandatory and discretionary considerations. Parameters of the model are estimated using detailed vehicle trajectory data. Santel [2] provided a scientific base for the derivation of reference values for the range of lateral movements in road traffic, and this is regarded as the amplitude of a vehicle trajectory on a straight track section. Gunay and Erdemir [3] found that the vehicles will prefer to lag behind or lead ahead of the neighboring vehicle until the flow is high. When the headway with respect to the front vehicle is less, it cannot accelerate and also in a condition of higher density vehicles are forced to move side by side. An attempt to mimic multilane traffic flow for better microscopic modeling was done by the authors. Jin et al. [4] modified time of collision equation with visual angle information and was introduced to the general motors model. Visual angle information takes into account the lateral separation characteristics between the follower and leader. The suggested model was checked with simulations done under different driving scenarios. The model could describe local and asymptotic stabilities, lateral movement, the effect of neighboring vehicles and complex behavior like partial lane changing. Maurya [5] developed comprehensive microscopic simulation model for uninterrupted unidirectional traffic stream which was able to determine both steering control as well as speed control actions of driver. Lateral control was described by goodness of path and longitudinal control by the acceleration of following vehicle. The model was evaluated using microscopic and macroscopic features such as acceleration noise, speed distribution, flow density and effect of road blockage on capacity. The model was validated with field data, and it was able to simulate realistically the behavior of traffic streams on wide roads without lane discipline. The input to predict following behavior was safe distance headway, relative velocity, relative position and relative acceleration of lead and following vehicles. Sreekumar and Maurya [6] identified the need for incorporating the no lane discipline and the heterogeneity into the model and developed *CUTSiM* (Comprehensive Unidirectional Traffic Simulation Model). *CUTSiM* is used to simulate single lane traffic streams as well as traffic streams on wide roads with and without lane discipline. Mahapatra and Maurya investigated the relationship between the vehicle longitude speeds with the lateral characteristics in the Indian traffic condition. The main purpose of their study was to observe the lateral acceleration, speed values in the moderate traffic conditions. There exists an inverse relation between longitudinal speed and lateral acceleration for all the three types of vehicle considered except at lower speeds. The vehicles rate of change of heading angle also reduces at higher speed. Metkari et al. [7] clubbed the two models inspired by the Gipp's model, one covering the aspect of weak lane discipline and another covering the vehicle type-dependent behavior. The study was conducted for uninterrupted unidirectional traffic on straight and level midblock section in Delhi. Mathew et al. [8] studied the weak lane discipline by dividing the lanes into small strips or bands and tracking the vehicle movement along them. Strip size is fixed according to the smallest vehicle being simulated. The lateral movement model calculates the benefit in making the lane change based on which whether decision for lane change is taken. If there is no change in lane, then it will have a normal following behavior. Geetimukta et al. studied the lateral characteristics of the vehicles with respect to the vehicle's speed is collected for five different types of vehicles. The effect of the vehicle's speed as wells as the vehicle type on the lateral behavior is studied for three metropolitan cities of India, and they observed that there exist an inverse relationship between the speed and the lateral characteristics.

Reviewing the literature helps us to acknowledge that the researches conducted to study the lane changing and other lateral movement were emphasized on the vehicular characteristics. In this study, efforts are made to study the driver's behavior of the lateral interacting vehicles. Thereby, a thorough understanding of the behavior and characteristics of the drivers of the various types of lateral moving vehicles in the Bengaluru's mixed traffic condition can be achieved.

3 Methodology

This project is aimed at studying the behavior of the laterally moving vehicles by observing certain parameters like their lane changing attitude, their lateral and longitudinal speed, acceleration and lateral gap considered between the vehicles while overtaking, all the mentioned attributes are reckoned by determining the path and positions of the vehicles along the path in the study area. The key steps involved in this study are: (1) selection of suitable site for field surveying (2) collection of essential traffic data by surveying (3) extraction of data by processing the video collected (Fig. 1).

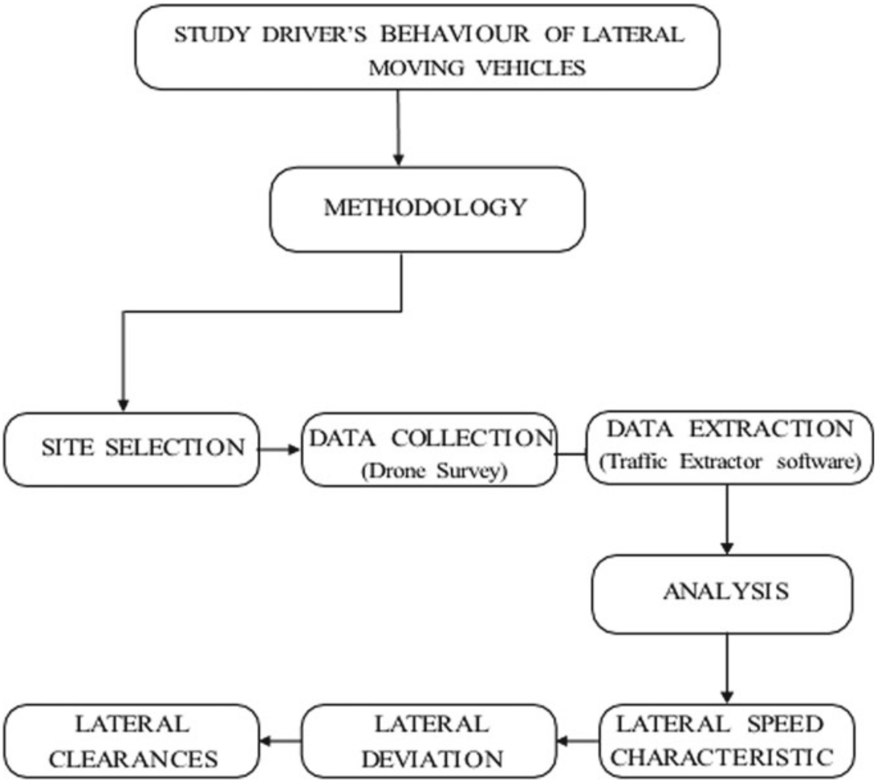


Fig. 1 Flow chart of methodology for the study

3.1 Site Selection

Two midblocks have been selected to collect the required data for this study in Bangalore suburban, (Location 1) K.R. Puram to Tin factory and Tin factory to Ramamurthy Nagar ring road (Location 2). These midblocks were selected in a criterion that they had a moderate traffic flow, thereby the movement of the vehicles is not restricted due to the traffic complexity. The two locations selected are shown in Fig. 2; they were 12-m wide 3-lane flexible pavement, connecting the major arterial road networks of the Bangalore city.

3.2 Data Collection

In this study video graphic, surveying using drone was adopted, to capture the video of the moving traffic at resolution of 720p (30 fps) in. wp4 format. The drone was

Fig. 2 Image of the study location K.R. Puram–Tin factory and Tin factory–Ramamurthy Nagar ring road

Fig. 3 Image of Drone view of site location at K.R. Puram–Tin factory

flown at an altitude of 170-m above the ground level such that it covers 120-m of the study stretch without interrupting the drivers. The swinging angle of the camera was maintained less than 0 degree to avoid distortion of the video. The movement of the traffic was video captured for the duration of 1 h from 4:00 pm to 5:30 pm. The drone view of the two surveying locations are shown in Fig. 3

3.3 Data Extraction

The foremost data required to be extracted from the video was the trajectories (i.e., positions that the vehicles occupy in the study region during the study period). This was achieved by feeding the captured video into the software TRAFFIC

Fig. 4 Image of *traffic extractor* trajectory analysis

EXTRACTOR developed by IIT Mumbai as shown in Fig. 4. This software transforms the image co-ordination based on vanishing point camera calibration technique. The expression for real-world co-ordination transformation are mentioned at Munigety et al. [8]. The respective positions of the different class of vehicles in the video are yielded in terms of coordinates (x, y, t) with x being the longitudinal points along the road, y being the lateral points across the road and t being the time in seconds. These coordinates were then plotted for time versus longitudinal movement to study the longitudinal interaction of the vehicle with other adjacent vehicles with respect to time and time versus lateral movement to study the lateral interaction of the vehicles with other neighboring vehicles with respect to time. To get more detailed insight, the lateral movements of the individual class of vehicles were plotted with respect to time. As this study is focused on the lateral behavior of the vehicles, the lateral trajectories of the vehicles across the width of the road are plotted with respect to time; this distinct lateral movement of the vehicles helps us to further understand its driver's lateral interacting behavior.

4 Data Analysis

The driver's behavior is analyzed by considering the vehicle driven pattern and examining its parameter. The essential parameters reckoned in this project include vehicles lateral acceleration with respect to its longitudinal acceleration, vehicular lateral clearances at the time of its interaction with the adjacent vehicles and the additional distance travelled by the vehicles due to the lateral movement.

Lateral Clearances. It is the minimum gap maintained between the vehicles at the time of negotiation. This is an important aspect to be considered in terms of safety of the drivers as this indicate the driver's perception of safe interaction with the neighboring vehicle at that instance of contact. The minimum clearance maintained by auto was with two-wheeler that is 0.1-m gap, two-wheeler exhibiting a minimum clearance of 0.07-m with the neighboring two-wheeler, bus show a least interaction of 0.1-m with car, minimum space between the cars are 0.09-m with two-wheeler, LCV with the least clearance of 0.1-m on interacting with car. HCV having 0.1-m gap with two-wheeler was observed. The observations of lateral clearances of vehicles with other classes of vehicles are tabulated in Table 1. The minimum value varies from 0.07 to 0.18-m, whereas the average value varies from 0.23 (bus and two-wheeler) to 0.66-m (auto and car). This clearance was observed between auto and car. The lower values indicate the absence of lane discipline and risky driving behavior.

Lateral Deviation. It is lane indiscipline which refers to the additional distances travelled by the vehicles from the actual distance across the path due to lateral movements of vehicles during the time of travel. In this study, the study zone at the selected two locations were limited to the length of 120-m and 12-m wide. The extra distance travelled by the vehicles within the study zone are computed and extrapolated to 1-km distance and the results are in Table 2. Accordingly, the auto and two-wheeler deviate to an average distance of 5.02-m and 6.54-m per 120-m, respectively, car differ from actual path by 8.03-m per 120-m, LCV and HCV on average diverge by 3.72-m and 5.07-m per 120-m, respectively. Percentage deviation is observed high for car about 6.69% followed by auto 5.45%

Lateral Acceleration. The lateral acceleration of the vehicles refers to the rate of change of velocity of the vehicle while they laterally interact with their adjacent vehicles either within the same lane or in the adjacent lane while changing the lane at the time of movement. The vehicles' longitudinal accelerations are also computed; the determination of this lateral and longitudinal acceleration of the vehicles is performed by computing the change in lateral and longitudinal velocity of the vehicles which is achieved by using the vehicular (x, y) trajectories data extracted with respect to time (t). In location 1, it was observed that buses exhibit least speed characteristic

Table 1 Analysis of *lateral clearances* between the vehicles

	Auto		2-wheeler		Car		Bus		Lcv		Hcv	
	Min	Avg	Min	Avg	Min	Avg	Min	Avg	Min	Avg	Min	Avg
Auto	0.17	0.43	0.10	0.41	0.14	0.66	0.14	0.50	0.17	0.37	0.14	0.47
2 wheeler	0.10	0.41	0.07	0.42	0.09	0.33	0.10	0.23	0.12	0.35	0.10	0.35
Bus	0.15	0.47	0.10	0.23	0.10	0.24	0.12	0.31	0.12	0.31	0.12	0.36
Car	0.14	0.66	0.09	0.33	0.18	0.39	0.13	0.40	0.10	0.24	0.18	0.20
LCV	0.17	0.37	0.12	0.35	0.10	0.24	0.12	0.36	0.13	0.32	0.14	0.35
HCV	0.15	0.47	0.10	0.35	0.18	0.20	0.12	0.36	0.14	0.35	0.14	0.35

Table 2 *Lateral deviation* of the vehicles due to lane indiscipline lateral movement

Vehicle type	Parameter	Distance in m/120 m	Distance in m/1 km	Percentage
AUTO	Min	0.69	5.74	0.57
	Max	9.35	77.96	7.80
	Avg	5.02	41.85	4.18
Two-wheeler	Min	0.31	2.61	0.26
	Max	12.77	106.39	10.64
2 Wheeler	Avg	6.54	54.50	5.45
	Min	2.91	24.25	2.43
Bus	max	4.08	34.01	3.40
	Avg	3.49	29.12	2.91
	Min	0.08	0.66	0.07
CAR	max	15.98	133.20	13.32
	Avg	8.03	66.93	6.69
	Min	0.45	3.76	0.38
LCV	max	7.00	58.37	5.84
	Avg	3.72	31.06	3.10
	Min	1.83	15.25	1.52
HCV	max	8.31	69.28	6.93
	Avg	5.07	42.26	4.22

values of 0.45 m/s lateral velocity and 0.69 m/s^2 lateral acceleration. In location 2, bus showed the least speed characteristics having 0.39 m/s velocity and 0.58 m/s^2 acceleration. The longitudinal and lateral accelerations of vehicles at location 1 are tabulated in Tables 3 and 4, and for location 2, accelerations are tabulated in Tables 5 and 6, respectively.

Table 3 *Lateral velocity and acceleration* of the vehicles in *K.R. Puram–Tin factory* location

Vehicles	Lateral velocity (m/s)	Lateral acceleration (m/s^2)
Auto	0.48	0.63
2-Wheeler	0.47	0.72
Bus	0.45	0.69
Car	0.50	0.72
LCV	0.47	0.72
HCV	0.50	0.80

Table 4 *Longitudinal velocity and acceleration* of the vehicles in *K.R. Puram–Tin factory* location

Vehicles	Longitudinal velocity (m/s)	Longitudinal acceleration (m/s^2)
Auto	19.40	2.99
2-Wheeler	22.08	3.25
Bus	21.52	2.75
Car	22.85	2.79
LCV	20.61	2.87
HCV	18.63	3.21

Table 5 *Lateral velocity and acceleration* of the vehicles in *Tin factory–Ramamurthy Nagar* location

Vehicles	Lateral velocity (m/s)	Lateral acceleration (m/s^2)
Auto	0.43	0.69
2-Wheeler	0.47	0.72
Bus	0.39	0.58
Car	0.45	0.76
LCV	0.38	0.66
HCV	0.42	0.68

Table 6 *Longitudinal velocity and acceleration* of the vehicles in *Tin factory–Ramamurthy Nagar* location

Vehicles	Longitudinal velocity (m/s)	Longitudinal acceleration (m/s^2)
Auto	22.30	2.61
2-Wheeler	23.68	2.73
Bus	20.39	2.51
Car	25.72	2.74
LCV	22.74	2.55
HCV	22.03	2.56

5 Conclusion

In this study, the lateral characteristics, i.e., lateral clearance, lateral deviations, lateral speed and lateral acceleration are computed from the data collected in 2 locations in Bangalore suburban. The main findings of this study are summarized in regards with the lateral clearances; the least gap maintained was found to be 0.07-m between the two-wheelers. This finding of two-wheeler pair maintaining the least lateral clearances corresponds with the finding by Mathew et al. [8] though the clearance he determined was higher (i.e., 0.2-m) and a lateral clearance of 0.09-m was found between the two-wheeler and car. The least clearances maintained by the two-wheels are correlated to size of the vehicles, as the size of the two-wheelers are apparently smaller than the other category of vehicles, thereby this gives the drivers a mindset to maneuver the vehicles to even smaller gaps available, thus leading to the lesser lateral clearance maintenance. The lateral deviation was observed more in cars showing a variation of 15.69-m per 120-m, which is computed to be 13.32% of lateral deviation, and the least variation was also observed in cars having a value of 0.08-m per 120-m that is 0.07% of variation from the actual path. In location 1, the lateral speed and acceleration was observed high in HCV having 0.505 m/s and 0.802 m/s^2, respectively. In location 2, cars exhibit highest lateral speed characteristics—0.452 m/s velocity and 0.761 m/s^2 acceleration. The lateral acceleration of the car in this study is higher than the acceleration of the cars (i.e., up to 0.4 m/s^2) determined in research paper by Geetimukta et al. (2015). This is associated to the vehicular characteristics that cars can attain higher lateral acceleration even at lower speed. The average lateral acceleration of auto is 0.66 m/s^2, two-wheeler is 0.72 m/s^2, bus is 0.63 m/s^2 and LCV is 0.69 m/s^2. This study of driver's behavior acknowledges the fact that the two-wheeler users exhibit a risky lateral interaction. The least gap maintained by the two-wheeler users leaves them in tight spot with the raising lateral acceleration, and the chances of collision is higher. The lower values indicate the absence of lane discipline and risky driving behavior. Thereby, it is significant for every vehicular driver's to be aware and conscious about the lane discipline while driving the vehicles.

References

1. Choudhury C, Ramanujam V, Ben-Akiva M (2008) A lane changing model for urban arterials. In: Proceedings of the 3rd international symposium of transport simulation, Gold Coast, Australia
2. Santel G (2010) Lateral driving behaviour. In: Proceedings of 10th Swiss transport research conference, ETH Zurich, Monte Verita, pp 1–26
3. Gunay B, Erdemir G (2011) Lateral analysis of longitudinal headways in traffic flow. Int J Eng Appl Sci 3(2):90–100
4. Jin S, Wang D, Yang X (2011) Non-lane-based car-following model with visual angle information. Transp Res Record J Transp Res Board 2249:7–14
5. Maurya AK (2011) Comprehensive approach for modelling of traffic streams with no lane discipline. In: Proceedings of conference on models and technologies for intelligent transportation systems, Leuven, Belgium, pp 36–40
6. Sreekumar M, Maurya AK (2012) Need for a comprehensive traffic simulation model in Indian context. In: Proceedings of international conference on emerging frontiers in technology for rural area, pp 13–18
7. Metkari M, Budhkar A, Maurya AK (2013) Development of simulation model for heterogeneous traffic with no lane discipline. Procedia-Soc Behav Sci 104:360–369
8. Mathew TV, Munigety CR, Cherian A, Ostawal A (2014) A space discretization based simulation approach for non-lane based traffic conditions. In: Proceedings of Transportation Research Board 93rd Annual meeting, No. 14–3854

Evaluation of Queue Discharge and Lane Occupancy Due to Seepage Behaviour of Small Sized Vehicles at Intersection

S. R. Kishore, R. Das Vivek, and K. Parthan

1 Introduction

The location of the intersection is Hope farm junction, near Whitefield area, Bangalore, Karnataka. The study location is the four-arm signalized intersection. The traffic regime observed at the node is the heterogeneous traffic condition where different class of vehicles makes use of same right of way. This leads to complex traffic condition with increased vehicular interaction. Additionally, the traffic condition at the node, the lane discipline and first-in-first-out rule were found to be scarce. The major contributors to this complex activity were found to be the small sized vehicles (bikes, autos, etc.). These small sized vehicles, instead of following the queue, which were found to seep through the voids between the stationary or almost stationary large size vehicles whilst advancing to the intersection stop line. The seepage phenomena in the study is described as per the journal ABMTRANS 2015, Salzufer 17-19, 10587 Berlin, Germany. This phenomenon might affect the discharge rate of the vehicles at the intersection considerably with increased queue length, thus causing the decrease in acceleration and increase in wait time of the large sized vehicles. In this study, a seepage zone is considered up to a distance of 30 m of the link from the intersection stop line where change in density of vehicles is observed due to the seepage behaviour of small sized vehicles. Further due to decrease in acceleration characteristics of large size vehicles, the speed of vehicles at the intersection is also affected, thus affecting the free flow of vehicles at intersection. The main objective of this study is to determine the percentage of lane discipline and other factors effecting the efficiency of the network.

S. R. Kishore (✉) · R. Das Vivek
Department of Civil Engineering, Ramaiah Institute of Technology, Bangalore, India, Karnataka

K. Parthan
Department of Civil Engineering, BMSCE, Bangalore, Karnataka, India
e-mail: parthank.civ@bmsce.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_18

2 Literature Review

Oketch [1] modelled the performance characteristics of heterogeneous traffic streams containing non-motorized vehicles; in this paper, author has specified characteristics and driver behaviour of non-motorized vehicle. Wang et al. [2] describe the effect of nonstandard vehicle's special behaviour on mixed traffic modelling, where the author has prescribed the simulation model on the behaviour of the traffic regime with queuing discipline and lane discipline. Eissfeldt et al. [3] describe the simulation of traffic flow with queues. Sperley and Pietz [4] describe the motorcycle lane sharing behaviour. Nair et al. describe a porous flow approach to modelling heterogeneous traffic in disordered systems. Asaithambi et al. [5] describe the evaluation of exclusive stopping space for motorcycles at signalized intersections under mixed traffic condition. Aupetit et al. [6] describe the naturalistic study of rider's behaviour in lane–splitting situations. Agarwal and Lammel [7] describe the seepage behaviour of small sized vehicles in a congested heterogeneous traffic regime and specify the merits and demerits caused due to seepage phenomenon. Fan and Work describe a cell transmission model for heterogeneous multiclass traffic flow with creeping. Agarwal and Lammel [8] further to the development of previous work, the author describes the seepage behaviour of small sized vehicles in mixed traffic regime based on agent-based simulation model. Caroline Sutandi and Dennis Dwika Siregar describe evaluation of exclusive stopping space (ESS) for motorcycle at signalized intersections in large cities in Indonesia; in this paper, the author has described the solution for growing motorcycle traffic, where the author has introduced ESS for motorbikes by dividing the lane width in two types square and p type. Qiao-Xu Qin and Yaun-Biao Zhang describe the effect of different lane occupancy on road traffic capacity, this paper demonstrates the change process of road capacity during the accidents, and the phenomena is described using fuzzy comprehensive evaluation method.

3 Methodology

3.1 Study Location and Data Collection

The inventory survey was conducted at the node at Hope farm junction, Whitefield, Bangalore. For each arm, the permanent objects present at the sidewalks like trees, electric poles, etc., were marked through the link up to certain distance where maximum queue length was observed and the distances between the objects were recorded in order to measure the queue length formed by the vehicles. The geometric details of the intersection were also measured. The videography survey was conducted by means of aerial drone from 8.30 am to 10.00 am on a week day, and the video recording of one hour was collected. The drone was flown up to a height of 400 m above the ground level such that the section length captured up to a

Fig. 1 Google satellite image of the study location (Google maps date 24-05-2019)

distance of 200 m radially covering all the phases of the intersection. An attempt was made to capture the top view of the study area and to achieve the swing angle close to zero which would support the usage of traffic data extractor software in achieving the vehicle trajectory information. The actual queue length formed by the vehicle was measured, and by considering the queue length and width of the carriage way, the queue area was measured. The area of each class of vehicles occupied in the queue was measured knowing the physical characteristics of each vehicular class. The actual queue area was then compared with the area of the vehicles occupied in the queue and effective occupancy of the vehicles in the queue was analyzed during the red time interval. The data such as traffic composition, the capacity of the intersection as per Indo-HCM, vehicle positioning and the varying in density of the vehicles in the seepage zone were measured (Figs. 1 and 2).

3.2 Data Extraction and Analysis

3.2.1 Traffic Composition and Signal Phase Information

The volume count of different class of vehicles was determined for the study area for each arm of signalized intersection against the signal cycle timings, and the cumulative volume count was determined. The signal phase information was collected, which was in practice during the survey duration. The signal timing design that is in practice during different time frame at the intersection was collected from traffic department and described in Table 1 (Figs. 3 and 4; Tables 2 and 3).

Fig. 2 Top view of the signalized intersection captured through drone

3.2.2 Capacity Analysis of the Intersection as per Indo-HCM

Based on the capacity analysis made as per Indo-HCM, it is seen that the volume to capacity ratio is above the stipulated limits of free flow of traffic. In each phase of the intersection, based on the analysis made, the LOS of service is found to be '*F*'. The analysis of LOS for each phase of intersection is mention in Table 4.

The delay caused by the vehicles at the intersection was analyzed as per Indo-HCM. The total delay occurred was found to be 30.557 s which are greater than the stipulated limits as per code. The LOS was found to be '*F*'.

Based on the above estimation, it is seen that the total delay experienced by the vehicles at the intersection is about 306 s which are signifying the most awful condition, where LOS as per Indo-HCM is found to be '*F*'.

3.2.3 Determination of Spot Speed of Vehicles Approaching Various Arms of Intersection

The speed of the vehicles dispersing and approaching to various arms of intersection during the signal green time was recorded, and the spot speed of the different class of vehicles utilizing the facility was determined. The existing speed of vehicles at the intersection was analyzed by means of cumulative frequency distribution diagram of spot speed, and 15th percentile and 85th percentile of speed were determined. The spot speed of the vehicles at intersection was analyzed for through, right, left movements approaching from each phase of intersection (Fig. 4).

Table 1 Signal timing design of the signalized intersection (resource traffic department Whitefield)

Hope Farm Junction, White Field Police Station—Fixed																
Timings data																
ITPL RD X Channasandra			Phase													Cycle time
Data	Road	From	1			2			3			4			5	
	A	ITPL	R	L	S								L		Exclusive pedestrian	
	B	Channasandra				R	L	S		L						
	C	Whitefield		L					R	L	S					
	D	Kadugodi					L					R	L	S		
Timings	07:00	08:00	40			30			40			40			10	160
	08:00	12:00	80			50			40			40			10	220
	12:00	16:00	50			40			40			40			08	178
	16:00	21:00	80			50			40			40			10	220
	21:00	22:00	40			30			40			40			07	157
Sunday	07:00	21:30	40			30			40			40			08	158

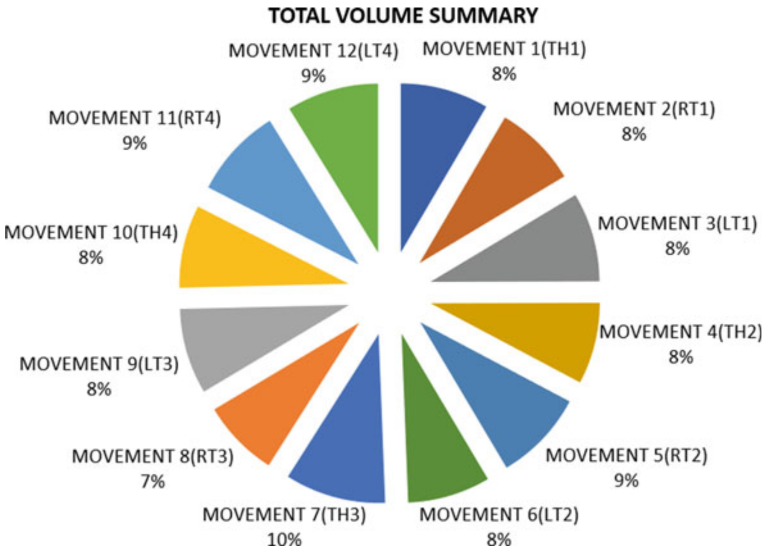


Fig. 3 Traffic volume composition

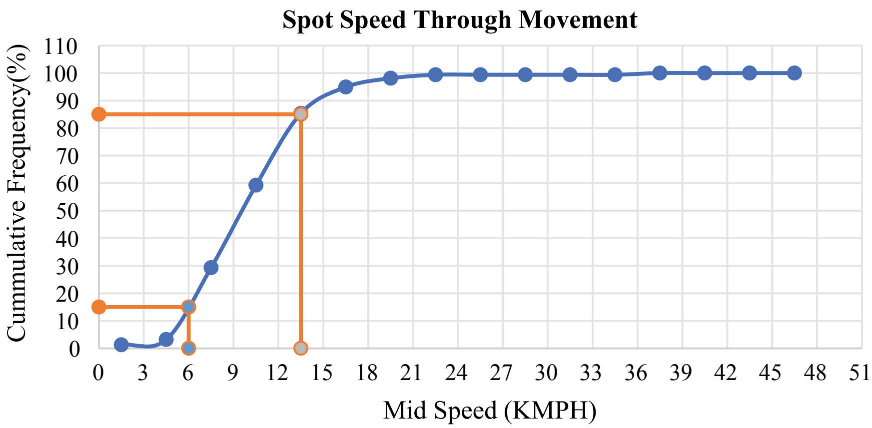


Fig. 4 Cumulative frequency graph of through movement at intersection

Based on the analysis, it is seen that the speed of the vehicles of through movement is 13 KMPH for 85th percentile and that of 15th percentile is found to be 6 KMPH, respectively (Figs. 5 and 6).

Based on the analysis, it is seen that the speed of the vehicles of right turning movement is 15 KMPH for 85th percentile and that of 15th percentile is found to be 6KMPH, respectively.

Table 2 Representation of volume count of each phase of intersection in terms of PCU

Movement													
Time (s)	1 (THI)	2 (RTI)	3 (LTI)	4 (TH2)	5 (RT2)	6 (LT2)	7 (TH3)	8 (RT3)	9 (LT3)	10 (TH4)	11 (RT4)	12 (RT4)	Total
9:00	122	214	169	184	208	237	192	203	180	231	159	220	2319
9:05	245	222	222	197	246	157	280	212	140	223	274	242	2660
9:10	268	225	182	167	259	226	237	196	249	219	206	228	2662
9:15	207	111	146	198	145	177	172	214	220	155	246	238	2229
9:20	159	240	208	221	214	240	246	234	200	190	262	209	2623
9:25	207	184	291	113	176	136	230	83	235	156	154	178	2143
9:30	185	204	214	164	185	258	226	167	146	138	180	202	2269
9:35	193	224	215	227	207	173	218	185	200	160	168	161	2331
9:40	201	198	118	159	129	127	251	116	259	192	203	210	2163
9:45	186	131	191	180	234	181	231	130	196	227	237	186	2310
9:50	214	129	205	263	227	180	197	201	173	217	194	230	2430
9:55	220	155	241	141	244	178	242	127	166	203	242	170	2329
10:00	204	223	242	197	223	230	255	223	172	140	149	247	2450
PCU/HR	**2611**	**2460**	**2644**	**2411**	**2697**	**2445**	**2977**	**2291**	**2536**	**2451**	**2674**	**2721**	**30,918**

Table 3 Representation of LOS of the intersection based on volume to capacity ratio

Phase	Phase 1	Phase 2	Phase 3	Phase 4
Details	**NB**	**EB**	**WB**	**SB**
Approach width, W (m)	9.5	7.62	9.30	9.36
Demand volume (PCU/H)	2572	2518	2602	2616
Peak hour factor (PHF)	0.972	0.933	0.87	0.96
Peak hour volume (PCU/h)	2647	2699	2991	2725
Number of Bus, (N_b) (buses/h)	57	37	69	96
Initial surge	**AB**	**AB**	**AB**	**AB**
Details	NB	EB	WB	SB
Approach width	9.5	7.62	9.30	9.36
Unit base saturation flow rate (PCU/h)	570	683	582	579
Bus blockage F_{bb}	0.91	0.927	0.888	0.846
Blockage caused by right turning vehicles (F_{br})	0.737	0.672	0.732	0.733
Initial surge (F_{is})	1	1	1	1
Effective green time (s)	80	50	40	40
Adjusted saturation flow rate (S_{fi}) PCU/h	3632	3243	3519	3361
Capacity (PCU/h)	1384	773	671	641
$\lambda = g/c$ ratio	0.381	0.239	0.19	0.19
$X = V/C$	1.91	3.49	4.45	4.25
LOS [V/C ratio]	**F**	**F**	**F**	**F**

Table 4 Estimation of LOS of intersection based on delay estimation

C-932 PCU/H	Cycle length-210 S	Min(X, 1) = (1.91, 1)	T-1 h
Delay 1 = 279 s	Total delay = 305.57 s		
Delay 2 = 26.557 s	LOS = F		
Delay 3 = 0 s			

Based on the analysis, it is seen that the speed of the vehicles of left turning movement is 11 KMPH for 85th percentile and that of 15th percentile is found to be 5 KMPH, respectively.

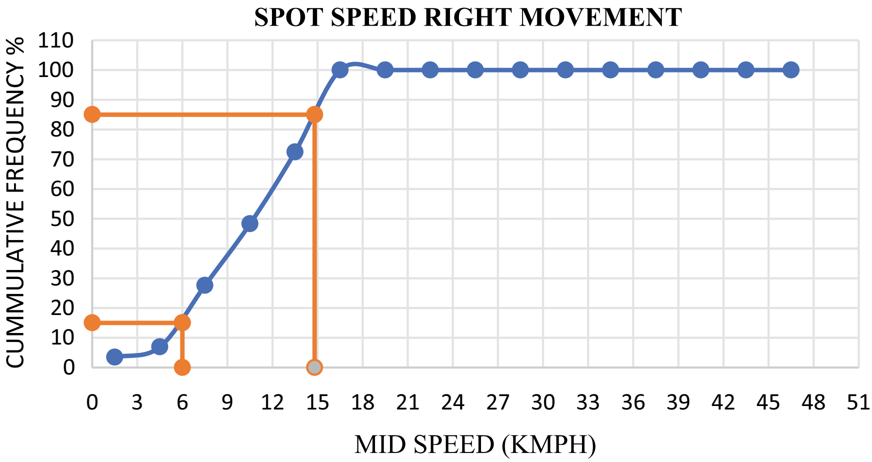


Fig. 5 Cumulative frequency graph of right movement at intersection

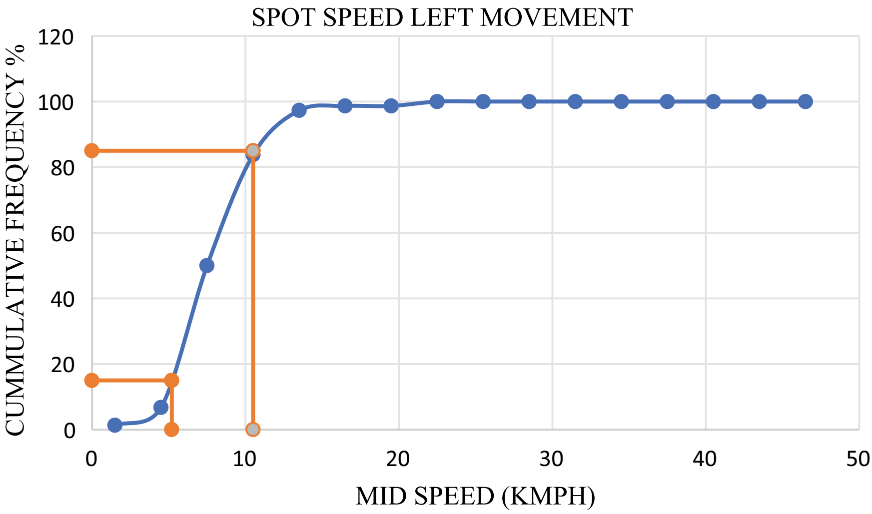


Fig. 6 Cumulative frequency graph of left movement at intersection

3.3 Representation of the Queue Area and Area Occupied by the Vehicles and Effective Utilization of the Queue Area by the Vehicles

The below bar graph represents the queuing data observed at the intersection in the 160 s of the red signal time in signal cycle. The queue length was measured, the length and road area up to which the vehicles are occupied were measured, and the area of the vehicles within the queue area was recorded for each 5 s interval. The variance in the queue area and the percentage area occupied by the vehicles was noted. The effective utilization and the area within the lane were analyzed (Fig. 7).

The actual queue length formed by the vehicle was measured, and by considering the length and width of the carriage way, the queue area was measured and was compared with actual area of the vehicles occupied in the queue and effective occupancy of the vehicles in the queue was analyzed during the red time interval. The above graphical representation is the queue area formed by the vehicles at the intersection. The queue area is determined, and the area of the vehicles occupying the queue was determined and both were compared. The effectiveness in the occupancy rate of the vehicles in the queue and effective utilization of the queue area was analyzed in red signal during each signal cycle. It was found that the area of the vehicle is minimum compared with overall queue area which is indicating the occupy of the vehicle in the queue area is not effective which is causing the formation of large queue length. By positioning the vehicles effectively within the queue, area will reduce the queue length.

3.4 Determination of Variance in Lateral Movement and Lane Changing Behaviour of Vehicles

The positioning of each vehicles approaching the node was determined by plotting the trajectory of vehicles obtained by means of traffic data extractor IIT Bombay. The movement of each vehicles approaching the node was analyzed. The typical movement of small sized vehicles seeping through the gaps of stationary or almost stationary vehicles and approaching the intersection stop line was determined. Two-wheelers completely represented in blue colour in the graph against other class of vehicles, so as to understand the positioning of small sized vehicle and its lateral movement in mixed traffic regime (Figs. 8 and 9).

The vehicle trajectory information obtained in Fig. 9 indicates the positioning of the different class of vehicles at the intersection. The *x*-axis in the graph represents the time duration in seconds during the vehicle movement within the defined grid, and *y*-axis represents the lateral deviation of the vehicles along the width of the road.

4 Observed Seepage Behaviour at Intersection in Each Cycle of Signal Timing

During the red truncation, initially after the red signal, the density of vehicles waiting at the node will be low up to certain period of red time. Since the vehicles waiting in the queue will be dissipated. Later during the red signal time, the small sized vehicle does not queue up instead they seep through stationary or almost stationary large

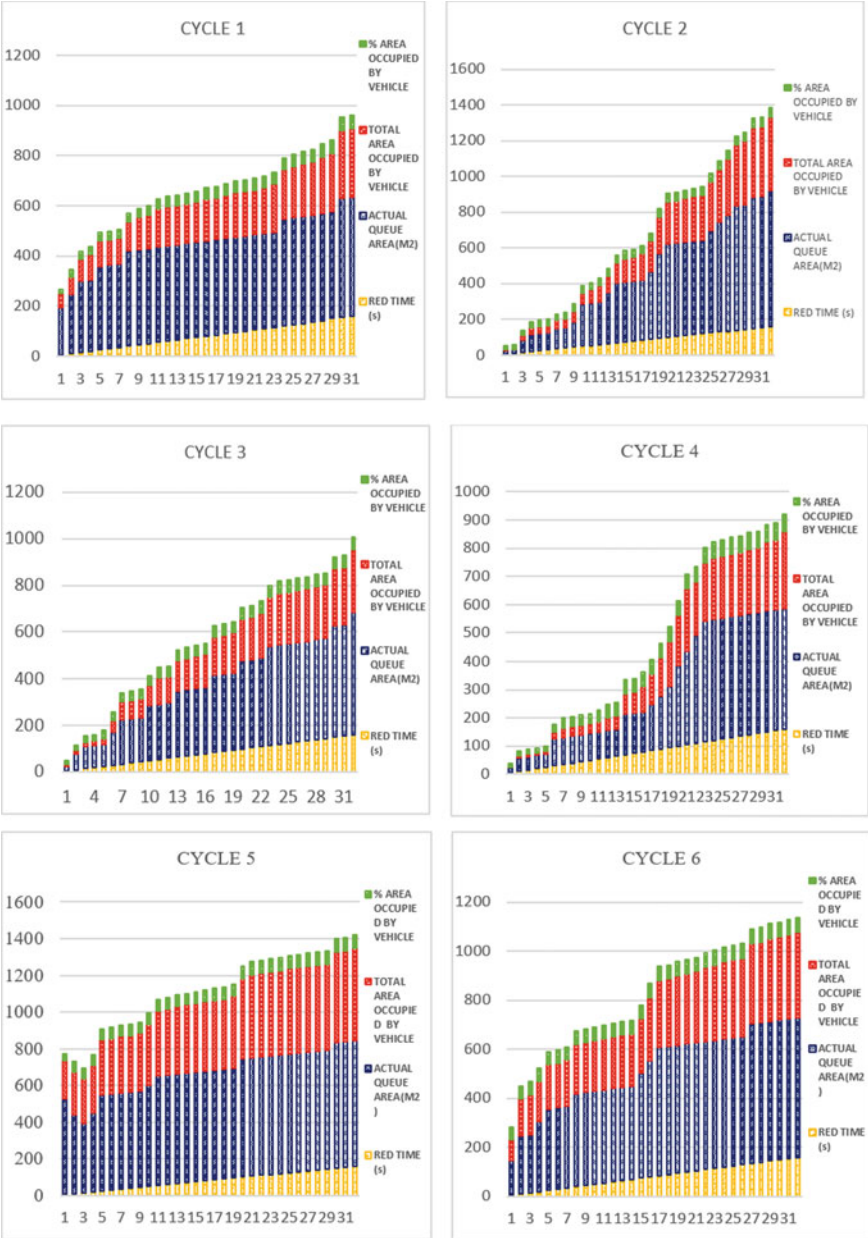


Fig. 7 Representation of queue area, area occupied by the vehicle within the queue and effective utilization of space in each signal cycle

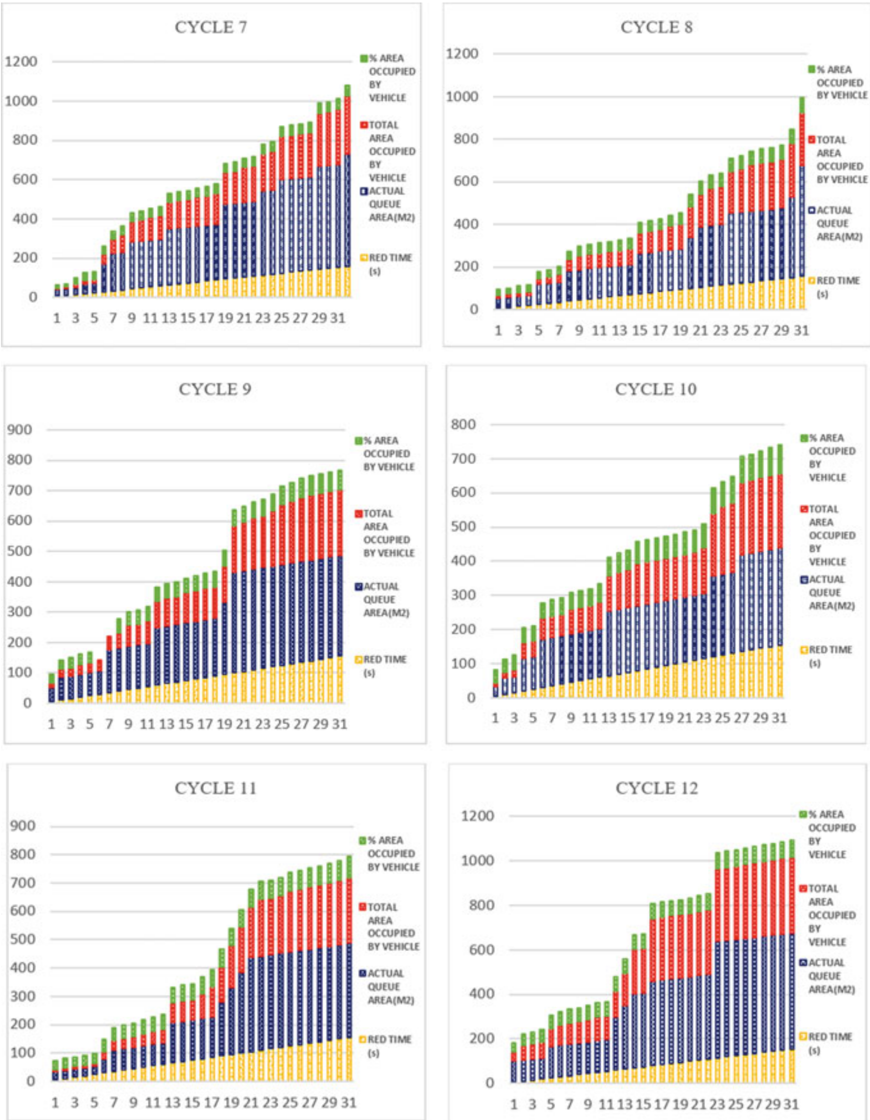


Fig. 7 (continued)

sized vehicles and approach to intersection stop line by violating the first-in-first-out rule or car following method. Here, the distance of 30 m is considered from the intersection stop line and is described as seepage zone. Initially, at the red signal time, the vehicles approach to the intersection stop line without any lane changing behaviour as the lane is free from any vehicles or obstruction, later once the vehicles are occupied in the seepage zone, the small sized vehicle does not follow the queue

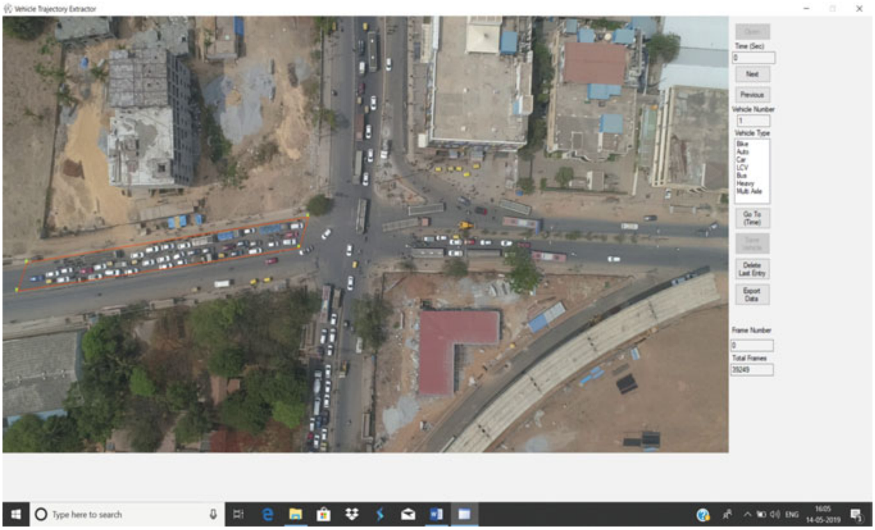


Fig. 8 Tracing vehicle trajectory information using traffic data extractor

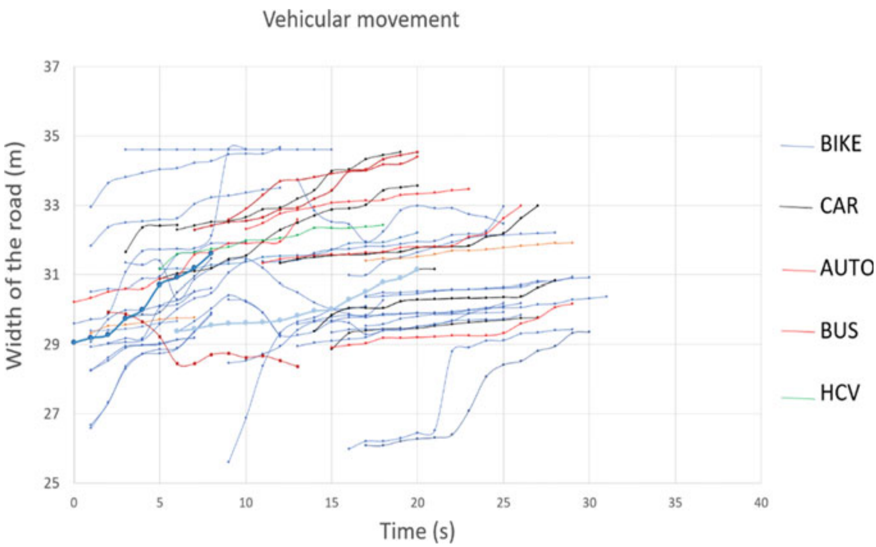


Fig. 9 Vehicle trajectory data of different class of vehicles obtained through traffic data extractor

and seep through the stationary or almost stationary large sized vehicles and approach to the intersection stop line. This phenomenon is represented here in terms of time, i.e. the particular time of the red signal after which the seepage behaviour is observed in each cycle.

Table 5 Representation of time duration within red time after which the seepage behaviour observed in each cycle

Signal cycle	Time duration after which the seepage behaviour observed (s)
1	40
2	90
3	60
4	75
5	25
6	65
7	105
8	115
9	125
10	90
11	105
12	110

The below graph is representing the volume count of different class of vehicles within the seepage zone of 30 m from the intersection stop line. The reference line is indicating the time period within the red signal, i.e. 40 s before which the vehicles approach to the seepage zone without any seepage behaviour or lane indiscipline. After the 40 s of red time, when the seepage zone is occupied by different class of vehicles, it is observed that the small sized vehicles like two-wheelers, etc., density is increasing and the density of other class of vehicles within the seepage zone is constant. This phenomenon is indicating that the small vehicles are not following the queue and seeping into the voids present in the seepage zone. The time interval of the red signal after which seepage behaviour is observed in each signal cycle is represented graphically, and values are mentioned in the Table 5 (Fig. 10).

In the above graphs, the *x*-axis represents the red signal timing in seconds and *y*-axis represents the density of the vehicles in the seepage zone. Each class of vehicles is represented in different colour in the graph. The reference line indicates the red time, where the seepage zone is occupied fully by the vehicles. After this time interval, there will be an obstruction for the vehicles to get into the seepage zone as the seepage zone is occupied by the vehicles. But the small sized vehicles instead of queueing up behind the vehicles, they seep through the gaps and enter the seepage zone and this increase in density is specified in the graph, where all large sized vehicles in the seepage zone remain constant, the small sized vehicles density are increasing. The reference line indicates that before the line or time, the vehicles enters the seepage zone with queue discipline and after this time or line, the increase in vehicles within the seepage zone indicates the seeping action of vehicles by abiding the queueing rule.

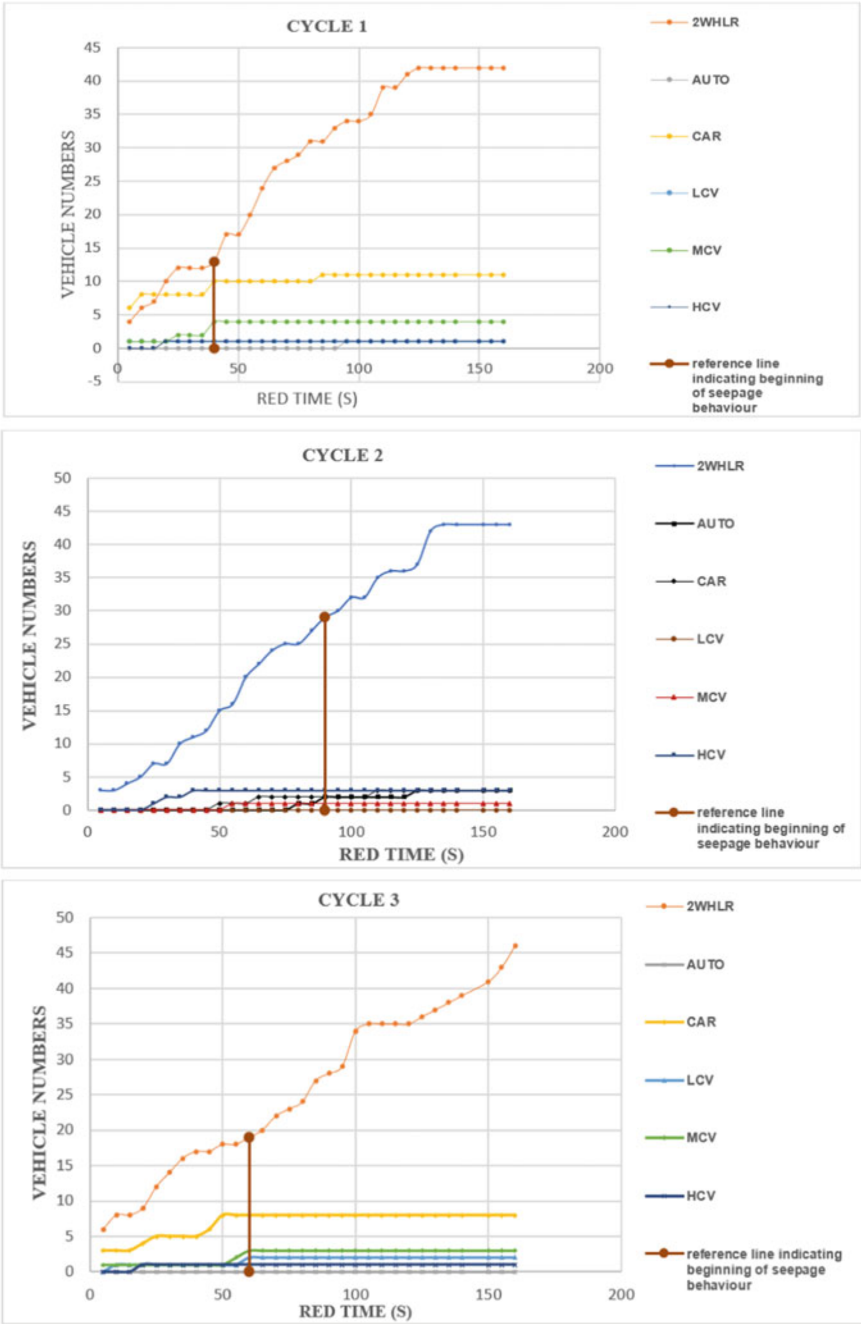


Fig. 10 Graphical representation of vehicles, seepage zone and red time after which the small sized vehicles fill the voids within seepage zone in each red signal cycle

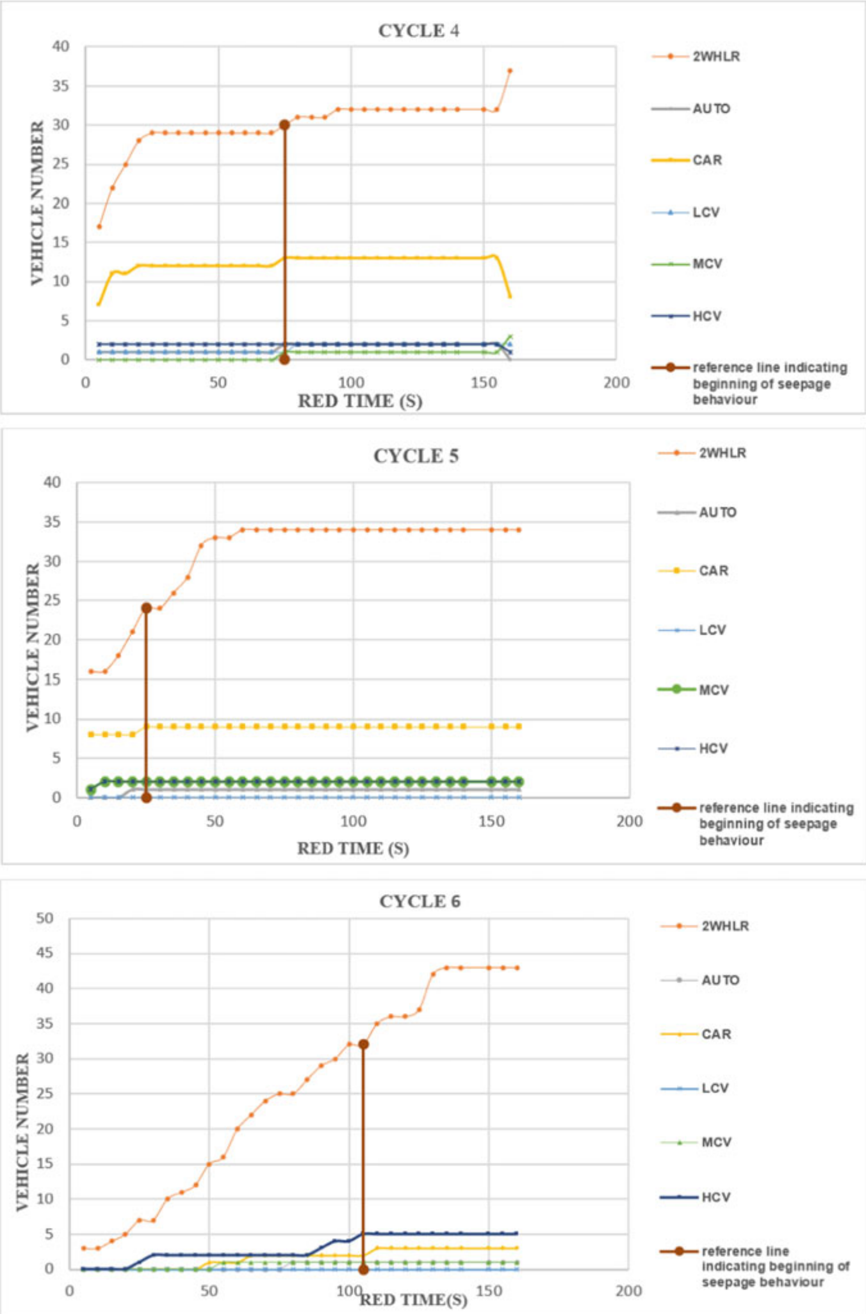


Fig. 10 (continued)

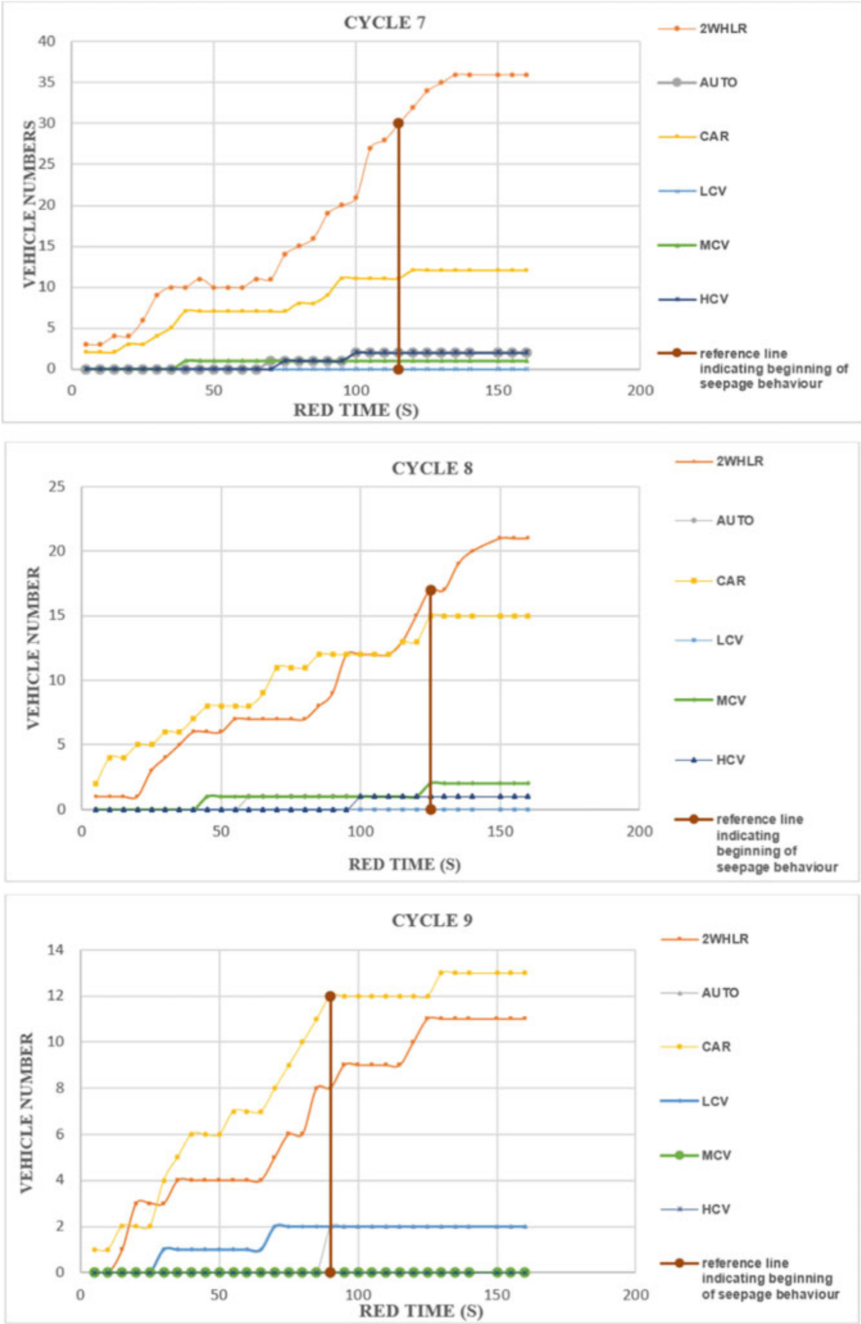


Fig. 10 (continued)

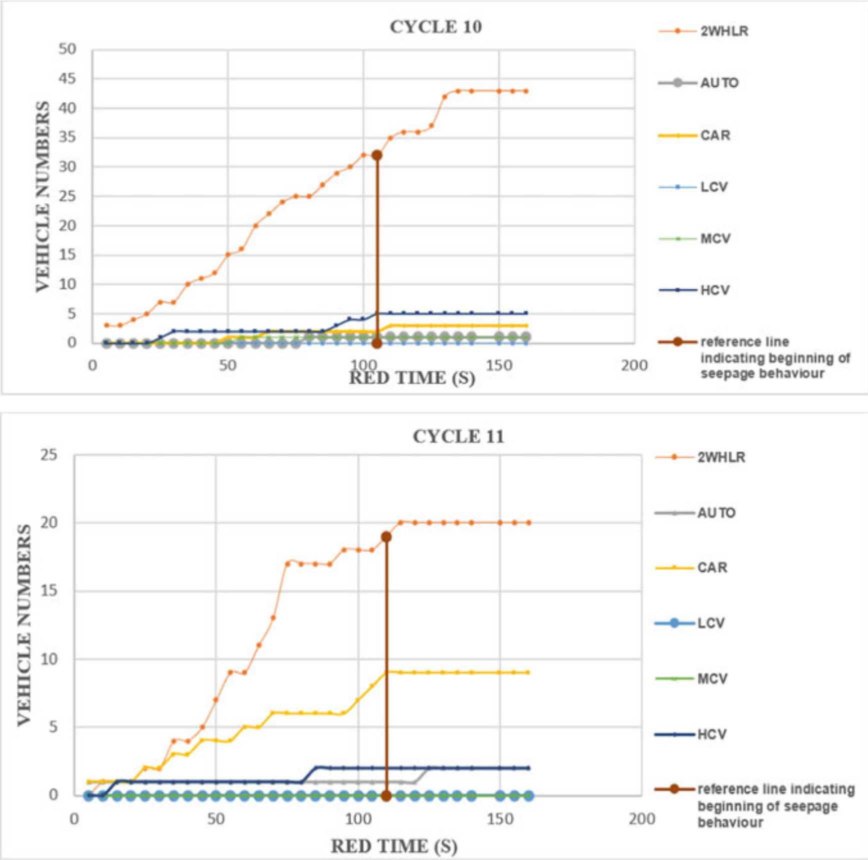


Fig. 10 (continued)

5 Results and Conclusion

In this study, an analysis was made to determine the queue area, area occupied by the vehicles in the queue. The seepage behaviour of small sized vehicles and their effects on other traffic characteristics were analyzed, and the following findings were concluded as below:

- The Hope farm signalized intersection connects the links which have a distinct land use of industrial and commercial activities, etc., which result in high intensity of traffic which comprises of different class of vehicles.
- This intersection comes under LOS *F* based on the analysis made as per Indo-HCM.
- The spot speed of the vehicles discharging at the intersection was found to be for:

Through movement	Right movement	Left movement
Minimum speed 6 KMPH	Minimum speed 6 KMPH	Minimum speed 5.5 KMPH
Maximum speed 13 KMPH	Maximum speed 14.5 KMPH	Maximum speed 11 KMPH

- Further based on the analysis made on the queue length, actual queue area and the area occupied by the vehicle during the red truncation, it is found that there is a significant difference between the queue area and the area occupied by the vehicles within the queue area in each signal cycle. The values are represented in the table below, which is indicating the inefficient occupancy of the vehicles in the queue.

Signal cycle	Average occupancy rate of vehicles (%)
1	49.3
2	48.74
3	51.2
4	55.5
5	74.81
6	66.03
7	51.87
8	58.25
9	57.99
10	64.01
11	76.5
12	46.63

- Based on the analysis made on the change in density of vehicles in the seepage zone, it was found that two-wheelers are involved in seepage action in each signal cycle that was analyzed and cars were involved in seepage action in cycle 1, 6, 7 and 9.
- Based on the vehicle trajectory data obtained, it is seen that two-wheeler has high lateral movement along the width of the road representing the seepage behaviour.
- The seepage behaviour also represents the unsafe vehicular movement between the vehicles resulting in increased conflict points, making it an accident-prone region.

References

1. Oketch T (2003) Modeled performance characteristics of heterogeneous traffic streams containing non-motorized vehicles. Annual Meeting Preprint 03–0721, Transportation Research Board, Washington D.C.
2. Wang J, Yang J, Li Q, Wang Z (2004) The effect of nonstandard vehicle's special behaviour on mixed-traffic modelling. In: Applications of advanced technologies in transportation engineering, pp 589–594
3. Krajzewicz EN, Nagel DK, Wagner P (2006) Describes the simulation of traffic flow with queues
4. Sperley M, Pietz AJ (2010) Motorcycle lane–sharing: literature review. Technical Report, OR–RD–10–20, Oregon Department of Transportation Research Section
5. Asaithambi G, Kumar RVY, Sivanandan R (2013) Evaluation of exclusive stopping space for motorcycles at signalized intersections under mixed traffic conditions using simulation model. In: 92nd Transportation Research Board Annual Meeting
6. Aupetit S, Espie S, Bouaziz S (2014) Naturalistic study of riders' behaviour in lane–splitting situations. Cognition, technology & work, pp 1–13
7. Agarwala A, Lammel G (2015) Seepage of smaller vehicles under heterogeneous traffic conditions. In: 4th International workshop on agent-based mobility, traffic and transportation models, methodologies and applications, ABMTRANS 2015, Salzufer 17–19, 10587. Berlin, Germany
8. Agarwala A, Lammel G (2016) Modelling seepage behaviour of smaller vehicles in mixed traffic conditions using an agent-based simulation. Transport Dev Econ 2.2:1–12. https://doi.org/10.1007/s40890-016-0014-9

Transit-Oriented Development to, for, and in Delhi

Patnala Phani Kumar, Manoranjan Parida, and Chalumuri Ravi Sekhar

1 Transit-Oriented Development: Living with Transit

Any city aims at achieving lower transportation costs, time, and distance, affordable housing options for all, efficient use of land space, and well-connected streets to accomplish good life to its inhabitants. When a city achieves its broad vision, it becomes competitive, efficient, economically active, and resilient to any urban problems. The neighborhoods of those cities will have the ability to accommodate higher population and employment densities. The residents living in those neighborhoods will be facilitated with high accessibility to opportunities through sustainable modes of transportation. The road networks will be free from traffic congestion and CO_2 emissions. It enables the government bodies to invest funds in providing high-quality public spaces and neighborhoods by capturing land values. Overall, such a vision integrates urban structure and transportation, which now is more relevant to rapidly-expanding cities than never before.

Transit-oriented development (TOD) is a sustainable policy approach that can achieve such a broad vision. TOD is the creation of high density, a mix of land uses, pedestrian and cycling-friendly streets, high accessibility to job opportunities, proximity to transit, appropriate management of travel demand, and affordable housing developments around transit facilities within a city [1–5]. It embraces the idea that locating good urban structure elements around transit facilities promotes the use of transit, walking, and cycling. A successful TOD at the metropolitan level is inclusive

P. Phani Kumar (✉) · M. Parida
Centre for Transportation Systems, Indian Institute of Technology Roorkee, Roorkee, Uttarakhand 247667, India
e-mail: pkumar@ct.iitr.ac.in

C. Ravi Sekhar
Transportation Planning and Environment Division, CSIR-Central Road Research Institute, New Delhi 110025, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_19

and is often an effective way to create land value at the neighborhood level. TOD creates a 'legible' place and evokes the spirit of enclosure, human linkage, and better quality of life [6–8].

Research on TOD can be traced back to 1990s when Calthorpe [1] introduced this term in his book 'The Transit Metropolis.' Since then, numerous researchers, planners, and policymakers have considered TOD as an urban planning strategy that acts as an effective means in reducing travel distances, vehicle ownership, and living costs [9–11]. In this context, substantial efforts have been made to evaluate the outcomes of TOD [12–14]. These studies prove that building TOD around transit facilities will lead to copious benefits in user, environment, and policy aspects. For instance, Cao and Lou [15] found that TODs tend to transit's 'value uplift.' Arrington and Cervero [16] proved that TOD neighborhoods can reduce 44% of car ownership in Washington DC and deliver a substantial 'ridership bonus' to transit systems. A study by Nasri and Zhang [13] evident that living in TOD areas of Washington DC and Baltimore has reduced 21%, and 38% of vehicle miles traveled, respectively. According to Renne [6], living in transit-oriented neighborhoods reduces car usage and traffic congestion. There is growing evidence on TOD that it creates 'social equity' and promotes 'public health' [7, 17].

From the three decades, TOD has been implemented in many cities worldwide and has been successful in achieving inclusive urban design and positive travel behavior. However, these success stories of sustainable TOD cities reveal a wide disparity in the application of urban structure elements [18, 19]. US cities, for example, focused on centralizing densities around transit stops, whereas Japanese cities aimed at distributing densities across transit corridors. Similarly, cities in other developed countries concentrated on their priority urban structure elements (e.g., cycling accessibility in the Netherlands) to achieve positive travel behavior. For rapid developing Indian cities, TOD needs to be beyond redevelopment of urban structure for conducive neighborhoods. An extensive analysis is necessary for rapidly-expanding cities to be successful in TOD planning.

In 2016, the government authorities of Delhi have proposed a TOD policy to facilitate TOD planning along with metro rail transit (MRT) corridors. These government bodies agree that TODs compliment MRT expansion within the city. But other stakeholders who belong to planning, policy, and research departments are skeptic on performance and credibility of TOD. Besides, this policy must consider suitable planning aspects of TOD or examine the impacts of TOD projects over a long period. In addition, the policy document must incorporate an evaluation procedure to measure existing levels of TOD and to establish a standard for planning and implementing TOD throughout study region. Therefore, stakeholder must reason improvements and introduce complementary strategies that allow successful TOD planning.

Therefore, the present study answers three major questions, which help fortify existing TOD policy in Delhi. Firstly, a broad review of existing TOD policies in India and abroad to answer, what interests government authorities in introducing TOD to Delhi? Secondly, a one-to-one consultation to stakeholders such as planners, policymakers, and researchers, in answering, which planning strategies better compliment TOD for Delhi? Thirdly, a TOD score at neighborhood levels will answer, what are the existing levels of TOD in Delhi? Appreciating and instituting this TOD planning framework in existing TOD policy is helpful to stakeholders to implement TOD in other Indian cities, more strategically.

2 Need for TOD Measurement Tool

Due to space constraints, any city rarely plans for Greenfield TODs. In general, urban structure in most cities replicate transit adjacent development (TAD), where transit facilities align across neighborhoods with available travel demand. However, any neighborhood possesses some levels of TOD-ness within [20]. The idea behind TOD planning is to (re)orient/(re)develop/renovate the existing urban structure toward transit facilities. A successful (re)development transforms a TAD to TOD, otherwise remains as TAD. It implies that TOD planning is not just to develop urban structure closer to transit facilities, but it is notably creating synergy between them. Therefore, policies that promote TOD planning may include an evaluation tool that expresses the existing levels of TOD-ness in the form of a TOD metric. This evaluation tool identifies special provisions according to underlying base conditions that guide (re)development within a neighborhood.

From the past three decades, TOD projects across the world were evaluated qualitatively or quantitatively by measuring changes in travel behavior [7, 8]. However, limited studies focus on extant levels of TOD-ness through a comprehensive measurement tool in the form of a single index or score [21]. A TOD index or score measures the degree to which a specific neighborhood orients toward transit facilities [10]. Such a metric measures TOD-ness and useful in comparing different neighborhoods, systematically, and objectively. Also, limited planning studies have spatially analyzed the two cornerstones of TOD (urban structure and travel behavior), which are both inherently spatial. Hence, several indicators that relate to urban structure and transportation need to be measured and aggregated into a TOD score that can further help planners, policymakers, and researchers in making long-term planning decisions.

3 What Is TOD to Delhi?

As Indian cities are experiencing rapid economic growth, Government of India (GoI) have initiated many urban reforming policies such as ‘Smart City’ framework for hundred cities, Atal Mission for Rejuvenation and Urban Transformation (AMRUT), Non-Motorized Transport (NMT), and Last Mile Connectivity to promote core infrastructure and provide decent quality of life to its citizens [22]. After these focused efforts through different policy programs, the GoI finds TOD as the last option to achieve sustainable mobility. New urban policies focus to plan and implement TOD in all such cities, where mass transit systems are existing. Hence, local and regional government bodies must allow TOD as a strategic tool by creating competent planning and policy framework for better quality of life in Indian cities.

Delhi government also sees TOD as a powerful tool to recast urban structure in conducive neighborhoods. Government agencies introduced TOD policy in the Master Plan for Delhi (MPD)—2021. In this policy, TOD is defined as 'any development, macro or micro that is focused around a transit node, and facilitates complete ease of access to the transit facility, thereby inducing people to prefer to walk and use public transportation over personal modes of transport.' As per these guidelines, TOD for Delhi is a development that enables pedestrian and cycling access to transit facilities. It is believed that such a development may lead to paradigm shift from private transport. The policy vision is focused on providing safe, affordable, and accessible living options within a maximum of 2000 m on both sides of MRT corridors.

However, a systematic measurement tool is still missing in the proposed TOD policy. As mentioned in the previous section, such a measurement tool guide stakeholders such as planners, policymakers, and builders while investing funds on infrastructure with prior knowledge on existing levels of TOD. However, measuring existing levels of TOD-ness will hinder repeating mistakes in investments [21]. Given this, the present study focuses on measuring existing levels of TOD-ness in 48 neighborhoods associated with MRT stations of Delhi using a 'TOD score.' Delhi metro is currently operating eight corridors connected with 250 stations through a network length of 343.43 km. Standing eighth-longest metro systems in the world, the neighborhoods of this city becomes a natural choice in measuring TOD score useful for TOD planning. Figure 1 shows existing MRT network and selected neighborhoods for the study. These 48 neighborhoods are potential TOD sites that were selected based on field observations and also future TOD projects as proposed by Delhi Development Authority (DDA).

4 TOD Definition for Delhi

TOD is a multifaceted planning approach, where planning and measurement of TOD involve large set of criteria/indicators. For example, Niles and Nelson [23] framed 16 indicators to explain the interaction between nodes and places in a TOD environment. Renne and Wells [21] mentioned 56 indicators categorized into five broad TOD aspects, namely urban structure, travel behavior, economic, environmental, and social. Later, Renne [6] added policy context as the sixth aspect. However, most of these studies limit their focus to investigate the linkage between travel behavior and urban structure while evaluating TODs. In this background, Cervero and Kockelman [2] evident that three criteria of urban structure, namely density, diversity, and design, showed significant effect on travel behavior. Later, Ewing and Cervero [3] found that 'distance' and 'destination' also contribute to travel behavior changes. Ewing et al. [5] finalized seven criteria, including demand and demographics, to the list. Literature suggests that a successful TOD must fulfill all these 'seven' criteria

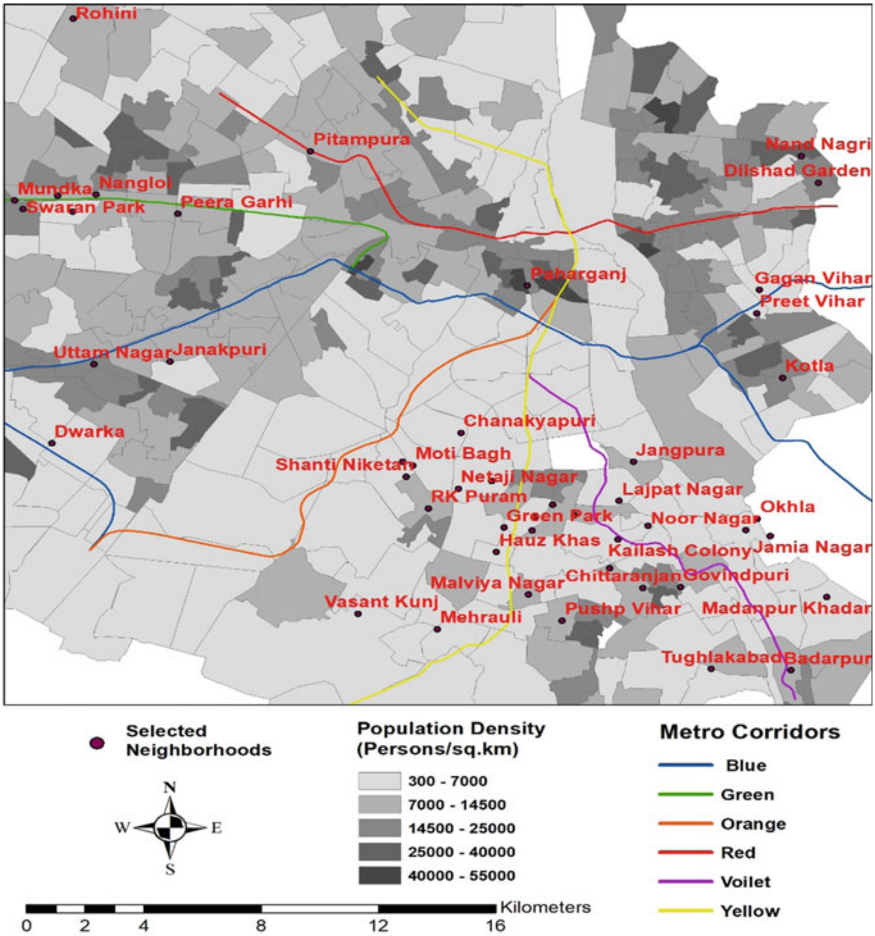


Fig. 1 Study Neighborhoods for Measuring TOD-ness

of urban structure [8]. Many researchers have conducted either qualitative analysis or quantitative analysis of available indicators to represent these criteria into a TOD metric. Table 1 demonstrates a summary of studies on TOD planning criteria and methods used.

From Table 1, it is evident that there is a disparity in criteria considered in different case studies, which signifies that criteria for TOD planning are context-sensitive. The planning criteria of one context may not be equally transferable to another context [19]. The TOD guidance document for Indian cities also mentioned design, network connectivity, multimodality, modal shift, place-making, balance as suitable criteria for TOD planning and implementation. Based on the extensive literature review of TOD planning studies and existing TOD policy for Delhi, this study has formulated

Table 1 Studies on TOD planning and methods used

Criteria	Wey et al. [24]	Aston et al. [25]	Motieyan and Mesgari [26]	Singh et al. [8]	Strong et al. [27]
Density	√	√	√	√	√
Diversity	√	√	√	√	√
Design	√	√	√	√	
Distance					
Destination	√			√	
Transit service				√	
Economic development			√	√	√
Travel behavior				√	√
Stakeholder partnership					
Housing choices					
MCDM approach	Fuzzy-ANP and GIS	Stepwise linear regression	Fuzzy-AHP	Spatial Multi-criteria analysis	AHP
Experts involved	Planner, Policymaker, Researcher	Researchers	Policymakers	Policymakers	Policymakers
Case study	Taipei, Taiwan	Melbourne, Australia	Tehran, Iran	Arnhem and Nijmegen, The Netherlands	Colorado, US

seven criteria suitable for measuring TOD-ness in the neighborhoods of Delhi. Table 2 gives descriptions of formulated criteria/indicators.

Despite the formulation of criteria, experts from planning, policy, and research play a decisive role in TOD planning. Literature suggests that different expert groups will have different perspectives and are critical for any decision-making problems. Multi-criteria decision-making (MCDM) methods are helpful in such critical decisions like TOD planning [7]. In this study, TOD experts from different groups such as government officers, industrial experts, and researchers were involved in planning process. Professional interest of these experts relates to TOD projects in Delhi. Since suitable TOD projects are missing in the Indian context, it is necessary to involve a broad range of expert groups for this study. For this purpose, 50 experts were communicated through emails and personal interviews.

Each expert was personally approached and asked for their relative preferences on planning criteria/indicators in an analytic hierarchical process (AHP) scale. We conducted reliability tests on all the expert decisions using the consistency index, and 31 (comprising 10 government officers, 9 industrial workers, and 12 researchers)

Table 2 Description of established criteria/indicators

Criteria	TOD guidance	Indicators	Measurement
Density	Balance	Population density	Number of persons residing within a unit buffer area
Diversity	Place-making	Entropy	Mix of land uses within a tract
Design	Design	Intersection density	Number of intersections within the unit buffer area
Distance	Modal Shift	Distance to transit	Euclidean distance from CBD to nearest MRT station
Destination	Multimodality	Accessibility to jobs	Number of jobs accessible using MRT within 45 min travel time
Demand	Network connectivity	Network length	Total road network length within the unit buffer area
Demographics	–	Household income	Average household income of individuals in a neighborhood

were found consistent and complete. The AHP relative preferences were converted to a fuzzy scale to handle uncertainty and vagueness in expert judgments (see Table 3). Fuzzy-AHP (FAHP) method was employed to evaluate expert decisions and calculate weights for criteria/indicators. Chang's extension theory is the most commonly used FAHP that has a series of mathematical formulations to convert fuzzy decisions into crisp weights [28]. All the calculations were conducted in MS Excel only.

Figure 2 shows the results of FAHP method. Weights to criteria at a group level are geometric mean of individual-level judgments and at a combined level is arithmetic mean of group-level judgments. There is clear evidence of divergence in expert group perspectives, whereas three expert groups converge in considering destination accessibility as criterion of prime importance for TOD planning in Delhi. At a combined level, the results suggest that Delhi government must consider destination (0.319),

Table 3 Fuzzy conversion Scale of AHP relative preferences

Definition	AHP scale		FAHP scale	
	Numerical value	Reciprocal value	Fuzzy value	Reciprocal fuzzy value
Extremely important	9	1/9	(9, 9, 9)	(1/9, 1/9, 1/9)
Very important	7	1/7	(6,7,8)	(1/6, 1/7, 1/8)
Strongly important	5	1/5	(4,5,6)	(1/4, 1/5, 1/6)
Moderately important	3	1/3	(2,3,4)	(1/2, 1/3, 1/4)
Equally important	1	1	(1,1,1)	(1, 1, 1)

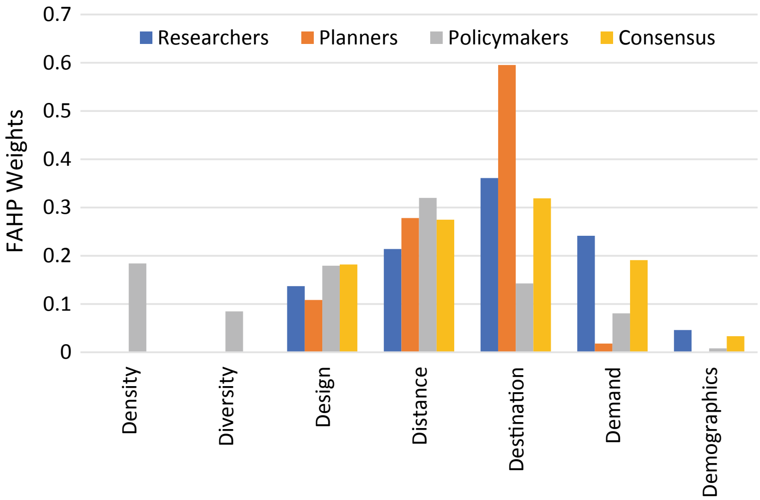


Fig. 2 Expert group perspectives on planning criteria using the FAHP method

distance (0.274), demand (0.191), and design (0.182), as vital criteria. Experts opine that two other criteria (density and diversity) are least important criteria, hence received 'zero' values using FAHP method. As explained in Chang [28], some criterion receives 'zero' value when other criteria are more important than them. Also, the neighborhoods of Delhi already comply mixed-use, compact, and dense urban structure [29].

At a group level, the results show noticeable differences in perspectives among expert groups. It is attributed to the following reasons. Firstly, policymakers always search for available case studies while planning any kind of urban developments. Since TOD planning is naïve in Indian context, policymakers made careful assessments for all criteria. Thus, no criteria have attained 'zero' values from judgments of this group. Secondly, planners of developing nations see TOD as an opportunity to distribute excess population along transit corridors, instead concentrating density around transit stations. They opine that station-level planning approach will worsen the traffic conditions at already congested urban streets [30]. Thirdly, researchers keep conscious about 'carrying capacity of city' while assessing TOD criteria [29]. For them, excess population growth will enlarge the gap between demand and supply trends. They believe that over-densification in rapidly-expanding cities is not a viable step to attain sustainability mobility. These may be the reasons why planner and researcher groups felt that 'density and diversity' criteria downplay in TOD planning.

Based on the results obtained from this decision-making process, the present study modified MoUD TOD definition as 'a land-use approach where opportunities are highly accessible through the walk and cycling-friendly streets within proximity to/from transit stations and that maximizes transit demand. It can be achieved only by diplomatic concern of local and central bodies on TOD planning at a regional scale.' This modified definition is widely applicable to high-dense neighborhoods of

Indian cities, while other Indian cities can employ this methodological framework in identifying priority criteria for TOD planning.

5 What Are the Existing Levels of TOD in Delhi?

Arriving at a comprehensive TOD score involves measurement and aggregation of indicators that represent each criterion. Numerous studies measured TOD-ness at neighborhood level in various case studies using different indicators [7, 8]. Table 4 presents a list of indicators within each criterion. These indicators were measured within each neighborhood buffer using spatial analysis in ArcGIS Software (Version 10.2.2). Unlike previous studies that considered 800 m buffer area, this study has considered 1200 m distance from CBD as appropriate neighborhood buffer to calculate TOD indicators [28]. The indicators ‘population density’ and ‘entropy’ were excluded in calculation of TOD scores, due to their less importance in TOD planning for Delhi. Indicators such as distance to transit, intersection density, and network density were measured by digitizing road networks and MRT network from Google Maps. The average household income values were obtained from SUSTRANS household survey data [31]. Census [32] provided the employment data for each neighborhood. Figures 3 and 4 show network density and job accessibility within neighborhood buffers, respectively. All indicator values represent existing levels of TOD-ness within each neighborhood buffer and help in arriving at TOD scores.

The estimated values of indicators were aggregated to obtain TOD scores for neighborhoods. These indicators were standardized and assigned weights according to their importance for TOD planning. Before producing TOD scores, a correlation analysis was conducted among five indicators to check their relationships. The analysis showed that all indicators followed expected directions and independent to each other. All indicators represented positive correlations except for destination

Table 4 Weights and description of measurement tools (indicators)

Criteria	Indicator	Weights	Source
Design	Intersection density (Nos./km^2)	0.182	Google maps
Distance	Distance to MRT (km)	0.274	Google maps
Destination	Job accessibility (Nos./45 min)	0.319	Census and DMRC
Demand	Network density (km/km^2)	0.191	Google maps
Demographics	Household income (INR.)	0.033	SUSTRANS data

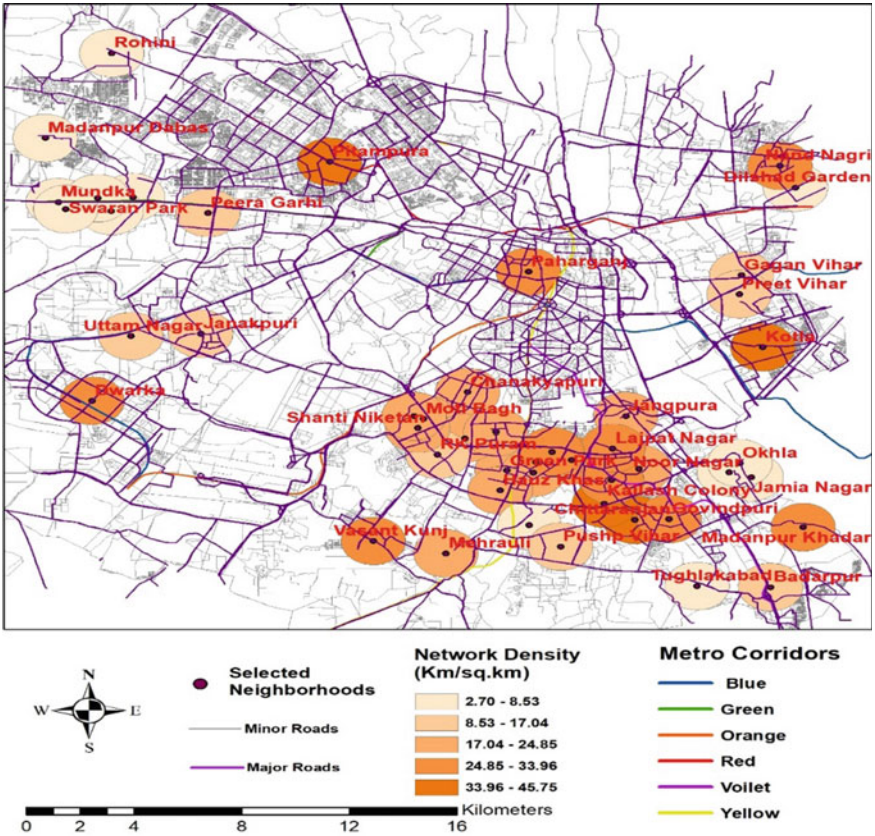


Fig. 3 Network density indicator measured within selected neighborhood buffers

accessibility, for example, higher distance from MRT lowers the accessibility to jobs. Therefore, these indicators were standardized to a scale of minimum 0 to maximum 1 using Eqs. 1 and 2 as

For positively correlated indicators,

$$\overline{x} = (x - x_{\min})/(x_{\max} - x_{\min}) \tag{1}$$

For negatively correlated indicators,

$$\overline{x} = (x_{\max} - x)/(x_{\max} - x_{\min}) \tag{2}$$

where $\overline{x}$ is the rescaled indicator.

A composite TOD score was calculated for each neighborhood buffer using the weight sum method of weights and standardized values of indicators. As mentioned in the previous studies, higher TOD scores indicate higher levels of TOD-ness within

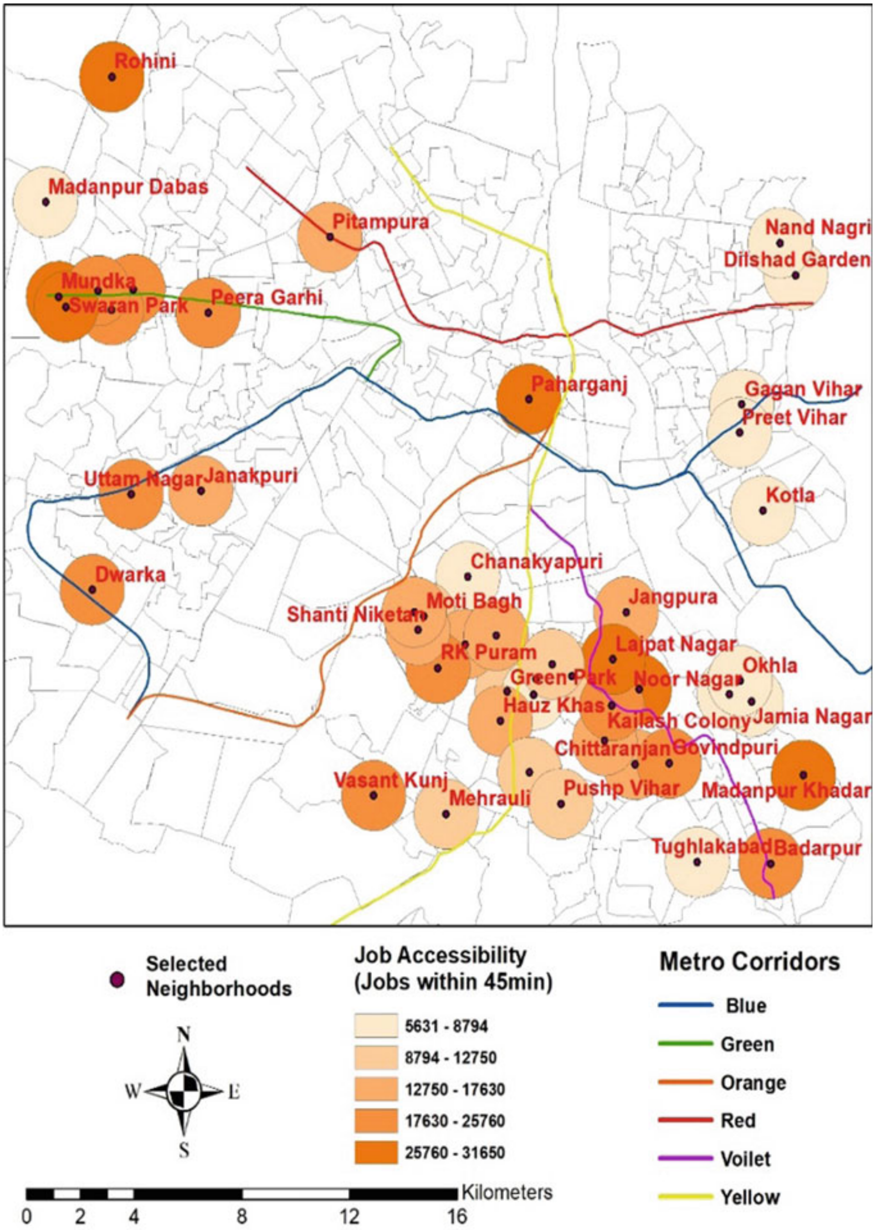


Fig. 4 Job accessibility indicator measured within selected neighborhood buffers

[8]. Figure 5 represents TOD scores of selected neighborhoods across Delhi City. Using this TOD measurement tool, each neighborhood was assigned to a score between 0 and 1, where TOD scores of Uttam Nagar and Chanakyapuri are 0.77 and 0.13, respectively. These scores illustrate that Uttam Nagar and Chanakyapuri have higher and lower levels of TOD-ness as compared to 47 other neighborhoods, respectively. This is how different neighborhoods can be compared in terms of TOD

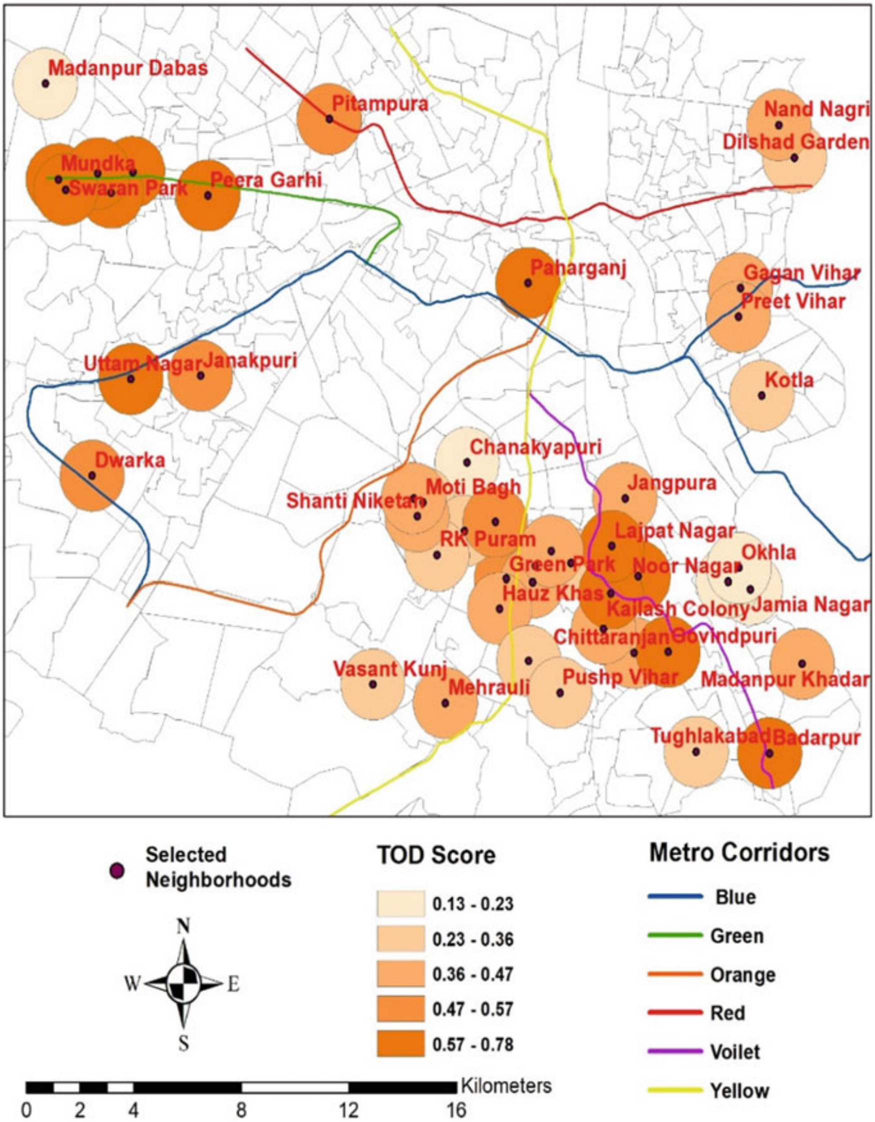


Fig. 5 Existing relative TOD scores of selected neighborhoods in Delhi

scores. The neighborhoods closer to transit facilities attained higher TOD scores, as compared to farther neighborhoods. This phenomenon is well-pronounced for neighborhoods close to violet and green corridors.

While the obtained TOD scores serve as a useful tool in identifying the best and worst neighborhoods, it does help in enhancing TOD levels within each neighborhood. Strengths and weaknesses within each neighborhood can be identified when comparing with best-performing neighborhood as ideal case study. However, it is to be noted that these TOD scores are relative in nature, and even best-performing neighborhood does not stand best when compared to ideal TODs. These TOD scores of neighborhoods can be improved by employing immediate and long-term action plans. The next section illustrates some examples on how Delhi government agencies can use TOD scores for improving TOD-ness at neighborhood level.

6 What's Next? Action Plans!

Now that we have identified neighborhoods with specific TOD scores (lower or higher), it remains unclear on how these TOD scores can be transferable to planning and implementation. Thus in this section, we have proposed some immediate and long-term action plans that suggest for the (re)development of neighborhoods toward TOD. For detailed illustration, five neighborhoods, namely Madanpur Dabas, Dwarka, Kotla, Chanakyapuri, and Dilshad Garden, were considered to suggest action plans based on existing TOD levels. Table 5 provides example neighborhoods and their existing TOD levels in terms of indicators and overall TOD score.

The following action plans are recommended for each neighborhood:

Chanakyapuri. This neighborhood has high land-use diversity; however, this strength cannot reflect on the overall TOD score. While all the other indicators show lower values compared to others. The following recommendations may improve the TOD score:

Table 5 TOD indicators and scores for example neighborhoods

Location name	Distance to transit (km)	Network length (km/km^2)	Population density (persons/km^2)	Entropy (0–1)	Intersection density (Nos./km^2)	Job Access by MRT (Nos./45 min)	Relative TOD score
Chankyapuri	2.5	19.05	10,400.00	0.730	18.50	7192.51	0.13
Dilshad Garden	1	8.53	43,737.38	0.412	47.00	7097.69	0.36
Dwarka	0.5	29.43	16,529.12	0.622	26.50	22,145.93	0.57
Kotla	1.6	39.97	51,754.36	0.455	100.50	7836.48	0.35
Madanpur	2.7	33.96	21,505.71	0.644	51.00	9781.28	0.22

1. Add more jobs to promote density and transit usage.
2. Promote surplus living options through new housing developments near transit facilities to withstand high densities.
3. Improve transit accessibility to give it a competitive advantage over private transport.
4. Add additional road network for bike lanes to increase walkability and cyclability.

Dilshad Garden. While there is already high population density in this neighborhood, make use of underutilized land space into mix of land uses that promote employment and transit ridership. This neighborhood needs more network length to enable walking and cycling. Additionally, housing options for all income people will balance the communities and help attract new businesses within the neighborhood.

Dwarka. This neighborhood has a TOD score of more than 0.5 and accessible to 22,000 jobs within 45 min travel using MRT. These neighborhoods are high-potential TOD sites when there is an increase in population density. The following action plans may improve neighborhood quality:

1. Add additional housing distributed around the transit stations to increase population density and boost transit ridership.
2. Attract business, employment, and amenities to offer more destinations within the neighborhood.
3. Implement parking maximums to decrease car ownership.
4. Provide housing options for low-income people, who mostly rely on public transit.

Kotla. This neighborhood has a TOD score of less than 0.5 but carries very high population density values. Such neighborhoods are high-potential TOD sites when there is a balanced distribution in population density. Therefore, government agencies must invest in vertical mix construction while safeguarding green zones. The road network length must have made use of improving walking and cycling facilities.

Madanpur Dabas. There is low job accessibility in this village due to a higher distance from MRT. Some of the action plans include: providing transit facilities proximity to such neighborhoods, which will increase job accessibility and promote transit usage; making use of vacant land spaces for vertical construction around transit stations; attracting business opportunities to offer new destinations within the neighborhood.

7 Conclusions

Delhi government promotes sustainable transportation in its neighborhoods, and TOD is found to be an eminent strategy to achieve its goal. For TOD planning, it is essential to quantify and understand existing levels of TOD-ness in neighborhoods using a TOD score. Since the proposed TOD policy lacks a comprehensive measurement tool, local and regional authorities rely on their own metrics to prioritize and fund TOD projects. Thus, TOD development authorities require generalized TOD metrics to promote standardize long-term planning in the upcoming TOD projects. To fulfill this gap, the present study proposed a measurement tool using existing urban structure in 48 neighborhoods of Delhi. This conceptual framework is an effective way of measuring TOD-ness at neighborhood level. The obtained TOD scores are useful in identifying opportunities for enhancing urban structure susceptible to TOD planning.

By making neighborhoods accessible to jobs using MRT, appropriate management of travel demand, walkable with quality housing, and business opportunities, the government agencies improve the quality of life for Delhi citizens. By improving transit facilities close to neighborhoods, the Delhi City can become an ideal case study in the country for TOD. Strong TODs not only focus on redevelopment of urban neighborhood structure but also consider the users who use transit facilities for daily trips. However, it is to be noted that the idea behind this TOD scoring tool is not only to assess urban structure in each neighborhood but also to balance the assets and flaws in that neighborhood. The TOD neighborhoods can complement one another and compensate for characteristics that certain neighborhoods may lack. Such holistic thinking may identify prospects to locate amenities and infrastructure within neighborhoods.

While this study carries certain limitations, firstly, the evaluation tool can be improved by including indicators from other aspects of TOD, such as travel behavior, policy, social, and environment. We have used criterion from an urban structure based on available data and on our understanding of how strongly those indicators contribute to the TOD score. Secondly, the limitation of the FAHP method has resulted in 'zero' values to certain most important criteria (density and diversity). Such results are misleading and omitted those criteria in further analysis. Enhanced FAHP can handle those 'zero' values. Thirdly, the TOD scores are relative. Since TOD is context-sensitive, the obtained TOD scores are unable to compare with the universal case studies. Standardized indicators can serve this purpose. For a more rigorous application of these TOD scores, this tool must consider criteria that belong to travel behavior, environment, social, economic, and policy aspects.

Acknowledgements This paper is part of a doctoral research work, which is supported by the scholarship from the Ministry of Human Resource and Development (MHRD), India. The authors also acknowledge the opportunity to submit the research work at the 5th Conference of the Transportation Research Group of India held at Bhopal (India) from December 18–21, 2019, which forms the basis of this article.

References

1. Calthorpe P (1993) The next American metropolis: ecology, community, and the American dream. Princeton Architectural Press
2. Cervero R, Kockelman K (1997) Travel demand and the 3Ds: density, diversity, and design. Transp Res Part D Transp Environ https://doi.org/10.1016/S1361-9209(97)00009-6
3. Ewing R, Cervero R (2001) Travel and the built environment. Transp Res Rec 1780:87–114
4. Ewing R, Cervero R (2010) Travel and the built environment. J Am Plann Assoc. https://doi.org/10.1080/01944361003766766
5. Ewing R, Tian G, Lyons T, Terzano K (2017) Trip and parking generation at transit-oriented developments: Five US case studies. Landsc Urban Plan 160:69–78. https://doi.org/10.1016/j.landurbplan.2016.12.002
6. Renne JL (2009) Evaluating transit-oriented development using a sustainability framework: lessons from Perth's network city. Planning Sustainable Communities: Diversity of Approaches and Implementation Challenges, 115–148. Retrieved from http://www.vtpi.org/renne_tod.pdf
7. Singh YJ, Lukman A, He P, Flacke J, Zuidgeest M, Van Maarseveen M (2015) Planning for transit-oriented development (TOD) using a TOD index. Transportation Research Board 94th Annual Meeting
8. Singh YJ, Lukman A, Flacke J, Zuidgeest M, Van Maarseveen MFAM (2017) Measuring TOD around transit nodes—towards TOD policy. Transp Policy 56:96–111. https://doi.org/10.1016/j.tranpol.2017.03.013
9. Cervero R (2002) Built environments and mode choice: toward a normative framework. Transp Res Part D: Transp Environ 7(4):265–284. https://doi.org/10.1016/S1361-9209(01)00024-4
10. Evans JE, Pratt RH (2007) Transit-oriented development, TRCP's traveler response to transportation system changes handbook series. Transportation Research Board, USA
11. CTOD (2010) Why transit-oriented development and why now? Reconnecting America, US
12. Cervero R, Murakami J (2009) Rail and property development in Hong Kong: experiences and extensions. Urban Studies. https://doi.org/10.1177/0042098009339431
13. Nasri A, Zhang L (2014) The analysis of transit-oriented development (TOD) in Washington, DC, and Baltimore metropolitan areas. Transp Policy 32:172–179
14. Chen F, Wu J, Chen X, Zegras PC, Wang J (2017) Vehicle kilometers traveled reduction impacts of transit-oriented development: evidence from Shanghai City. Transp Res Part D Transp Environ https://doi.org/10.1016/j.trd.2017.07.006
15. Cao X, Lou S (2018) When and how much did the green line LRT increase single-family housing values in St. Paul, Minnesota? J Plann Educ Res 38(4):427–436
16. Arrington GB, Cervero R (2008) TCRP Report 128: effects of TOD on housing, parking, and travel. Transportation Research Board, US
17. Jeihani M, Zhang L, Ardeshiri A, Amiri, A, Nasri A, Zamir KR, Baghaei B (2013) Development of a framework for transit-oriented development (TOD) (No. SP209B4N). State Highway Administration, Maryland
18. Voulgaris CT, Taylor BD, Blumenberg E, Brown A, Ralph K (2016) Synergistic neighborhood relationships with travel behavior: an analysis of travel in 30,000 US neighborhoods. J Transp Land Use 10(1):437–461. https://doi.org/10.5198/jtlu.2016.840
19. Thomas R, Bertolini L (2015) Defining critical success factors in TOD implementation using rough set analysis. J Transp Land Use 139–154. https://doi.org/10.5198/jtlu.2015.513
20. Laaly S, Jeihani M, Lee YJ (2017) A multiscale, transit-oriented development definition based on the context-sensitive paradigm. Transp Res Rec 2671(1):31–39
21. Renne J, Wells J (2005) Transit-oriented development: developing a strategy to measure success. Research Results Digest 294. Transportation Research Board, Washington, DC
22. Ministry of Urban Development (MoUD) (2016) TOD Guidance Document http://www.sutpindia.com/skin/pdf/TOD%20Guidance%20Document%20compressed.pdf
23. Niles J, Nelson D (1999) Measuring the success of transit-oriented development: retail market dynamics and other key determinants. In: American planning association 1999 annual conference

24. Wey WM, Zhang H, Chang YJ (2016) Alternative transit-oriented development evaluation in sustainable built environment planning. Habitat Int 55:109–123. https://doi.org/10.1016/j.habitatint.2016.03.003
25. Aston L, Currie G, Pavkova K (2016) Does transit mode influence the transit-orientation of urban development?—an empirical study. J Transp Geogr 26
26. Motieyan H,Mesgari S (2017) Towards sustainable urban planning through 27 transit-oriented development (a case study: Tehran). ISPRS Int J 28 Geo-Inf 6(12):402
27. Strong KC, Ozbek ME, Sharma A, Akalp D (2017) Decision support framework for transit-oriented development projects. Transp Res Rec J Transp Res Board. https://doi.org/10.3141/2671-06
28. Chang D-Y (1996) Application of the Extent analysis method on Fuzzy AHP. Eur J Oper Res. https://doi.org/10.1016/0377-2217(95)00300-2
29. Suzuki H, Cervero R, Iuchi K (2013) Transforming cities with transit. Transform Cities Transit. https://doi.org/10.1596/978-0-8213-9745-9
30. Kumar PP, Ravi Sekhar C, Parida M (2018) Residential dissonance in TOD neighborhoods. J Transp Geogr 72:166–177. https://doi.org/10.1016/j.jtrangeo.2018.09.005
31. Central Road Research Institute (CRRI) (2017) Development and application of technologies for sustainable transportation (SUSTRANS). Council of Scientific & Industrial Research (CSIR), India. 12th Five Year Plan Network Project (ESC-0106)
32. Census (2011) Census of India 2011. http://www.censusindia.gov.in/2011-provresults/prov_data_products_ani.html

Study on Impact of Newly Opened NH 66 Bypass in Kollam City

S. Shaheem, R. S. Remjish, and T. Ramakrishnan

1 Introduction

Construction of bypass around CBD area of major towns/cities is definitely required for relieving the severe congestion posed by the city dwellers and road users. However, it has its impacts on the immediate neighborhoods as well the urban areas for which the bypass is intended. Impacts on the immediate neighborhood pertain to change in property values, shift in land use pattern from say residential to commercial, etc. In the case of urban areas, there is lessening of traffic congestion, increase in safety of vehicles and pedestrian and also environmental improvement. The present paper, however, assesses the impact of bypass on the traffic flow and accident scenario variation in the urban area. In the above context, it is quite important to assess how far the bypass has addressed the issues of traffic congestion, what are the inherent drawbacks in the newly opened bypass, the scope for further improvement for the smooth flow of traffic, safety issues faced by the road users plying along and across and the steps for needed for addressing these issues.

1.1 Study Area

Kollam city is one of the five corporate cities of Kerala State. It is located about 70 km from Thiruvananthapuram, the capital city of Kerala. Kollam city can be accessed through three National Highways viz. NH 66, NH 183 and NH 744. First, is the North–South Corridor connecting Kanyakumari and Mumbai, running parallel to coastline and passing through Kollam city. NH 183 is a radial road in the west–east direction starting from High school junction in Kollam city and ends at Thiruamangalam city

S. Shaheem (✉) · R. S. Remjish · T. Ramakrishnan
National Transportation Planning and Research Centre, Thiruvananthapuram, Kerala, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_20

in Tamil Nadu. NH 744, another radial road in the west–east direction starts from Chinnakkada junction in Kollam city and ends at Madurai in Tamil Nadu. Apart from these three major roads, there are a number of roads leading to places outside as well as within the city. The city roads are maintained by both PWD and the City Corporation. Figure 1 shows the location of NH 66 and NH 66 bypass in Kollam city.

The railway route network also connects the city to neighboring cities with Kollam junction railway station. Earlier, water way was the major transport mode of the district due to the presence of T.S canal, backwaters, Kallada River, Ithikkara River and Achankovil River. Due to encroachment and silting, the T.S canal is presently not in use. A major portion of the Kollam city is sandwiched between Arabian Sea and Ashtamudi Lake, and most of the roads run parallel to the sea.

In the absence of a bypass, most of the traffic from northern side of Kollam city destined to southern part of Kollam city pass through the congested city road corridors resulting in traffic congestion, transit delay, accidents and environmental pollution. In order to address the above issues, a bypass was constructed which connects Mevaram (southern part of the city) to Kavanad (northern part of the city), thus entirely bypassing the city. With the opening of bypass, large number of accidents has been reported in the bypass. Thus, the present paper focuses on the traffic impact of the bypass and the safety issues it has inherited.

2 Literature Review

Earlier studies [1] undertaken on impact of bypass deal with assessment of economic impacts on the settlements around the project area and quantify them through various statistical/econometric models. The present paper tries to quantify the major impacts—traffic congestion and safety—due to construction of a bypass by making use of the procedures laid down in the relevant guidelines of Indian Road Congress [2, 3] with appropriate modifications.

3 Study Objectives

Major objectives of the study as per the problem appreciation described above are;

- To assess the impact of newly opened NH 66 Bypass on city roads (NH 66)—traffic flow characteristics like journey speed, transit time, vehicle operating cost (VOC), accidents, etc.
- To assess the road safety issues along NH 66 Bypass and suggest suitable remedial measures.

Fig. 1 Kollam city bypass

4 Methodology Adopted for the Study

The methodology adopted for the study included a set of tasks which are graphically presented in Fig. 2 and briefed below:

4.1 *Data Collection*

Large quantity of observational data on traffic volume, parking and pedestrian volume, speed and delay characteristics, roadway characteristics, etc. were compiled from the ground on the city roads and newly constructed bypass.

To supplement the above data, pass traffic data and accident data pre- and post-construction stages from secondary sources like police traffic, past study report, etc.

Road safety audit was carried out to assess the safety deficiencies and work out remedial measures. RSA was carried out as per checklist given in 'IRC SP 88—2010: Manual on Road Safety Audit'.

4.2 *Data Analysis*

The data thus compiled were analyzed by drawing the following major inputs for both pre- and post-construction scenarios.

(i) Traffic volume on city roads and bypass in terms of Passenger car unit
(ii) Parking accumulation
(iii) Pedestrian vehicle conflicts
(iv) Crash data analysis

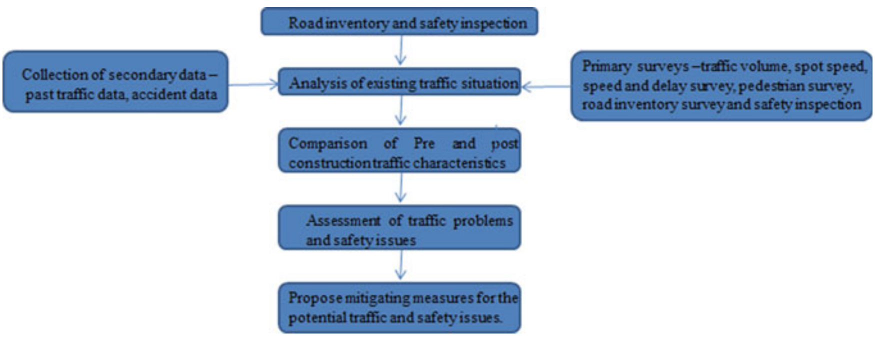


Fig. 2 Methodology adopted for the study

In order to make a comparison among pre- and post-construction stages so as to assess the impact, parameters like travel speed, vehicle operating cost, volume capacity ratio, pedestrian vehicle conflicts were worked out by making use of IRC guidelines. Table 1 gives the details of homogeneous sections identified.

Table 1 Details of homogeneous sections identified

S. No.	HS No.	Road section	Length (km)	Name of homogeneous section
1	1.1	NH 66 Bypass	3.1	Mevaram Junction—Ayathil Junction
	1.2		1.5	Ayathil Junction—Kallumthazham Junction
	1.3		4	Kallumthazham Junction—Kadavoor Junction
	1.4		2.1	Kadavoor Junction—Toll Plaza
	1.5		2.3	Toll Plaza—Kavanad Junction
2	2.1	NH 66	0.5	Kavanad Bypass Junction—Kavanad Junction
	2.2		2.4	Kavanad Junction—Vellayittambalam Junction
	2.3		1.5	Vellayittambalam Junction—Civil Station
	2.4		0.6	Civil Station—High School Junction
	2.5		0.5	High School Junction—Taluk Kachery Junction
	2.6		0.8	Taluk Kachery Junction—Chinnakkada Junction
	2.7		0.5	Chinnakkada Flyover
	2.8		0.2	Chinnakkada Flyover—Railway station Junction
	2.9		0.3	Railway station Junction—BP Petrol pump
	2.10		0.8	BP Petrol pump—SN College Gate
	2.11		0.5	SN College Gate—Kappalandimukku Junction
	2.12		1.55	Kappalandimukku Junction—Vendermukku Junction
	2.13		0.65	Vendermukku Junction—Pallimukku Junction (Koonambaikkulam road)
	2.14		0.1	Pallimukku Jn (Koonambaikkulam road)—Pallimukku Jn (Eravipuram road)
	2.15		2.4	Pallimukku Junction (Eravipuram road)—Mevaram Junction

5 Existing Situation Analysis

For the purpose of analysis, the entire road stretch of NH 66 was divided into 15 homogeneous sections and newly opened NH Bypass into four homogeneous sections. Traffic impact of NH Bypass on 15 homogeneous sections of NH 66 (Table 1) was examined.

5.1 *Road Network Characteristics*

The newly opened NH 66 Bypass from Mevaram to Kavanad has a total road length of approximately 13 km. The whole road stretch is constructed as standard two-lane highway with a carriageway width of 7 m with two-way traffic. Paved shoulders of about 1 m are provided on both sides and the whole stretch is devoid of footpath. Bus bays are provided with a length of about 200 m. Traffic signs and markings are found sufficient.

The National Highway 66 has about 13.3 km length, passing through the CBD area of Kollam city, from Mevaram to Kavanad. About 80% of the road stretches have 2-lane with carriageway width of 7–8 m. Remaining 20% of the road have four-lane divided carriageway with width of about 7 m each on both sides. All sections allow two-way traffic. Paved/Unpaved shoulders are available throughout the road stretch and about 70% of the road are provided without footpaths. The adjoining land use is predominantly commercial.

5.2 *Speed and Delay Characteristics*

Average Journey Speed. On NH Bypass, the average journey speed varied from 29.23 to 46.55 km/h. The average journey speed of the entire road stretch was 39.84 km/h during peak hours and 43.71 km/h during off-peak hours.

On NH 66 passing through the city, the average journey speed was observed to be in the range of 20 km/h to 39.31 km/h. The average journey speed of the entire stretch during peak hours was 27.92 km/h. Before the introduction of bypass, the average journey speed of NH 66 was 21.3 km/h during peak hours.

5.3 *Causes of Delay*

An analysis of the delays and causes of delay has revealed that waiting time at signals was the major cause of delay for traffic flow on both NH and NH Bypass. On bypass, during peak hours, total delay of about 141 s (12% of journey time) was observed because of traffic signals. On NH road, the observed total delay time was about 181 s (8% of journey time), caused mostly due to signals and vehicle crossings.

5.4 Traffic Volume at Major Intersections

In bypass road, Ayathil junction handled the maximum peak hour flow of 8248 PCU followed by 7492 PCU in Kallumthazham junction. Other junctions handled the peak flow in the range of 3700 PCU to 5000 PCU. In NH road, it could be seen that Taluk Kachery junction witnessed the maximum peak hour traffic flow of 7773 PCU. Other intersections handled peak hour traffic in the range of 5000 to 7000 PCU.

5.5 Vehicle Crash Data of NH Bypass

An analysis of the reported vehicle crash data during the three months period after the opening of 13 km NH reveals that 57% of the vehicles involved were two wheelers followed by 29% cars. 62% of the causalities were vehicle users while the rest of 38% were pedestrians. 13% of the persons involved died while 83% of road user experienced grievous injury. Majority of the crash victims belong to the age group of 15–45 years. 70% of the victims were males and 30% of were females.

5.6 Road Safety Audit (RSA)

Pre-opening Road Safety Audit was carried out on the Kollam Bypass to review the road safety features provided along the study stretch. Safety audit gives information about any hazardous conditions that may have been created during its lifetime such as encroachments, ribbon development or deterioration of road conditions, traffic conditions.

6 Impact Analysis

The study aims at evaluating the impact of newly opened NH Bypass between Mevaram and Kavanad Bypass junction in Kollam city. Two situations were considered and compared—'without the bypass road' and 'with bypass road'.

For the situation 'with NH bypass', all the observed traffic data, speed and delay data and crash data were used. In the 'without bypass road' situation, to estimate the traffic volume on NH 66, traffic passing through the Kallumthazham Junction on NH

Table 2 Average journey speed and travel time for passenger car along NH 66

S. No.	Parameter	With bypass	Without bypass	Percentage change
1	Average journey speed (km/h)	28.4	20.3	40% (increase)
2	Travel time (min)	28.1	39.3	28% (decrease)

Bypass (middle of bypass) was added to the existing traffic as these traffic otherwise would have passed through the city without the bypass.

The speed of the vehicle was determined using speed flow equations in IRC SP 30. The vehicle operating cost (VOC) is evaluated for both situations as per IRC SP 30.

From the data collected, the following parameters were considered for traffic impact assessment of bypass road.

(a) Travel speed and Travel time
(b) Volume/Capacity ratio (V/C ratio)
(c) Vehicle operating cost (VOC)
(d) Pedestrian safety on NH 66
(e) Vehicle crashes in NH 66

6.1 Travel Speed and Travel Time

It is found that the travel speed of the vehicles would be severely affected if the service of the bypass was not provided. Currently, in NH 66, all 15 homogeneous sections experience journey speed of more than 10 kmph with seven sections having journey speed in the range of 30–40 kmph. But, in the absence of bypass, six sections would have journey speed less than 10 kmph and seven sections have the journey speed in the range of 10–20 kmph. Overall, there is an increase in travel speed of 40% with the commissioning of bypass (Table 2).

With increase in speed, the travel time of the passenger cars in NH 66 has come down by around 28% with the introduction of the bypass during peak hours. Currently a passenger car can traverse from Mevaram to Kavanad through NH 66 in less than 30 min during peak hours, while, before the introduction of bypass, it was around 40 min.

6.2 Volume-Capacity Ratio (V/C Ratio)

Volume-Capacity ratio is an effective tool to express the intensity of traffic in road link. From the analysis, it was found that presently eight sections have V/C ratio value of more than three and five sections have the value in the range of one to two. But if the bypass has not provided, nine sections would have the V/C

Table 3 V/C Ratio value distribution in NH 66

S. No.	Homogeneous sections in NH 66	V/C ratio values	
		With bypass	Without bypass
1	Kavanad Bypass Junction–Kavanad Junction	3.05	4.25
2	Kavanad Junction—Vellayittambalam Junction	3.05	4.25
3	Vellayittambalam Junction—Civil Station	2.59	3.79
4	Civil Station—High School Junction	1.08	1.58
5	High School Junction—Taluk Kachery Junction	1.95	2.55
6	Taluk Kachery Junction—Chinnakkada Junction	1.98	2.48
7	Chinnakkada Flyover	3.06	4.26
8	Chinnakkada Flyover—Railway station Junction	3.06	4.26
9	Railway station Junction—BP Petrol pump	1.34	2.54
10	BP Petrol pump—SN College Gate	0.56	1.06
11	SN College Gate—Kappalandimukku Junction	1.41	1.91
12	Kappalandimukku Junction—Vendermukku Junction	3.38	4.58
13	Vendermukku Junction—Pallimukku Junction (Koonambaikkulam road)	3.38	4.58
14	Pallimukku Junction (Koonambaikkulam road)—Pallimukku Junction (Eravipuram road)	3.83	5.03
15	Pallimukku Junction (Eravipuram road)—Mevaram Junction	4.03	5.23

ratio value of more than three, three sections would have the value in the range one to two (Table 3). The V/C ratio influences the speed and therefore the travel time of the vehicles.

6.3 Vehicle Operating Cost (VOC)

The vehicle operating cost (VOC) is the cost which depends on factors like vehicle usage, fuel usage, maintenance, mileage, etc. The factors affecting the VOC are vehicle type, vehicle speed, speed changes, gradient, curvature and road surface characteristics. The calculated VOC includes distance-related and time-related cost of an operating vehicle. The VOC for the study stretches has been determined using IRC SP 30 2009 recommendations and tabulated for 'with' and 'without bypass' situations. It is found that there would be VOC savings to the extent of 9% of VOC per day with the commissioning of bypass (Table 4). In monetary terms, it works out to Rs. 6.23 lakhs per day and Rs. 23 crores per year.

Table 4 Vehicle operating cost for NH 66

VOC/day (Lakhs)	
With bypass	63.41 lakhs
Without bypass	69.64 lakhs
Savings (%)	9

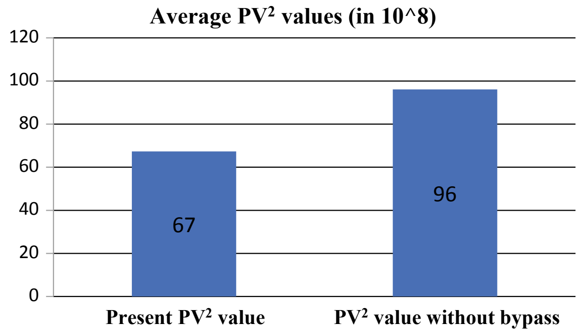


Fig. 3 Average PV^2 values with and without bypass operation

6.4 Pedestrian Safety on NH 66

Another major impact of the bypass is the substantial reduction in pedestrian vehicle conflicts witnessed in the city roads, namely NH 66 which augurs well for pedestrian safety. **PV^2** is an indicator of pedestrian vehicles conflicts where *P* represents peak hour pedestrian crossing at a location and V is the traffic volume during the peak hour. Reduction in PV^2 value is caused due to reduction in number of vehicles passing through the city roads. It is observed that quantum of large commercial vehicles inside the CBD area has been reduced drastically, especially during peak hours with only 14% of them entering the CBD area.

Overall, it is estimated that there is a 43% reduction in pedestrian vehicle conflicts with the introduction of bypass (Fig. 3) which will certainly enhance the safety of pedestrians as well as two wheeler users.

6.5 Vehicle Crashes in NH 66

Commissioning of bypass has brought forth an alarming rise in number of accidents in the city roads. For the purpose of assessing the intensity of accidents, vehicle crash data for 13.3 km of NH 66 was compared with crash data for three km control section from Kottiyam to Mevaram. It is observed that the vehicle crash rate in NH 66 was 36 crashes/km against 16 crashes/km in control section. The fatality rate in NH 66 was 3.68 fatalities per km against 1.0 fatality in control section.

However, in the commissioned bypass, an accident rate of 12 crashes per km has been reported with fatality rate of 1.2 per km.

It could be inferred that increase in travel speed along with pedestrian and pedestrian activities caused by the urbanization of city roads (NH 66) have resulted in increased accident level in city roads.

7 Recommended Proposals for Safety Issues on Bypass Road

The following are the recommendations from road safety audit conducted on the Bypass Road:

(a) At many locations, side roads meet the main road from a lower level at a gradient. Vehicles approaching from side road are not fully visible due to this gradient difference causing hazardous situation.
(b) Some of the side roads meeting the NH Bypass at an angle less than 90° creates accident proneness.
(c) Improved design of Kavanad Bypass junction is essential to reduce the vehicle conflict at intersection area.
(d) Zebra crossings and grade-spirited pedestrian facilities like subways, foot over-bridge will be needed at locations of high pedestrian crossing volume.
(e) Crash barriers need to be erected at locations where the level difference is more than 0.6 m between the road and adjoining land to avoid hazardous situations. Concrete barriers are to be replaced along the main road as they themselves are hazardous.
(f) Improper stop sign installation has been identified. STOP traffic sign placed in conjunction with traffic signal should be avoided.
(g) The curves should be marked with proper chevron markings as per IRC 67 specifications.
(h) Medians, dividers and crash barriers should be provided with hazard markers in order to provide safe night time driving.
(i) Wayside amenities for heavy vehicle drivers should be provided as the improper parking of those vehicles may interrupt the smooth flow of traffic.
(j) Bus stops and auto/ taxi stands right at the vicinity of the junction hinder visibility and smooth flow of traffic and need to be relocated to suitable locations away from the junction area.

8 Conclusions

The paper has dwelt in length the impacts of newly constructed bypass on the traffic scenario of Kollam. The single most impact is the traffic congestion which is reflected in volume capacity ratio, travel speed, transit time, vehicle operating cost, crash rate and pedestrian vehicle conflicts. These were assessed through a set of indicators developed from combination of observed traffic data, past traffic data and secondary data on crashes. Guidelines of Indian Road Congress have been extensively utilized to quantify the indicators to the extent possible.

A cursory look at the existing situation analysis and quantified impacts of newly opened NH 66 Bypass from Mevaram to Kavanad for bypassing the CBD area of the Kollam city have revealed that construction of bypass possesses several long-term benefits. The time and cost savings of the road users along with enhanced safety are the supreme benefits of investments on bypass roads. The study proved that bypassing the through traffic could be a successful solution for reducing traffic congestion in CBD areas of small as well as large cities. The salient findings derived from the study are as follows:

1. The bypass has enhanced the travel speed by 40% and reduced the travel time by 28% reflecting on the reduced traffic congestion.
2. The average value of Volume-Capacity (V/C) ratio of the homogeneous sections has come down from 3.49 to 2.51, as the result of reduced traffic congestion in city roads.
3. Vehicle operating cost (VOC) for different categories for the vehicles has been reduced by 9%.
4. Pedestrian vehicle conflicts have lessened to the extent of about 43% with lesser number of vehicles on city roads resulting in better safety of pedestrians and all road users
5. The road crashes in city roads have, however, increased by about 55% which can be attributed to high travel speed on the roads. Traffic regulatory and enforcement measures needed to be higher priority for arresting the accident rate.
6. Newly constructed bypass also has witnessed higher accident rate when compared to other section of NH 66. Road safety audit conducted on the NH 66 Bypass has assessed the causative factors for high crash rates and mitigation measures have been proposed to ensure safe travel through the bypass road.

References

1. Wafa E (2006) The influence of a bypass road on urban development and safety. Faculty of Architecture and Town Planning, Technion, Haifa, 32000, Israel
2. IRC SP 30 (2009) Manual for economic evaluation of highway projects in India
3. IRC SP 88 (2010) Manual on road safety audit

Development of Speed Management Measures at Uncontrolled Intersection; Case Study of Hyderabad, India

Adarsh Raj Srivastava, Agnivesh Pani, Bandhan Bandhu Majumdar, and Prasanta Kumar Sahu

1 Introduction

Traffic safety is a major concern all over the world, India being no different from the rest. Speed is a principal concern in traffic safety that relates to both the driver and the road. Thus by controlling speed, safety can be increased, leading to a decrease in speed-related accidents that are a major cause of death and injuries worldwide. With the increase in population, minimum income, standard of living, and ease of buying personal vehicles, vehicle count on the road is increasing exponentially every year. The road infrastructure in most of the developing countries has not been developed at the same pace to match this surging vehicle count, the effect of which can be seen with the reported number of road accidents (both when fatal and if severe injuries sustained by user) in most of the countries. After, the main reasons due to the accident were basically dependent on the intersection and driver characteristics like lack of speed control measures on that intersection, the vehicle traveling at high speed, not following traffic rules, driver not being familiar with the road, and intersection.

A closer look at the accidents statistics in India reveals that in 2017, a total of 464,910 road accidents were reported in the country that claimed 147,913 lives and caused injuries to 470,975 persons, which translates to 405 deaths and 1290 injuries each day from a total of 1274 accidents. These 2017 figures though being a marginal improvement over 2016 data (− 0.9% decline in fatal accidents) are still shocking [1]. Also, for the second year straight, the number of accidents decreased marginally over the previous years. In Hyderabad, Telangana, India, where the study was conducted, the total number of accidents reported each year (2015–2018) has seen an upward trend. Still, the number of deaths reported has decreased over the last few years from 380 in 2015 to 303 in 2018, out of which major accidents were sustained by people

A. R. Srivastava · A. Pani · B. B. Majumdar (✉) · P. K. Sahu
Department of Civil Engineering, BITS Pilani, Hyderabad, Telangana 500078, India
e-mail: majumdar@hyderabad.bits-pilani.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_21

of age group 20–40 years. As per the data available from the Hyderabad Traffic police department, approximately, 35% of the total accidents that took place were at an unsignalized intersection [2]. These data tell us about the importance of road safety measures needed to reduce the total number of accidents all over the country. In order to achieve this target, the speed trends should be studied, and based on those results, the necessary management technique should be chosen to reduce the speed at unsignalized intersections.

In order to understand the existing problems and associated studies taken in the past to tackle the said problems both in a national and international context, a brief review of literature has been taken up. In their research, Wakabayashi and Matsumoto [3] calculated travel time reliability indexes based on the cumulative distribution function outcome from the model developed for estimating travel time variation. These results were then compared, and the result reported was that the indexes established behave differently for the same route, which means understanding an index's formulation and characteristics is important for selecting the appropriate index according to both the purpose of use and the characteristics of the study route. Patil and Pawar [4], in their research, collected traffic data at three uncontrolled intersections, one each is from the city center (Type I), suburb (Type II), and outskirt (Type III). Based on this data, traffic parameters, such as traffic composition, speed variations, lane distribution, trajectories, conflict points, and pedestrian movements, were analyzed. On analysis, several conclusions were made. They found out that all the vehicle classes prefer the inner lane, except auto-rickshaws. The speed on the inner lane is higher than in outer lane vehicles as the latter is affected by the roadside friction.

Chodur and Ostrowski [5], in their research, designed and developed the models of traffic performance at unsignalized intersections for the Department of Highway and Traffic Engineering of the Cracow University of Technology, based on complex empirical and simulation studies conducted over a dozen years. Using the results obtained from empirical research models, mathematical formalization of the traffic processes at intersections, determining the parameter estimators of these processes, and the verification and calibration of theoretical models were conducted. Ghamdi [6], in his study on spot speed data in Riyadh, spot speed data were collected from different sites and analyzed. It was found that the prevailing operating speed of vehicles at all study sites exceeds the posted speed limit and the design speed. Based on statistical analysis, speed variances were homogenous on freeways but not on arterials.

2 Objectives of the Study

Unsignalized intersection being a grave problem in India. Not many studies have been conducted to study the traffic characteristics. The research gap evident in the literature can be summed as below:

- Studies on effective speed management in Indian scenarios are limited.
- No specific study has been taken up to identify the speed limits
- There is a need to take up temporal changes variation while deciding the speed limits and management system.

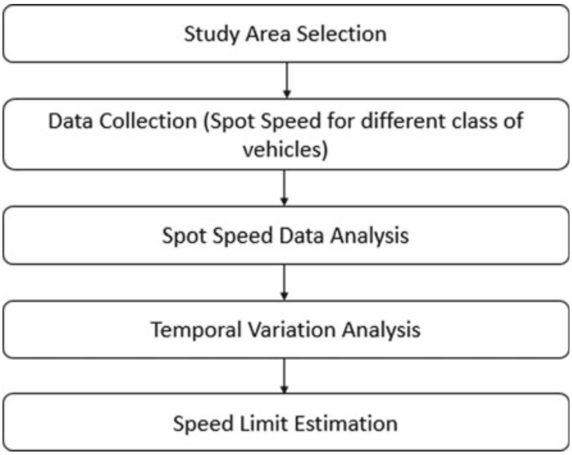
To overcome these gaps, a study at an unsignalized intersection was taken up with the following objectives:

- To study the speed characteristics of an unsignalized intersection through 15th, 50th, 85th, and 98th percentile speed on different approaches on a typical intersection.
- To study the temporal distribution of the speed across various periods for different classes of vehicles.
- To study the significance of different sampling intervals for the collection of spot speed data.
- To recommend appropriate speed limits at different approaches based on a scientific methodology.

3 Methodology

The entire study is divided into three stages. In the first stage, the study area was searched and selected for the study. In the second stage, spot speed data of approach speed toward the intersection of three different classes of vehicles at each leg of a three-legged unsignalized intersection was collected. In the third stage, the analysis of all the collected data was done and concluded (Ref. Fig. 1).

Fig. 1 Methodology



3.1 Study Area Selection

To study the speed characteristics of an unsignalized intersection, an existing intersection was located at road SH1 (Hyderabad—Shameerpet—Karimnagar—Ramagundam) and a road connecting Shameerpet—Keesara from SH1. It is a three-legged intersection with a high travel speed road (State Highway—SH) and a sub-road connecting Keesara (Major District Road—MDR). The intersection is designed as a priority-controlled intersection where the SH1 is treated as a major road and Shameerpet—Keesara road is treated as a minor road. There is no observed delay occurring on the major road. Vehicles on the minor road are controlled by the "GIVE WAY" or "STOP" sign. The coordinates of the intersection are 17.34′ 25.45″ N, 78.33′ 58.00″ E.

3.2 Data Collection

The spot speed data for different classes of vehicles were collected using a radar speed gun (also known as a speed gun), a device used to measure the speed of moving objects. A radar speed gun is a Doppler radar unit that may be hand-held, vehicle-mounted, or static. It measures the speed of the objects at which it is pointed by detecting a change in frequency of the returned radar signal caused by the Doppler effect, whereby the frequency of the returned signal is increased in proportion to the object's speed of approach if the object is approaching, and lowered if the object is receding. The radar gun was positioned at three points depicted by yellow dots in Fig. 2, and all the other details are mentioned in Table 1. The vehicle location is

Fig. 2 Study area. *Courtesy* Google Earth Pro

Table 1 Speed gun location point

S. No.	Location	Road classification	Coordinates	Traffic	
				Origin	Destination
1	1	MDR	17° 34′ 24.27" N	Keesara	Shameerpet
			78° 33′ 58.03" E		
2	2	SH 1	17°34′24.97"N	Secunderabad	Shameerpet
			78° 33′ 59.01" E		
3	3	SH 1	17° 34′ 25.77" N	Shameerpet	Secunderabad
			78° 33′ 57.43" E		

denoted by a red dot that is approximately 100 m away from the radar gun location. The approach speed of three different classes of vehicles (Two-wheelers, passenger cars, light commercial vehicles) was noted while moving toward the junction for 4 weeks on 3 days, namely Monday, Tuesday, and Wednesday from 2:30 pm to 3:30 pm study the traffic behavior for that intersection. The data were collected in different sampling intervals on different days to check for the validity of the data collected based on the methodology mentioned by Sarkar et al. [7].

3.3 Spot Speed Data Presentation

Frequency Distribution Table A frequency distribution table is created using the raw spot speed data when categorized into different speed intervals of uniform size (5 kmph interval/10 kmph interval). The frequency of each vehicle is mentioned in a particular speed interval (Ref. Table 2) [8].

Frequency Distribution Curve Using the frequency distribution table, the percentage frequency of each speed interval speed is plotted against the central value of the speed interval (Ref. Fig. 3).

Cumulative frequency distribution curve For speed interval, the cumulative percentage frequency is calculated and then plotted concerning the central speed of each interval. The speed vs. the percentage cumulative frequency graph shows the percentage of vehicles traveling at or below that particular speed using which the different percentile speeds could be found out (Ref. Fig. 4).

3.4 Spot Speed Data Distribution and Temporal Analysis

The spot speed data collected for all the three locations on different days in different weeks taken for a fixed proportion was categorized for different classes of vehicles

Table 2 Cumulative frequency and % cumulative frequency table for location 1 with sampling 1 in 1

Speed of vehicles			No. of vehicles frequency			Cumulative frequency			% cumulative frequency		
Min. speed	Max. speed	Avg. speed	Two-wheeler	Car	LCV	Two-wheeler	Car	LCV	Two-wheeler	Car	LCV
5	10	7.5	1	0	0	1	0	0	1.62	0	0
10	15	12.5	0	1	1	1	1	1	1.62	2.44	3.85
15	20	17.5	0	0	2	1	1	3	1.62	2.44	11.54
20	25	22.5	4	1	1	5	2	4	8.07	4.88	15.39
25	30	27.5	6	2	4	11	4	8	17.75	9.76	30.77
30	35	32.5	13	2	4	24	6	12	38.71	14.64	46.16
35	40	37.5	8	3	5	32	9	17	51.62	21.96	65.39
40	45	42.5	13	5	6	45	14	23	72.59	34.15	88.47
45	50	47.5	9	9	3	54	23	26	87.1	56.1	100
50	55	52.5	4	2	0	58	25	26	93.55	60.98	100
55	60	57.5	3	5	0	61	30	26	98.39	73.18	100
60	65	62.5	1	6	0	62	36	26	100	87.81	100
65	70	67.5	0	5	0	62	41	26	100	100	100
70	75	72.5	0	0	0	62	41	26	100	100	100
75	80	77.5	0	0	0	62	41	26	100	100	100

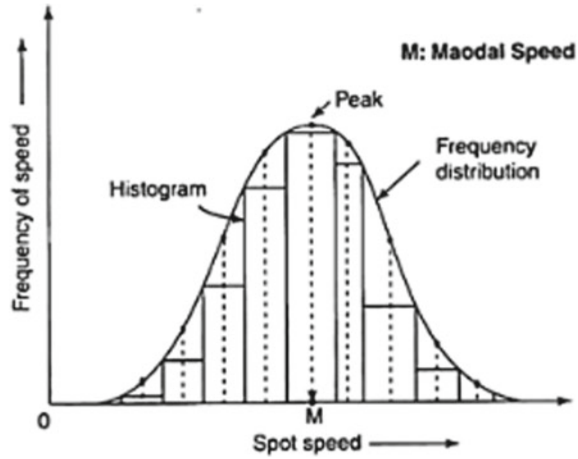


Fig. 3 Frequency distribution curve [9]

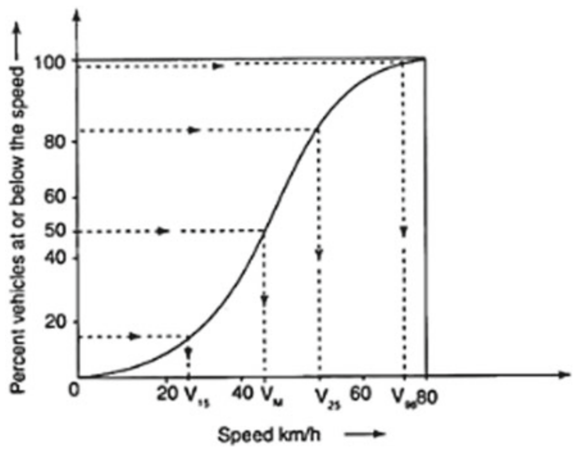


Fig. 4 Cumulative percentage frequency distribution curve [9]

according to the speed range. The cumulative frequency table was calculated using the 15th, 50th, 85th, and 98th percentile calculated and tabulated in Table 3.

Measure of Central Tendency

Mean The arithmetic mean (average) is the most frequently used speed statistics. It is calculated by.

$$v_t = \frac{\sum f_i v_i}{n} \quad (1)$$

where v_t is the mean or average speed, f_i is the frequency of vehicles having individual speed as v_i. Besides this, space mean speed (if data are obtained over a section

Table 3 Speed data for different class of vehicles

S. No.	Location	Car				Two-wheeler				LCV			
		Percentile speeds (kmph)											
		15th	50th	85th	98th	15th	50th	85th	98th	15th	50th	85th	98th
1	1	26.1	36.9	46.8	57.1	32.8	46.2	61.6	66.7	22	33.5	41.8	46.7
2	1	29.2	38.5	47.4	61.3	34.1	48.1	58.2	67	24.5	32.5	42.5	59.1
3	1	24.3	32.8	44	58.4	30.4	47.2	57.3	65.4	25.9	32.3	42.3	49.6
4	1	23	31.2	42.6	57.4	30.5	45.3	55.8	62.5	23.7	31	44.7	51.5
5	2	28	39.2	50	63.1	24	33.8	45.2	54.4	21.9	29.3	37	41.8
6	2	28	39.4	50	56.3	22	35.8	46	61.8	24.4	32.5	39.8	46.1
7	2	26.9	38.3	51.4	59.7	23.2	34.7	46.6	62.2	22.6	29.4	37.5	44.8
8	2	29.5	38	49.8	58	26.5	36.9	48.4	62.6	22.4	29.2	36.4	46.2
9	3	34.4	46.2	55.5	62.5	29.5	37.3	51.7	57.5	29.7	35.3	41	51
10	3	32.5	42.7	57.3	66.2	28.9	37	48.9	65.2	26.8	32.7	37.5	44
11	3	39.6	49.5	57.5	72.5	28.4	39.5	48.4	60.4	24.6	33.5	41.7	54
12	3	38.8	48.4	56.3	67.5	32.1	41.8	53.2	62.5	24.3	33.1	42.5	54.1

of road instantaneously, resulting in speed distribution in sce and mean) and time mean speed (if data are collected from one stationary point as in the case of spot speed study) are also used for speed statistics [8].

Median speed is defined as the speed at the peak of the frequency distribution curve; it divides the distribution into two equal half. It is not affected by the extreme values of the distribution and is just a positional value. That is median speed is 50th percentile speed (Ref. Table 3).

Measure of Dispersion

Standard Deviation Standard deviation is one of the most common statistical measures of dispersion in distributing data from the mean value [8]. The standard deviation is calculated by:

$$\sigma_s = \sqrt{\frac{\sum f_i (v_i - v_t)^2}{n - 1}} \tag{2}$$

Percentile speeds Percentile speeds are calculated using the percentage cumulative frequency curve as mentioned earlier. Typically, 15th, 50th, 85th, and 98th percentile speeds are calculated for speed characteristics studies.

ANOVA is the analysis of variance (ANOVA) is a statistical technique that is used to check if the means of two or more groups are significantly different from each other. ANOVA checks the impact of one or more factors by comparing the means of different samples.

4 Result and Analysis

The results analyzed from the percentile speed calculation using the raw spot speed data for different classes of vehicles are used to calculate the speed limit for each location (Ref. Table 3). For location 1 (MDR), location 2 (SH1 from city to outskirts), and location 3 (SH1 from outskirts to city), for the posted speed limit, 85th percentile speed is used. The average 85th percentile speed for all three classes of vehicles for the four weeks is calculated to be 50 kmph for all the three locations. Based on the analysis done for spot speed data, the different percentile speeds for different classes of vehicles at each leg of intersection (Location 1, 2, and 3) when different sampling intervals were followed for spot speed data collection are tabulated in Table 3. The sampling intervals that were followed for different locations and days are mentioned in Table 4.

Based on the percentile speeds obtained from the cumulative frequency curve for different sampling sizes, ANOVA was performed for different class of vehicles' percentile speed at different sites having different sampling sizes. For different sampling intervals (1 in 1, 1 in 3, 1 in 5, and 1 in 10), the mean difference and significance (values within bracket) for 15th, 50th, 85th, and 98th percentile speeds can be observed in Table 5. Based on the ANOVA results, the sampling interval was not significant at a 5% significance level while calculating the 15th, 50th, and 85th percentile speed of different classes of vehicles. Also, it was observed that the maximum mean difference among the values of different sampling intervals was observed to be 2 kmph.

The percentile data calculated from spot speed analysis are used for plotting the speed trends for all different locations throughout the time frame of the study showing the temporal variation for the different class of vehicle when the data was collected using the different sampling interval (Ref. Figs. 5, 6 and 7) (Ref. Table 4). Based

Table 4 Volume and sampling data for different days and location

S. No.	Location	Volume/h	Sampling	Day
1	1	248	1 in 1	Monday
2	1	265	1 in 3	Monday
3	1	232	1 in 5	Monday
4	1	241	1 in 10	Monday
5	2	344	1 in 1	Tuesday
6	2	357	1 in 3	Tuesday
7	2	372	1 in 5	Tuesday
8	2	326	1 in 10	Tuesday
9	3	334	1 in 1	Wednesday
10	3	318	1 in 3	Wednesday
11	3	352	1 in 5	Wednesday
12	3	346	1 in 10	Wednesday

Table 5 Speed data for different class of vehicles

Sampling (i)	Sampling (j)	Car				Two wheeler				LCV			
		Percentile speeds (kmph)											
		15th	50th	85th	98th	15th	50th	85th	98th	15th	50th	85th	98th
1 in 1	1 in 3	− 0.4	0.56667	− 0.8	− 0.367	0.4333	− 1.2	1.800	− 5.133	− 0.7	0.133	0	− 3.233
		(1)	**(1)**	**(0.998)**	**(1)**	**(0.999)**	**(0.994)**	**(0.984)**	(0.38)	**(0.986)**	**(1)**	**(1)**	(0.892)
	1 in 5	− 0.767	0.567	− 0.2	− 2.633	1.433	− 1.367	2.067	− 3.133	0.167	0.967	− 0.567	− 2.967
		(0.999)	**(1)**	**(1)**	(0.938)	**(0.978)**	**(0.992)**	**(0.976)**	(0.732)	**(1)**	(0.941)	**(0.996)**	(0.913)
	1 in 10	− 0.933	1.567	1.2	− 0.0667	− 0.9333	− 2.2333	0.36667	− 3	1.06667	1.6	− 1.2667	− 4.1
		(0.998)	**(0.991)**	**(0.994)**	**(1)**	**(0.994)**	**(0.967)**	**(1)**	(0.755)	**(0.953)**	(0.79)	**(0.956)**	(0.807)
1 in 3	1 in 5	− 0.367	0	0.6	− 2.267	1	− 1.6667	0.2667	2	0.86667	0.8333	− 0.567	0.26667
		(1)	**(1)**	**(0.999)**	**(0.959)**	**(0.992)**	**(1)**	**(1)**	(0.907)	**(0.973)**	**(0.96)**	**(0.996)**	**(1)**
	1 in 10	− 0.533	1	2	0.3	− 1.367	− 1.0333	− 1.4333	2.133	1.7667	1.46667	− 1.267	− 0.8667
		(1)	**(0.998)**	**(0.974)**	**(1)**	**(0.98)**	**(0.996)**	**(0.992)**	(0.891)	(0.827)	(0.828)	**(0.956)**	**(0.997)**
1 in 5	1 in 10	0.167	1	1.4	2.567	− 2.367	− 0.867	− 1.7	0.13333	0.9	0.6333	− 0.7	− 1.1333
		(1)	**(0.998)**	**(0.991)**	(0.943)	(0.911)	**(0.998)**	**(0.986)**	**(1)**	**(0.971)**	**(0.982)**	**(0.992)**	**(0.994)**

The bold face values are correspond to the P value of two way ANOVA test which are higher than the selected significance level of 95%, therefore these values are not statistically significant

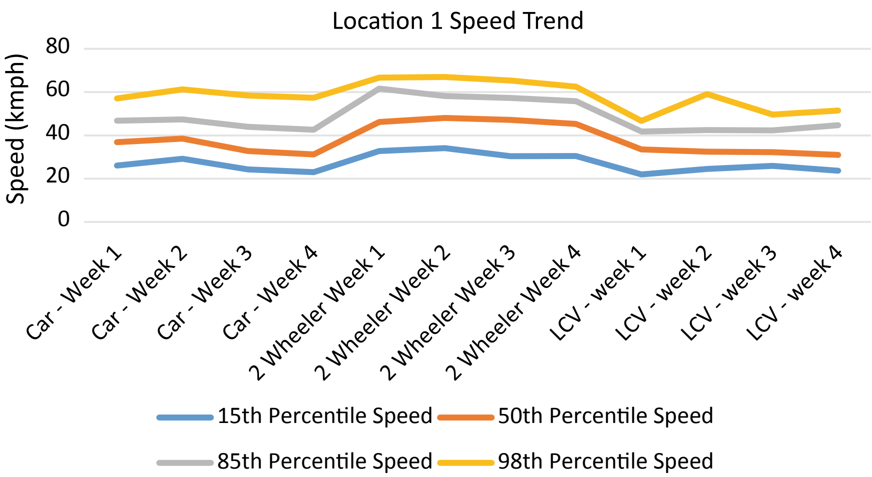


Fig. 5 Location 1 weekly speed trend for different classes of vehicles

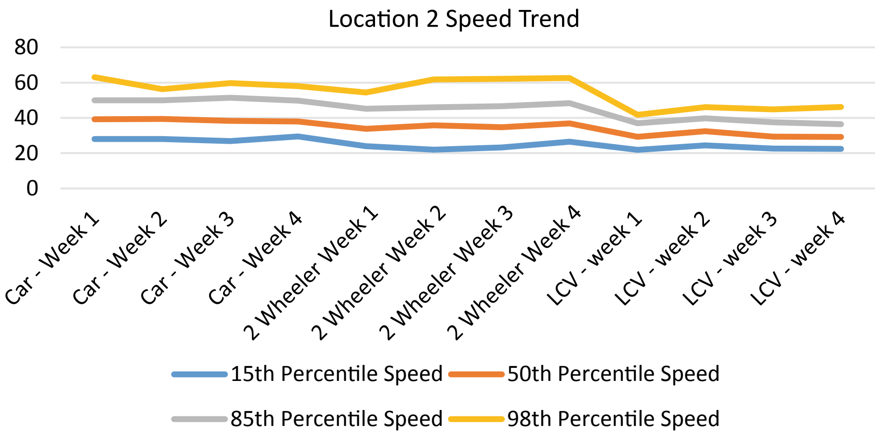


Fig. 6 Location 2 weekly speed trend for different class of vehicles

on the speed trend observed, it was found out that the 15th, 50th, 85th, and 98th percentile speed for location 1 (MDR connecting to SH) showed a constant trend from week 1 to week 4 for cars and two-wheelers, whereas a slight variation was noted in 98th percentile speed of LCV in week 2 (Ref. Fig. 5). Also, the different percentile speeds for cars were maximum among the other two classes of vehicles. For location 2 (on SH from city to outskirts), the speed trend was observed to be constant for 15th, 50th, 85th, and 98th percentile speeds for all the three selected classes of vehicles throughout the four week time frame. The speeds for cars and two-wheelers showed similar trends that can be observed from the speed trend graph (Ref. Fig. 6), whereas for LCV the speeds were lower than the other two classes of vehicles. This

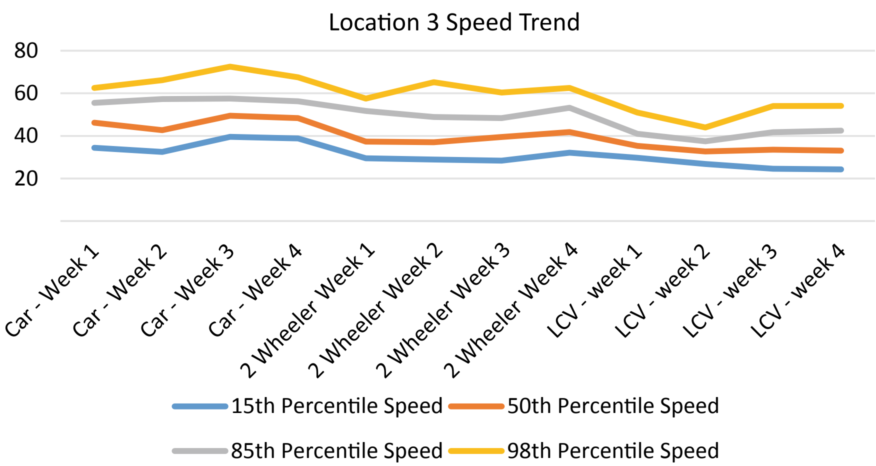


Fig. 7 Location 3 weekly speed trend for different class of vehicles

variation was observed possibly due to overloading and the low engine capacity of LCV. For location 3 (on SH from outskirts to city), the speed trends had variations in 15th, 50th, 85th, and 98th percentile speeds for the three selected classes of vehicles throughout the four week time frame. The speeds for cars showed relatively constant trends, whereas, for two-wheelers and LCV, the trend observed was varying (Ref. Fig. 7). The 85th percentile speeds of cars and two-wheelers were observed to be similar. In contrast, for LCV, the speeds were lower than the other two classes of vehicles, possibly due to overloading and low engine capacity.

5 Conclusion

Based on the results obtained from the study and analysis done, the following conclusions can be made.

- The results from ANOVA suggest that there is no significant deviation in results whether we take a sampling interval of 1 in 1 or 1 in 10; hence, sampling interval does not affect speed calculation. Thus, a sampling interval of 1 in 10 can be adopted for spot speed studies.
- All the vehicles were traveling within the specified speed limit for SH, but for MDR, it was observed that no speed limit was posted. Based on the analysis, a speed limit of 50 kmph can be proposed for MDR for all classes of vehicles.
- Not much variation was observed in the volume of traffic flowing through the unsignalized intersection during the four week time frame.

References

1. Government of India (2018) Road Accidents in India-2017. Ministry of Road Transport and Highway, New Delhi, India
2. Hyderabad Traffic Police (2019) Accident analysis. Retrieved from https://www.htp.gov.in/AccidentAnalysis.htm. Accessed on 2019/05/05
3. Wakabayashi H, Matsumoto Y (2012) Comparative study on travel time reliability indexes for highway users and operators. J Adv Transp 46(4):318–339
4. Patil GR, Pawar DS (2016) Microscopic analysis of traffic behavior at unsignalized intersections in developing world. Transp Lett 8(3):158–166
5. Pulugurtha IGS, Ghosh I, Biswas S (2019) Advances in transportation engineering. Springer, Singapore
6. Al-Ghamdi AS (1998) Spot speed analysis on urban roads in Riyadh. Transp Res Rec 1635(1):162–170
7. Sarkar PK, Maitri V, Joshi GJ (2017) Transportation planning: principles, practices and policies. PHI Learning Pvt. Ltd.
8. Mathew TV, Rao KK (2006) Fundamental relations of traffic flow. Introduction Transp Eng 1:1–8
9. Kadiyali LR (2013) Traffic engineering and transport planning. Khanna publishers

Objective and Subjective Approaches Towards Analysis of Level of Service of Sidewalks

G. R. Bivina and Manoranjan Parida

1 Introduction

In recent times, researchers have started identifying the importance of psychological factors such as perceptions, mental maps in travel behavior analysis. Most of the travel research studies highlighted objective assessment of environmental measures. This may be due to the fact that it is difficult to capture peoples' perceptions of environment for large sample size in a meaningful and consistent manner. Urban planners and designers have long held that people walk, internalize and interpret their surrounding in complex ways. Appleyard et al. [1] attempted to devise the most important environment factors for pedestrians [1]. Likewise, many studies have indicated environment is to people, but none of them have given a view that how people perceive the environment.

Pedestrian Level of Service is an evaluation of service quality of pedestrian infrastructure including aspects such as contextual situation and facilities themselves. Hence, in order to measure PLOS, more complex approaches are needed than LOS for vehicles. Pedestrians' perceptions of environmental characteristics called subjective measures of environment are need to be captured. The qualitative factors of sidewalks such as safety, security, comfort are equally important along with quantitative factors for the measurement of PLOS.

Those studies that examined relationship between built environment and walking have factors measured objectively, subjectively, or in combination. The subjective measurements are obtained from questionnaire surveys [2], whereas, objective measurements are obtained from field data collection [3]. Most of the past studies

G. R. Bivina (✉)
Department of Civil Engineering, Maulana Azad National Institute of Technology, Bhopal, India
e-mail: bivinagr@manit.ac.in

M. Parida
Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_22

found that a variety of built environment factors measured both objectively and subjectively are related to walking [4–6]. However, study conducted by Lin and Moudon [7] found that objective measures have strong associations with walking than subjective measures. Measuring environmental factors objectively is found to give better prediction of behavior than subjective measurements [7]. However, perceptions are said to affect behavior in more direct ways than reality [8]. High perceptions of urban environment tend to high levels of walking [9]. Handy et al. [8] found that consistency of results will be poor for assessment of perceived and objective measures [8]. Objective measures were argued to have reliability allowing replication by capturing same variables. However, the physical measures do not capture a complete comprehension of walking experience in the environment individually. Especially, objective measures do not comprehend walking behavior, and pedestrian's perception may highly influence it than objective measures of environment. Several studies in transportation, planning and urban designs have measured built environment factors such as sidewalk continuity, ease of street crossing [10] and block length [11] objectively using field surveys. But field survey are highly expensive making the study limited to a small area or small part of streets that ultimately results in biasness. Geographic Information Systems (GIS) is used as an alternative to measure environment for large study areas by researchers. In contrast, most of the researches in public health field adopted perceived built environment measures for determining the impact of environmental measures on walking. Pedestrian perception on traffic was found not be related to walking [12]. However, study conducted by Huston et al. [13] found a positive relation between physical activity and perception of traffic [13]. Objective measures were collected for those factors that influence walking behavior such as traffic speed, traffic volume, sidewalk width, etc. Many studies have found the impact of objective or subjective measures on walking behavior but not on PLOS. Most of the studies have measured PLOS objectively using route assessment surveys or audits. However, there are lack of studies which identifies the contribution of objectively and subjectively measured built environment factors on PLOS. Alfonzo [14] found that objectively measured factors could define the walking environment better [14]. On the other hand, the study conducted by Ewing and Handy [15] suggested that perception of the environment determine the overall walking behavior [15]. Limited numbers of studies have incorporated objective and subjective measured built environment factors of walking to assess their impact on Pedestrian Level of Service. Therefore, it has been suggested that objective and subjective measures of environment need to be used hand in hand to improve the understanding on PLOS.

2 Study Area

Thiruvananthapuram, capital of Kerala, has been chosen as the study area. It is one of the most populous and largest cities in Kerala. The total metropolitan population of the city is 1.68 million of which 815,200 are males and 872,206 are females [6].

The road network consists of National Highway 47, State Highways and District roads. There are both intracity and inter-district bus public transport owned by the state-owned Kerala State Road Transport Corporation (KSRTC) buses. There are several low platforms AC Volvo buses along certain routes of the city. The other modes of public transportation in the city are cabs and auto-rickshaws and people mostly use cars and two-wheelers as personalized modes. There are even a number of private buses running along selected routes within the city. In the last two decades, traffic in the city road has tremendously increased due to an increase in personalized vehicles because of the improvement in financial status and living standards of people. This scenario is leading to traffic congestions and snarls on road streets as road development activities are not keeping a pace with traffic growth. With the increasing trend of pedestrian accidents in central business areas of the city, pedestrian safety has become a serious concern. In order to ensure safe and comfortable movement of pedestrians and vehicles, streamlining of pedestrian movements is necessary. There is a potential conflict between vehicular traffic and pedestrians due to the absence of sidewalks and other segregation mechanism. Ten locations (corporation zones) from different land uses were selected from the CBD area of the city. The inventory exercise was conducted for the site. In general, it was noted that sidewalk with standard measures are restricted to Central Business District Areas of Thiruvananthapuram city. These sidewalks are available along four-lane divided and two-lane divided roadways to serve various land use facilities on either side such as shopping malls, bus stops, etc. It is observed that most of these sidewalks lack continuity and facilities for persons with disability. The study is mainly focused on the locations with the availability of sidewalks where segregated or raised sidewalks are available. The study locations are displayed in Fig. 1. Sixty street segments from ten wards were assessed in this study to collect micro-scale built environment data. Total 12 km

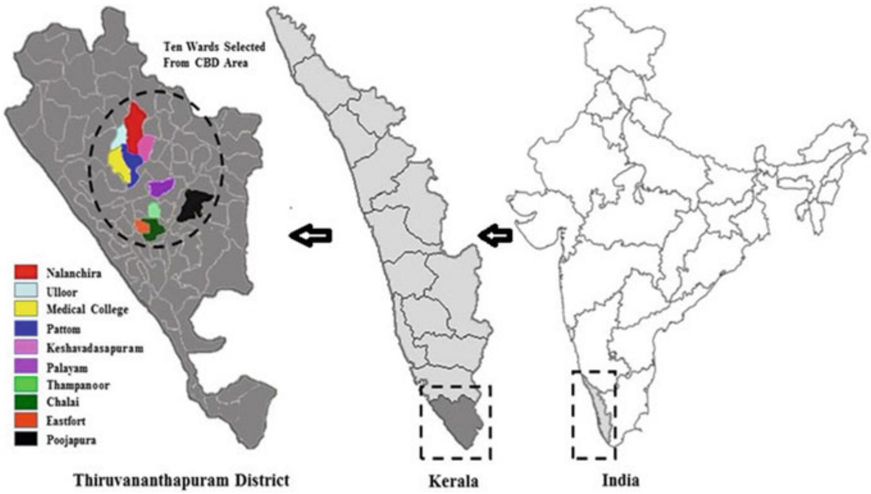


Fig. 1 Map showing Thiruvananthapuram study area locations

sidewalk street segment was selected for this study. From each zone, six sidewalk segments were selected for the survey and each sidewalk segment is of 200-m length.

3 Methodology

The data collection included primary data collection and secondary data collection. The primary data collection is in the form of questionnaire survey and walk audit survey.

3.1 Perceptual Data/Subjective Data Collection

Individual information was collected about each respondent through questionnaire surveys. Face to face surveys is the best method to understand the pedestrians' perception about the built environment affecting walking and PLOS of sidewalks. After identifying the survey method, sample selection is another important issue in the questionnaire survey, which is supposed to represent the entire population. Simple random sampling method has been selected for the survey. The questionnaire survey was conducted in the month of March and April 2017 including weekdays from morning 6'o clock to evening 6'o clock. The survey was conducted for only those pedestrians who belong to that particular ward. The respondents were requested to give their perception about various built environment factors which affect their walking on sidewalks in that zone in the form of level of satisfaction.

3.2 Selection of Factors Affecting PLOS

The selection of factors has been done as a two-stage process. Various design guidelines [16, 17] and walkability studies of Indian practices [18] as well as previous sidewalk assessment tools [19, 20] were reviewed. In addition, several pedestrian level of service models developed in other countries such as the U.S. [21, 22], Australia [23], Malaysia [24], Japan [25], and Germany [26] were also reviewed. In the second stage, a pilot study was conducted with 50 pedestrians to obtain responses regarding the priority ratings of factors affecting walk accessibility to metro stations. Open questions about the factors considered by pedestrians for using sidewalks were also asked. Cronbach's alpha value was found to be greater than 0.70 [27] for each factor indicating that the questionnaire is reliable. Importance of various factors was measured by analyzing the questionnaire responses. Based on this analysis, final factors selected for the full scale questionnaire survey is given in the following Table 1 with their descriptions.

Table 1 Factors and their description

S. No.	Factors	Description
1	Traffic volume	Quality of streets gets affected by high traffic volume
2	Traffic speed	High speed traffic makes walking environment uncomfortable and unsafe
3	Availability of crossing	Availability of crossing facilities such as zebra crossings, pedestrian foot over bridges, subways, etc. makes pedestrians feel safe to walk
4	Presence of buffer	Buffer in the form of grass, landscape can be provided between the sidewalk and the lane
5	Shaded walk	Shaded sidewalk comforts walking
6	Presence of bus shelters	Presence of bus shelters would positively enhance pedestrians' decision to walk
7	Police patrolling	Presence of police along walkways enhance pedestrians' sense of security
8	Street lighting	Presence of street lighting provides good visibility and personal security
9	CCTV cameras	CCTV surveillance sense of security of pedestrians
10	Unattended dogs	Presence of unattended dogs also discourages people to walk. They are threat to pedestrians' sense of security
11	Presence of guard rails	Guard rails are design elements of sidewalk infrastructure that prevent haphazard spilling over of pedestrians on to the right of way. Guard rails helps in pedestrian safety and avoid the chances of pedestrian vehicle conflict
12	Interesting things	Presence of interesting things in the form of graffiti, advertisement boards, etc. would attract more people to walk
13	Trees and landscape	Trees and landscaping along sidewalks helps in attracting pedestrians and also acts as a buffer between right of way and sidewalk
14	Street furniture	Availability of benches, seating areas, etc. provides comfort and convenience for people and therefore, enhances their choice of walking
15	Traffic signs and signals	Traffic signs and signals are important elements that improve pedestrian safety and guide them to their destinations
16	Sidewalk width	Sidewalk width affects pedestrians' comfort and mobility

(continued)

Table 1 (continued)

S. No.	Factors	Description
17	Sidewalk surface quality	Surface quality of sidewalk is important to ensure the mobility of pedestrians. The sidewalk texture should be firm, non-slip surface and stable
18	Obstructions	Sidewalk obstructions are in the form of telephone poles, sign boards, electric posts, utilities, etc.
19	Encroachment	Street vending has been an integral aspect of Indian cities and towns since olden days. Street vendors are inevitable on urban streets, as they provide affordable services to a majority of the urban population. It also helps pedestrians to cater their day-to-day needs. But sometimes, encroachment extends to such a level that it reduces the clear space of sidewalks for pedestrians making the sidewalks inaccessible or sometimes non-usable
20	Sidewalk continuity	Continuity is one of the main factors of sidewalk infrastructure especially for aged persons and persons with disability. Frequent up and downs of sidewalk affect pedestrians' smooth mobility
21	Sidewalk cleanliness	Cleaner sidewalks attract more people to walk. The sidewalk should be free from garbage and litter
22	Pedestrian amenities	Availability of amenities such as toilets, drinking water provisions, etc. enhances the attractiveness of pedestrian environment bringing more people into walking
23	Facilities for physically disabled	A sidewalk should provide equitable access to all sections of society. It should provide accessibility for people with disability through tactile pavements, ramps
24	Raised/segregated sidewalk	A sidewalk should be either raised or segregated from the right of way where the vehicles ply

3.3 Design of Questionnaire Survey

The first step of designing the questionnaire was to decide the information that has to be collected accordingly with the prescribed objectives. The questions were made as comprehensive as possible in the listing. The phrasing of questions was done in such a manner that questions appear close to the point. As the next step, the response format was developed and this study adopted Likert scale responses that give a rating in the scale of 1–5. Afterwards the questions were arranged in sequences that would help in bringing logic and flow to the interview. The questionnaire was finalized by giving a clear introduction to the study. Finally, pretesting and revision of the questionnaire was done by conducting a pilot study.

3.4 Pilot Study

Before conducting full-scale survey, pretesting of the questionnaire has been done through a pilot study. Pilot study was undertaken with 50 respondents from Thiruvananthapuram city. Data collected through this survey was analyzed to assess whether the data collected helped the researcher to achieve the objectives of the study aside from the testing the validity of the questionnaire. The pilot study also helped in determining the degree of complexity and total time taken to answer the questions. Cronbach's alpha value has been used to examine the reliability of the scale used in the questionnaire. Alpha value 0.7 or greater indicates that the questionnaire used is reliable. A standard questionnaire developed was used for the pilot study. An edited version of the questionnaire with relevant additions, modifications and deletions will make it suitable for use in the main survey in the field.

Most of the participants identified were working in government or private sector. Two analyses have been conducted on pilot study samples such as reliability analysis and correlation analysis. Results were positive as the reliability coefficient is very high indicating high-quality data collected from respondents and correlation analysis results indicate that PLOS being associated with the environmental attributes. Thus, the limitations and inefficiencies of the questionnaire were identified through the pilot survey. Then the questionnaire was modified by removing and replacing few factors and changing the question patterns on the basis of feedback provided by pedestrians and experts.

3.5 Final Questionnaire and Process of Data Collection

The final survey instrument consisted of 3 parts. First part gathered information about socio-demographic characteristics like age, sex, occupation, educational qualification and income. Second part collected data about trip characteristics like frequency of walk trip, walking distance, the purpose of walk trips and time taken for walk trips. Third part of the questionnaire focused on the pedestrians' perception about the sidewalks. It gathered information about the pedestrians' level of agreement/satisfaction with the given statements about the quality of each environmental attributes that affect PLOS of sidewalks. For example, statements like "*There is so much traffic along the street where I work/live which makes it difficult to walk*". Respondents were asked to state their level of agreement with sidewalk attributes on a 5 point Likert scale ranging from strongly disagrees to strongly agree (Fig. 2). The final question was about the overall level of service offered by the sidewalk which was

Fig. 2 Questionnaire survey at sidewalks

asked twice during the survey; first at the beginning of the third section of the survey and second time at the end of the questionnaire after the pedestrians were reflected on the factors describing the Level of Service.

3.6 Sampling Method Adopted

Simple random sampling method has been selected for the interview. In this method of sampling, each sample has an equal probability of being chosen. It is one of the simplest approaches to collect data from the total population. In this sampling, each respondent of the subset gets an equal opportunity of being selected as a part of the sampling process. But, there is always a possibility that the sample does not represent the population as a whole. Sample chosen need to be an unbiased representation of the population. Sampling error needs to be checked when adopting the simple random sampling method. In order to draw significant and reliable conclusions, an unbiased random sample is necessary. The sample collected in the survey has been checked for its representativeness.

$$S = (\chi^2 \mathrm{NP}(P-1))/(d^2(N-1) + \chi^2 P(P-1)) \tag{1}$$

where, S = required sample size; χ^2 = the table value of chi-square at anticipated confidence interval (3.8342); N = Population size (The number of walk mode trips i.e. 17% of the total population); P = Population proposition of crossing trip; d = the degree of accuracy as a proposition (0.05). The collected questionnaire survey sample in this study is more or equal to the required sample size (S).

Table 2 Samples collected from each ward

S. No.	Name of ward	Sample size
1	Chalai	120
2	East Fort	109
3	Palayam	134
4	Keshavadasapuram	144
5	Medical College	141
6	Pattom	109
7	Nalanchira	162
8	Ulloor	106
9	Thampanoor	113
10	Poojapura	116
Total samples collected		1254

3.7 Determination of Sample Size

Since it is not feasible to interview the whole population, sampling was adopted. It was ensured that the sample is truly representing the population. Sample size was calculated from the walk trips produced in the selected area at a confidence interval of 95 and 3% margin of error. There are three factors such as variability of a population, the degree of precision and population size that significantly affects the determination of suitable sample size. Walking accounts for 17% of total trips produced in the study area [28]. Therefore, the target population considers only walk trip percentage of whole population of the study area. The minimum number of sample size obtained is 1068 and about 1254 responses were collected for the study (Table 2). Only 1091 complete and valid responses were considered for the analysis. The following Eq. 1 is applied to estimate the sample size of pedestrians from the population of walk trips at selected wards.

3.8 Objective Data Collection

In this study, built environment attributes were collected using assessment surveys. These surveys are carried out in ten zones of the study area. Aim of the survey was to measure and investigate conditions of walking environment such as sidewalk width, surface quality, cleanliness, etc. Sixty street segments from ten wards were assessed in the study to collect data on micro-scale built environment factors (Fig. 3). From each zone, six sidewalk segments were selected for the survey and each sidewalk segment is of 200-m length. Table 3 presents the data captured for route assessment surveys.

Fig. 3 Route assessment survey at sidewalks

Table 3 Data captured using route assessment surveys

S. No.	Data item	Method	Unit
1	Sidewalk width	Measured	meters
2	Sidewalk surface quality	Estimated	% length
3	Presence of bus shelters	Count	Number
4	Continuity of sidewalks	Estimated	% length
5	Availability of crossing facilities	Count	Number
6	Presence of buffer	Estimated	% length
7	Shaded walk	Estimated	% length
8	Sidewalk encroachment	Count	Number
9	Availability of pedestrian amenities	Count	Number
10	Amenities for physically handicapped people (tactile tiles, ramps)	Estimated	% length
11	Sidewalk obstructions	Count	Number
12	Cleaner sidewalks	Estimated	% length
13	Presence of police patrolling	Visual	Yes/No
14	Unattended dogs	Visual	Yes/No
15	Presence of street light	Count	Number
16	Presence of CCTV cameras	Count	Number
17	Traffic speed	Estimated	km/h
18	Traffic volume	Count	Number
19	Presence traffic signs and signals	Count	Number
20	Availability of street furniture	Count	Number
21	Interesting things around	Visual	Yes/no
22	Trees and landscapes	Visual	Yes/no
23	Presence of guard rails	Estimated	% length
24	Raised/segregated sidewalk	Visual	Yes/no

4 Formulation of Indices

The characteristics of data sources for each of the factors are different, hence, these factors often do not line up. Thus, accumulating these data and building indices are considered to be important. Both objective and subjective indices need to be as comprehensive as possible. Hence, to form a meaningful and comprehensive comparison of these both objective and subjective measures, these indices need to be theoretically equal in construct. This agreement necessitates that even factors are better at explaining the aspect of environment; they cannot be incorporated in the index if there are no equivalent factors in the other classification (Objective/Subjective). The equivalence between the subjective and objective attribute is considered as most important factor as the value of this comparative analysis is the backbone of this study. The indices are constructed using a 5 step methodology; conducted literature review to scrutinize the factors that are most relevant for PLOS of sidewalks; then chosen measures are consolidated by retaining only those factors that could be measured both objectively and subjectively. In the third step, correlation and multi-collinearity of the explanatory factors are assessed. Relevant factors are grouped in the fourth step using Principal Component Analysis (PCA) to confirm that all groups include factors. In the final step of the index construction, chosen measures are normalized and clustered to form indices. The whole process of index construction is presented in Fig. 4.

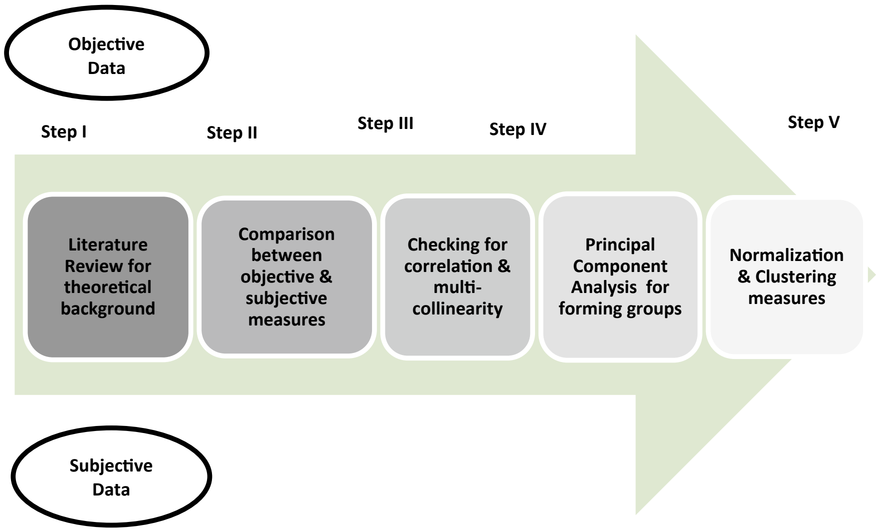


Fig. 4 Steps in index construction

4.1 Choosing Factors from Literature

As said above, construction of indices started with literature review exploring the factors used in previous researches to aggregate environmental characteristics (Step 1, Fig. 4). Most of the previous researches considered factors that are important for walking activity not specifically for PLOS. Quality of the micro-scale street walking environment is in the interest zone of planners. For example, Sarkar [22] discussed various factors of built environment that affect PLOS such as comfort, convenience, security, continuity, system coherence and attraction [22]. Indian code, IRC-103:2012 [16] presented micro-scale qualitative built environment factors to be considered for assessing quality of sidewalks, but it has not differentiated the safety and security factors such as safety from traffic flow and security from crimes. The code has considered a generalized factor 'walking environment' for the qualitative factor to be considered in assessing PLOS.

4.2 Finding Association

Once the factors that are most relevant to PLOS had been scrutinized, the chosen factors are measured from objective and subjective viewpoints and are compared for any direct association (Step 2, Fig. 4). Those factors that are not having a direct association are removed. For Instance, even though the presence of pedestrian amenities might be related to PLOS of sidewalks, objectively measuring pedestrian amenities would be difficult, therefore, it was removed. Similarly, presence of interesting things, presence of trees and landscapes and unattended dogs were also removed due to the difficulty in objectively measuring it.

4.3 Checking for Correlation

The selected PLOS factors are tested to examine the correlation between explanatory variable before conducting any statistical analysis (Step 3, Fig. 4). The correlation coefficients of some factors are greater than 0.3, implying that the multi-collinearity problem may arise in the analysis. Variation Inflation Factor (VIF) was analyzed to identify multi-collinearity between the factors. The explanatory variable with a VIF value greater than 10 has a multi-collinearity problem in the model. The VIF values for all variables are presented in Table 4. VIF is given by the following formula:

$$\mathrm{VIF}_k = \frac{1}{1 - R_k^2} \tag{2}$$

Table 4 Variance inflation factor (VIF) for all factors

Factors	VIF (subjective measures)	VIF (objective measures)
Traffic volume	1.871	3.353
Traffic speed	1.976	1.379
Presence of buffer	1.099	1.638
Shaded walk	1.449	1.740
Presence of bus shelters	1.349	1.414
Presence of police patrolling	1.348	1.971
Availability of street light	1.325	1.894
Sidewalk width	1.766	2.170
Sidewalk surface quality	1.977	2.018
Continuity of sidewalks	1.746	1.248
Sidewalk obstructions	1.410	2.225
Sidewalk encroachment	1.341	1.570
Cleaner sidewalks	1.791	1.293
Facilities for persons with disability	1.323	1.600
Pedestrian amenities	1.532	1.823
Availability of crossing facilities	1.626	1.712
CCTV	1.535	1.456
Raised sidewalks	1.845	1.365

In the formula, R_k is the deterministic coefficient of the variable k in the model; generally, if R_k is 10 or above, it could likely be an explanatory variable with multi-collinearity problem.

4.4 Statistical Confirmation of Groups

The final measures of PLOS chosen through theoretical means which are comparable in objective and subjective measurement are analyzed with Principal Component Analysis (PCA). PCA is performed using SPSS to assess whether each chosen index provides a good comprehension of all input variables and thereby process the index measures (Step IV, Fig. 4). PCA results provided groups that measured a maximum of the original variance of data. Nevertheless, PCA in this study was not used for factor reduction or justifying which measures have to be included, as PCA does not specify which measures are the best for gauging the construct. More specifically, PCA was not conducted to determine the suitability of variables. It does not disclose which factors influence PLOS, which is of uttermost regard in this research. Measures loading at 0.5 or greater than 0.5 are retained for the study. The factors such as presence of guard rails, traffic signs and signals, raised sidewalks and availability of street furniture are removed due to their poor loading.

4.5 Normalization and Aggregation of Factors to Form Indices

Variables obtained after PCA are combined and indices are constructed (Step V, Fig. 4). These variables are measured in different units and are normalized because of their different scales. The average of measures is computed for calculating each index values. The index measures are provided in Table 5.

Table 5 Index measures

Index	Objective measures
Mobility and infrastructure	Sidewalk width
	Sidewalk surface quality
	Cleaner sidewalks
	Continuity of sidewalks
	Sidewalk encroachment
	Sidewalk obstructions
	Facilities for people with disability
Comfort and convenience	Presence of bus shelters
	Shaded sidewalks
	Pedestrian amenities
Security	Presence of police patrolling
	Presence of street light
	Presence of CCTV cameras
Safety	Traffic speed
	Traffic volume
	Availability of crossing facilities
	Presence of buffer

5 Results and Discussion

5.1 Socio-Demographic Characteristics of Respondents

A total of 1254 people were interviewed from ten corporate wards of Thiruvananthapuram city, which included people from various socio-economic backgrounds. And 1091 samples are used for the analysis. Preliminary analysis of collected data was done to check whether the sample represents true population and to identify the walk characteristics of people. The samples are stratified as follows:

Stratifications based on gender and age group are illustrated in Fig. 5. Out of 1091 persons, 49% were males. The female to male ratio of Thiruvananthapuram city from Census data (2011) is 1.05 and that of collected data is 1.08, which are comparable. Stratification based on age group shows that 12% were less than 18 years old, 46% were in the age group of 18–45 years old, 33% in 45–60 years old and 9% who were greater than 60 years. Based on the profession, samples were distributed as individuals were classified into seven groups (Fig. 6), students (22%), public service (24%), private service (26%), house wife (13%), retired (6%), unemployed (3%) and others such as businesspersons, self-employed included 6%. Based on the level of income, maximum people have income between Rs. 20,000 and Rs. 30,000 (25%). Walk characteristics such as walk distance and walking time have been stratified as shown in Fig. 7. It is noted that maximum number of people walk for work (21%). About 18% people walk for exercise and 14% for relaxing or evening walk. Stratification based on walk distance shows that collected data consisted of 29% of people who walk 500–1000 m per day. About 26% people used to walk 1000–1500 m per day and 275 people walk 400–500 m per day.

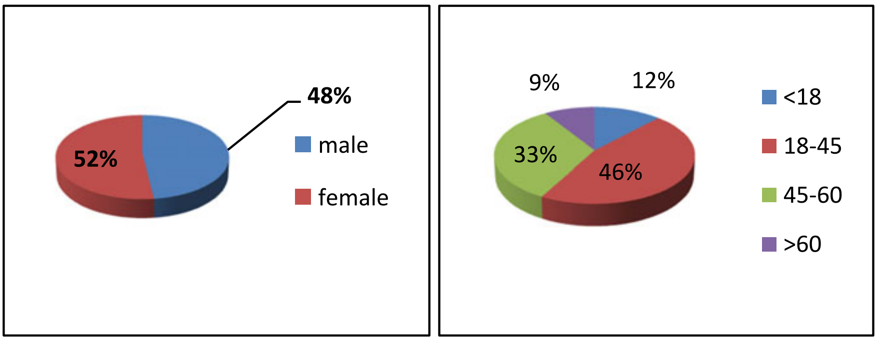


Fig. 5 Sample stratification based on gender and age group

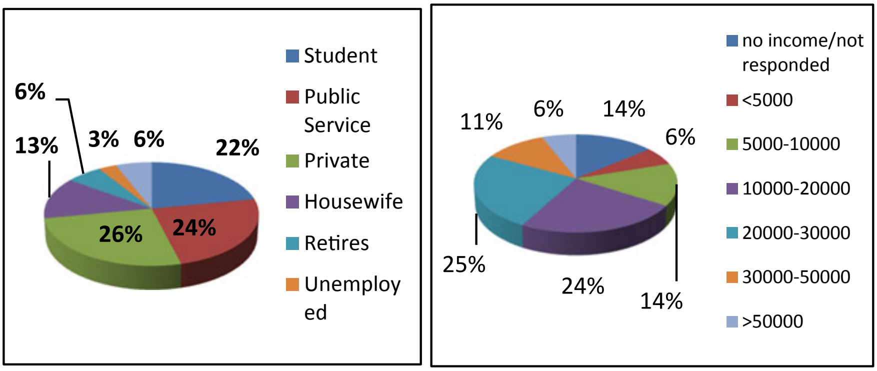


Fig. 6 Sample stratification based on profession and personal income

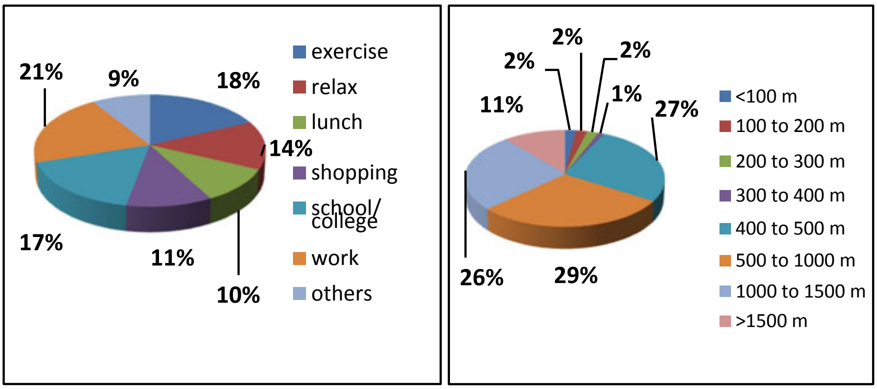


Fig. 7 Sample stratification based on purpose of walking and walking distance

5.2 *Distribution of Perceived PLOS Categories Before and After Evaluation*

The final question in the questionnaire was about the overall Level of service offered by sidewalks. This question was asked twice during the survey; first at the beginning of the third section of the survey and second time at the end of the questionnaire, after that the pedestrians reflected on the factors describing the Level of Service. It has been found that pedestrian perception towards LOS A is 5.3% before evaluation while after evaluation this value was increased to 7.6%. The category C was perceived by the higher number of the pedestrians before the evaluation (37.5%). But after evaluation, when pedestrians are introduced to the PLOS attributes, this trend has changed and highest category was LOS B with 37.5%. Among 1091 pedestrians, 17.5% (Fig. 8) and 21.2% perceived LOS D before and after evaluation respectively.

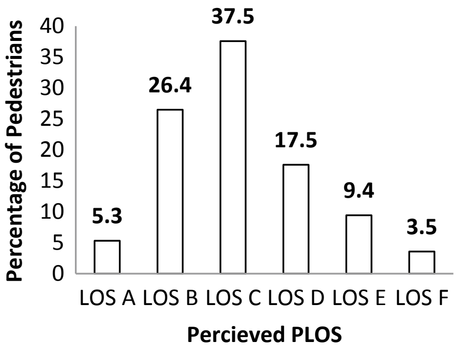


Fig. 8 Sample stratification based on purpose of walking and walking distance

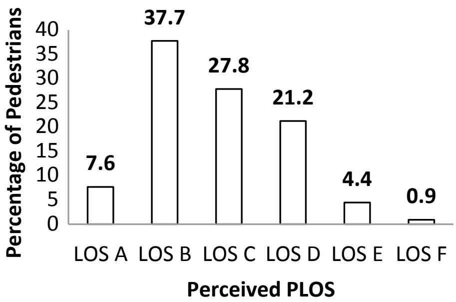


Fig. 9 Distribution of perceived PLOS categories after evaluation

In general, the perception about the level of service of sidewalks may vary with gender. Out of 1091 respondents who provided perception towards LOS, 51% (556) were identified as male pedestrians, 49% (535) were female pedestrians. Perceived LOS categories for before evaluation and after evaluation have been plotted separately for gender (Figs. 9 and 10). In case of before evaluation, the proportion of male and female pedestrians gave equal percentage of response for LOS C which is the maximum percentage (38%) obtained when compared to all other LOS categories (Fig. 9). After evaluation, LOS B received the maximum percentage of responses i.e. 37% from female pedestrians and 36% from male pedestrians. All LOS categories of ‘After Evaluation’ are greater than the ‘Before Evaluation’ suggests that PLOS is better perceived by pedestrians when they are reflected on the characteristics describing the PLOS of sidewalks.

5.3 Objective and Subjective Factors

Tables 6 and 7 summarize the descriptive analysis of objective and subjective measures of built environment affecting PLOS of sidewalks. From Tables 6 and 7, it can be observed that all the subjective measures are measured in the scale of 1

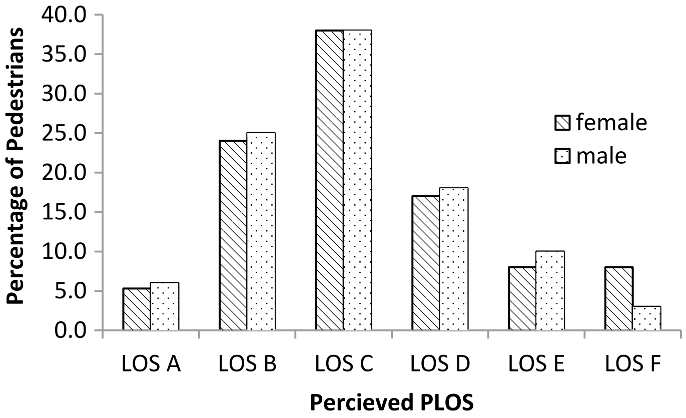


Fig. 10 Effect of pedestrian gender on perceived PLOS categories before evaluation

to 5 Likert scale using questionnaire survey and all objective measures are measured by route assessment survey respectively. Minimum, maximum, mean and standard deviation values of all subjective and objective factors are presented. Among the subjectively measured factors, traffic speed obtained maximum average value (3.61) and standard deviation value (1.07). Facilities for persons with disability received the least average value and standard deviation value of 2.03 and 1.05 respectively. The lowest standard deviation value of 0.94 is obtained for traffic volume factor among the subjective measures. Table 7 shows that minimum traffic volume obtained is 312 vehicles per hour and maximum is 1174 vehicles per hour. About 70.8% of the road length is having good surface quality. Facilities for persons with physical disability are provided only for 15.23% of corridor which is very low. About 76.2% of sidewalks stretches are cleaner and about 50.23% of sidewalks are continuous.

5.4 Association of Objectively Measured Factors with PLOS

The dependent variable (PLOS) in this model is obtained from the questionnaire survey. Since the dependent variable in this model is ordinal, and ordinal logistic regression model has been adopted for analyzing the factors influencing perceived PLOS. PLOS obtained after evaluation has been taken in the analysis. Respondents were asked to rate the overall PLOS of sidewalks they walk using a 6 point Likert scale from 1 to 6 for LOS A to LOS F respectively. The model results are presented in Table 8. Models are run for objectively measured variables (M1), subjective measured variables (M2), a combined objective, and subjective measured variables (M3).

Socio-demographic characteristics such as age, employment and service are not significant in any of the models (Table 8). Work trips and educational trips are also positively and significantly associated with PLOS of sidewalks. However, gender and

Table 6 Descriptive statistics of subjective measures

S. No.	Data item	Min	Max	Mean	Std. Dev.
1	Sidewalk width	1	5	3.36	0.97
2	Sidewalk surface quality	1	5	3.23	1.04
3	Presence of bus shelters	1	5	3.28	1.00
4	Continuity of sidewalks	1	5	3.13	1.02
5	Availability of crossing facilities	1	5	3.32	1.01
6	Presence of buffer	1	5	2.15	0.70
7	Shaded walk	1	5	3.08	1.02
8	Sidewalk encroachment	1	5	3.08	1.02
9	Availability of pedestrian amenities	1	5	3.13	1.03
10	Facilities for people with disability	1	5	2.03	1.05
11	Sidewalk obstructions	1	5	2.69	1.05
12	Cleaner sidewalks	1	5	2.99	1.03
13	Presence of police patrolling	1	5	2.92	1.19
14	Unattended dogs	1	5	3.07	1.11
15	Presence of street light	1	5	3.32	1.05
16	Presence of CCTV cameras	1	5	2.42	1.15
17	Traffic speed	1	5	3.61	1.07
18	Traffic volume	1	5	3.58	0.94
19	Presence traffic signs and signals	1	5	2.99	1.07
20	Availability of street furniture	1	5	2.80	1.07
21	Interesting things around	1	5	2.92	0.99
22	Trees and landscapes	1	5	2.79	0.98
23	Presence of guard rails	1	5	3.17	1.14
24	Raised/segregated sidewalks	1	5	3.28	0.42

walking distance are significant in many of the models. Compared to women, men had negative association in perceiving PLOS which indicates that women are more likely to perceive PLOS better than men are. Therefore, they have a high perception for the PLOS of sidewalks. These results are quite expected but when compared to the previous researches, these results are different. More women at younger ages can walk for leisure than men. But at older ages, these differences among gender for walking is small [29]. Women with children walk more for leisure activities than men [30]. This result also may be due to the fact that women population in the sample is higher than males and also most of the female participants have children with which they would probably walk more for shopping and leisure activities. Similar results are found for gender variables in the study of Rahul and Verma [30] where men had a negative effect on probability of walking. Compared with other trips, work trips showed a positive association with PLOS. For PLOS, the walk purpose variable is insignificant with a p-value greater than 0.05 in all the three models. This may be

Table 7 Descriptive statistics of subjective measures

S. No.	Data item	Min	Max	Mean	Std. Dev.
1	Sidewalk width (m)	0.80	2.80	2 0.00	0.23
2	Sidewalk surface quality (%)	32.00	80.23	70.80	2.30
3	Presence of bus shelters (number)	0.00	1.00	0.20	0.40
4	Continuity of sidewalks (%)	23.00	63.00	52.30	1.79
5	Availability of crossing facilities (number)	0.00	2.00	1.07	0.76
6	Presence of buffer (%)	0.00	12.00	4.33	4.66
7	Shaded walk (%)	0.00	46.00	13.90	27.14
8	Sidewalk encroachment (%)	0.00	52.00	24.80	3.28
9	Availability of pedestrian amenities (number)	0.00	2.00	0.15	0.44
10	Amenities for physically handicapped people (%)	13.80	18.9	15.23	3.42
11	Sidewalk obstructions (number)	3.00	8.00	5.6	1.16
12	Cleaner sidewalks (%)	28.00	82.30	76.20	2.39
13	Presence of police patrolling (yes = 1/no = 0)	0.00	1.00	0.16	0.56
14	Unattended dogs (yes = 1/no = 0)	0.00	1.00	2.45	0.56
15	Presence of street light (number)	0.00	6.00	0.40	1.15
16	Presence of CCTV cameras (number)	0.00	3.00	52.30	0.80
17	Traffic speed (km/h)	40.50	60.95	722.08	3.01
18	Traffic volume (number)	312.00	1174.00	0.40	237.34
19	Presence traffic signs and signals (number)	0.00	4.00	0.32	0.76
20	Availability of street furniture (number)	0.00	1.00	0.45	0.53
21	Interesting things around (yes = 1/no = 0)	0.00	1.00	2.92	0.32
22	Trees and landscapes (yes = 1/no = 0)	0.00	1.00	0.79	0.98
23	Presence of guard rails (%)	49.00	95.50	86.13	69.32
24	Raised/segregated sidewalks	0.00	1.00	0.80	0.45

due to the fact that major number of school trips uses school bus or public transport for their journey and their perception on PLOS could be insignificant.

Model 1 included all factors of objective measurement along with the socio-demographic factors of walking. All objectively measured built environment factors showed a significant relation with the perceived pedestrian level of service. This implies that each of the objectively measured factors has a significant influence on PLOS of sidewalks irrespective of their signs. 'Comfort' and 'Security' measure obtained significant negative loading on PLOS which indicates that it is difficult to measure comfort factors manually for PLOS evaluation. These factors vary from person to person and it actually depends on the majority of user's perception. The obstruction free, clean and shaded pedestrian network with considerable number of crossing facilities provide a comfortable walking environment. In addition, facilities for persons with physical disability such as tactile pavements, ramps, etc. are provided

Table 8 Model analysis results

Variables		Model 1		Model 2		Model 3	
		Coeff.	Sig.	Coeff.	Sign.	Coeff.	Sign.
Dependent variable PLOS of sidewalks LOS A, LOS B, LOS C, LOS D, LOS E and LOS F							
Socio demographic and trip characteristics	Gender (males)	− 0.41	0.00	− 0.46	0.00	− 0.47	0.00
	Age (18–45 years)	− 0.05	0.52	− 0.14	0.10	− 0.09	0.33
	Income (above 20,000)	0.08	0.05	0.02	0.69	0.05	0.31
	Walking time (≤10 min)	− 0.10	0.31	− 0.21	0.23	− 0.22	0.04
	Walking distance (upto 500 m)	0.08	0.04	0.09	0.04	0.04	0.37
	Work (public/private service)	0.07	0.04	0.09	0.020	0.10	0.03
	Walk purpose (education)	0.05	0.65	0.05	0.666	0.05	0.67
Objective variables	obj_comf	− 0.27	0.00			− 0.04	0.68
	obj_mobility	1.28	0.00			0.87	0.03
	obj_security	− 0.49	0.00			− 0.31	0.22
	obj_safety	0.51	0.00			− 0.39	0.11
Subjective variables	sub_comfort			0.20	0.04	0.25	0.03
	sub_mobility			1.93	0.00	− 2.22	0.00
	sub_secuirty			0.13	0.00	0.45	0.06
	sub_safety			0.76	0.00	0.77	0.00
Model fit statistics							
Cox and snell		0.462		0.539		0.629	
Nagelkerke		0.482		0.578		0.676	
Mc-Fadden		0.298		0.331		0.401	

wherever necessary. A secure pedestrian network has adequate number of street lighting and CCTV cameras for surveillance and it also provides police patrolling especially at night. All these factors have psychological traits. Specifically, these factors can be measured in a way that people internalize them from the environment they interact with; therefore, they cannot be measured as such easily by any audit methodology.

5.5 Association of Subjectively Measured Factors with PLOS

Micro level subjective factors of built environment are positively associated with pedestrian level of service. These factors when measured objectively have also showed a positive association with perceived PLOS of sidewalks except ‘Comfort’ and ‘Security’ measures. The comfort factor was associated with pedestrian level of service both objectively and subjectively but with opposite signs, which indicates that ‘comfort’ factor when measured objectively has less impact on perceived PLOS. The significance of ‘Mobility’ is not a surprising association on pedestrian level of service of sidewalks, as this measure is significant in almost all past studies of pedestrian level of service [31]. ‘Mobility’ measure has a significant positive association with PLOS when measured objectively and subjectively. This result implies that factor ‘Mobility’ measured in perception survey or by audit can impact the PLOS. Model 2 integrated socio-demographic factors and subjective factors of walking environment. In this model, all factors of subjective measurement are significant in predicting Pedestrian Level of Service. All the factors of subjective measures of walking environment such as Comfort, Safety, Security and Mobility are positively associated with perceived pedestrian level of service. Results imply that all environmental factors of walking both objectively and subjectively measured are significantly associated with PLOS and walking behavior. Therefore, both qualitative and quantitative measures can be subjectively measured to determine its influence with the PLOS of sidewalks (Table 8).

5.6 Association of Objectively and Subjectively Measured Factors with Pedestrian Level of Service

The final model (Model 3) includes both perceptual and objective measures. Almost all factors remained and preserved same directionality (Table 8). ‘Comfort’ and ‘Safety’ factor are again negatively associated with PLOS from the perspective of objective evaluation as in the Model 1. This result may imply that in case of measures such as ‘Comfort’ and ‘Safety’, what people experience from environment is quite different from what is there in reality. And both actual and perceived environmental measures are potentially related to behavioral characteristics. Even though indices are developed to be equal, they are not the mirror images of each other. This may be due to the mismatch in measurement of factors. Interestingly, only ‘Mobility’ measure is the significant objective measure associated with PLOS in the final model. All other association of objective measures on PLOS is insignificant. In the final model, most of the subjective factors have a significantly a good association with PLOS and walking behavior than objective factors. These results imply that ‘Mobility’ factor which being a quantitative factor can be measured quantitatively as well as qualitatively for the measurement of PLOS, but other qualitative factors such as safety, security and comfort can only be measured subjectively or perceptually for the analysis of PLOS. Model 3 is compared with others for fit indices, which indicated that Model 3 has significantly better fit than other models.

5.7 Association of Objectively and Subjectively Measured Factors with Pedestrian Level of Service

The final model (Model 3) includes both perceptual and objective measures. Almost all factors remained and preserved same directionality (Table 8). ‘Comfort’ and ‘Safety’ factor are again negatively associated with PLOS from the perspective of objective evaluation as in the Model 1. This result may imply that in case of measures such as ‘Comfort’ and ‘Safety’, what people experience from environment is quite different from what is there in reality. And both actual and perceived environmental measures are potentially related to behavioral characteristics. Even though indices are developed to be equal, they are not the mirror images of each other. This may be due to the mismatch in measurement of factors. Interestingly, only ‘Mobility’ measure is the significant objective measure associated with PLOS in the final model. All other association of objective measures on PLOS is insignificant. In the final model, most of the subjective factors have a significantly a good association with PLOS and walking behavior than objective factors. These results imply that ‘Mobility’ factor which being a quantitative factor can be measured quantitatively as well as qualitatively for the measurement of PLOS, but other qualitative factors such as safety, security and comfort can only be measured subjectively or perceptually for the analysis of PLOS. Model 3 is compared with others for fit indices, which indicated that Model 3 has significantly better fit than other models.

6 Conclusion

Researchers have started giving importance to psychological factors in the travel behavior analysis. In order to increase the understanding of human behavior, understanding of environment is necessary. People behave in accordance with the emotions and other psychological factors. Most of the past PLOS studies have focused on objective assessment of environment. This may be due to the fact that for a large sample, it is difficult to capture peoples’ perception of the environment in consistent manner. This study attempts to bring all the factors, perceptions, behavior and environment together. It tries to explore the relationships to better understand how PLOS is determined for sidewalk and which types of measurement of environmental factors have high association with the PLOS. The results of this study support some direct relationship between the environment and Pedestrian Level of Service (PLOS) of sidewalks. While both subjective and objective measures for mobility factor is strongly associated with PLOS, factors of other domains yielded weaker and non-significant associations. The study results imply that subjective/perceptual data along with objective evaluation of environment are significant in many ways and play a good role in predictions of Pedestrian Level of Service analysis.

The study indicates some of the environmental measures are only significant when they are measured subjectively. For instance, the factors such as comfort, safety and security are only significant when measured subjectively. Therefore, in order to completely understand the association between environment and PLOS of sidewalks, it is essential to assess each of the environmental factors separately, i.e. some factors should be measured objectively and some factors should be measured subjectively. The study results prove that when factors which are measured objectively and subjectively are taken together, they reveal more about the association of environmental measures and walking behavior (PLOS). The study supports the need to measure the qualitative environmental factors subjectively in the PLOS modelling.

References

1. Appleyard D (1981) Livable streets. University of California Press, Berkeley
2. Wilbur J, Chandler PJ, Dancy B, Lee H (2003) Correlates of physical activity in urban Midwestern African–American women. Am J Prev Med 25(3):45–52
3. Hoehner CM, Ramirez LKB, Elliott MB, Handy SL, Brownson RC (2005) Perceived and objective environmental measures and physical activity among urban adults. Am J Prev Med 28(2):105–116
4. Handy SL, Boarnet MG, Ewing R, Killingsworth RE (2002) How the built environment affects physical activity: views from urban planning. Am J Prev Med 23(2):64–73
5. Saelens BE, Sallis JF, Black JB, Chen D (2003) Neighborhood-based differences in physical activity: an environment scale evaluation. Am J Public Health 93(9):1552–1558
6. Lee C, Moudon AV (2004) Physical activity and environment research in the health field: implications for urban and transportation planning practice and research. J Plann Liter 19(2):147–181
7. Lin L, Moudon AV (2010) Objective versus subjective measures of the built environment, which are most effective in capturing associations with walking? Health Place 16(2):339–348
8. Handy S, Cao X, Mokhtarian PL (2006) Self-selection in the relationship between the built environment and walking: empirical evidence from Northern California. J Am Plann Assoc 72(1):55–74
9. Moudon AV, Lee C, Cheadle AD, Garvin C, Johnson DB, Schmid TL, Weathers RD (2007) Attributes of environments supporting walking. Am J Health Promot 21(5):448–459
10. Greenwald MJ, Boarnet MG (2001) Built environment as determinant of walking behavior: analyzing nonwork pedestrian travel in Portland, Oregon. Transp Res Rec 1780(1):33–41
11. Cervero R, Kockelman K (1997) Travel demand and the 3Ds: density, diversity, and design. Transp Res Part D Transp Environ 2(3):199–219
12. Eyler AA, Brownson RC, Bacak SJ, Housemann RA (2003) The epidemiology of walking for physical activity in the United States. Med Sci Sports Exerc 35(9):1529–1536
13. Huston SL, Evenson KR, Bors P, Gizlice Z (2003) Neighborhood environment, access to places for activity, and leisure-time physical activity in a diverse North Carolina population. Am J Health Promot 18(1):58–69
14. Alfonzo M, Boarnet MG, Day K, Mcmillan T, Anderson CL (2008) The relationship of neighbourhood built environment features and adult parents' walking. J Urban Des 13(1):29–51

15. Ewing R, Handy S (2009) Measuring the unmeasurable: urban design qualities related to walkability. J Urban Des 14(1):65–84
16. IRC: 103-2012 (2012) Guidelines for pedestrian facilities. The Indian Roads Congress, New Delhi
17. UTTIPEC, Unified Traffic and Transportation Infrastructure (Planning and Engineering) Centre Regulations (2010). Annual report
18. Parida P, Parida M (2007) Development of qualitative evaluation methodology for sidewalks in Delhi. ITPI J 4(3):27–33
19. Pikora T, Bull F, Jamrozik K, Knuiman M, Giles-Corti B, Donovan R (2002) Systematic pedestrian and cycling environmental scan (SPACES). Survey of the physical environment in local neighborhoods: observer's manual. University of Western Australia, Nedlands, Western Australia
20. Clifton KJ, Smith ADL, Rodriguez D (2007) The development and testing of an audit for the pedestrian environment. Landsc Urban Plan 80(1–2):95–110
21. Landis B, Vattikuti V, Ottenberg R, McLeod D, Guttenplan M (2001) Modelling the roadside walking environment: pedestrian level of service. Transp Res Rec J Transp Res Board 1773:82–88
22. Sarkar SS (1994) A method for evaluation of urban pedestrian spaces. Ph.D. thesis, Department of City and Regional Planning, University of Pennsylvania, Philadelphia, PA, 194
23. Gallin N (2001) Quantifying pedestrian friendliness–guidelines for assessing pedestrian level of service. Road Transp Res 10(1):47
24. Asadi-Shekari Z, Moeinaddini M, Shah MZ (2014) A pedestrian level of service method for evaluating and promoting walking facilities on campus streets. Land Use Policy 38:175–193
25. Muraleetharan T, Adachi T, Uchida KE, Hagiwara T, Kagaya S (2003) A study on evaluation of pedestrian level of service along sidewalks and at intersections using conjoint analysis. In: Annual meeting of Japanese society of civil engineers (JSCE) infrastructure planning, Toyohashi, Japan
26. Jensen SU (2007) Pedestrian and bicyclist level of service on roadway segments. Transp Res Rec 2031(1):43–51
27. Gliem JA, Gliem RR (2003) Calculating, interpreting, and reporting Cronbach's alpha reliability coefficient for Likert-type scales. In: Midwest research-to-practice conference in adult, continuing, and community education
28. NATPAC (2014) Annual report. Transportation planning and research wing. National Transportation Planning and Research Centre
29. Pollard TM, Wagnild JM (2017) Gender differences in walking (for leisure, transport and in total) across adult life: a systematic review. BMC Publ Health 17(1):341
30. Smith AL (2009) Contribution of perceptions in analysis of walking behavior. Transp Res Rec 2140(1):128–136
31. Kang L, Xiong Y, Mannering FL (2013) Statistical analysis of pedestrian perceptions of sidewalk level of service in the presence of bicycles. Transp Res Part A Policy Practice 53:10–21

Development of Public Transport Serviceability Index for Metro Cities—A Case Study of Kochi City

S. Shaheem, K. Shijil, and S. Sreelekshmi

1 Introduction

Developing countries like India are in search of sustainable solutions to their transportation problems arising due to exponential growth of vehicular traffic in its cities. Finding sustainable solution is extremely difficult due to the unplanned growth of cities coupled with rapid pace of urbanization, mixed traffic conditions, poor service level of public transport facilities and deteriorating air quality.

In the past decade, Kochi has witnessed increased economic growth with investments in projects such as Kochi metro, Vallarpadom International Container Terminal (VICT), shopping malls, Smart City and Info park. All these investments are boosting the regional economy and employment, resulting in increased vehicular traffic in the city roads. Kochi has diverse public transport mode choices such as bus, ferry, train and metro rail services. But the city is experiencing increased dependency on private motor vehicles for personal trips, leading to increased vehicular congestion, accidents and emissions. The earlier studies in Kochi have indicated the inadequacy of existing transportation infrastructure to serve the future travel demand and a high growth in the private vehicle share in the city.

A measure of existing public transportation system and its shortcomings and the need for coordination between different modes of transport need to be considered developing policy measures for improving the traffic scenario of the city. The concept of public transport serviceability index is used for evaluating the performance of the public transportation system of the city, with the help of which users can presumably discern various levels of serviceability among different cities and modes of transport. The serviceability of a public transport system is a user's judgement of the level of service that a particular mode of transport provides at any point in time. By finding serviceability index values, it becomes possible to see the period of time during

S. Shaheem (✉) · K. Shijil · S. Sreelekshmi
National Transportation Planning and Research Centre, Thiruvananthapuram, Kerala, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_23

which the system provided acceptable service. The study aims to find how the public transportation system of Kochi city performed through the concept of serviceability and serviceability index.

2 Scope and Objectives

The main objectives of this study are

1. To study the characteristics of existing public transportation system of the city
2. To develop a public transport serviceability index for Kochi city
3. To suggest policy measures for increasing the public transportation services in Kochi.

The scope of the study was limited to three major metro stations namely, Kaloor, Edappally and Aluva, and its immediate surroundings.

3 Literature Review

Errampalli [1] developed a methodology to determine the level of integration (Sustainability Integration Index) between metro rail and bus incorporating sustainability. The approach identified 12 indicators under three main domains of sustainability namely, economic, social and environmental, and it was used as a measure of existing level of integration between metro rail and Buses. The methodology developed was applied for evaluating policies referring for integrating Metro and Bus service in Delhi. The study found that the policy of increasing bus frequencies resulted in maximum Sustainability Integration Index value of 4.8% average increase for all four metro stations followed by the policy of relocating bus stops to improve connectivity and the policy of common fare collection with 4.3% and 3.8% increase, respectively.

Litman [2] proposed guidelines for the selection of indicators for sustainable transportation planning. It described factors to consider while selecting indicators and identified potential problems with conventional indicators. With suitable examples, the paper provides recommendations for selecting indicators for use in a specific transportation problem. Jeon [3] found that traditional method of analysing transportation sustainability considered only the transportation system effectiveness, efficiency and environmental impacts of the system. Author observed that the existing indicator systems did not capture the importance of education and infrastructure security as critical tool for moving infrastructure systems towards sustainability. The study proposed refining the existing visions and indicator systems to support the progress towards sustainable infrastructure systems.

4 Study Area

Kochi is a large city in Kerala and is widely referred as the commercial capital of Kerala. Recently, Kochi has witnessed increased economic growth with major investments in projects such as Kochi metro, Vallarpadom International Container Terminal (VICT), Lulu mall, Smart City and Info park resulting in boosting the regional economy and employment in the city. The city, which is greatly dependent on public transport for its mobility needs, has following modes of Public transport:

- Metro
- Public transport bus
- Inland waterways (Ferry)
- Sub urban train services.

In case of inland waterways, majority of the boats are old and are in a dilapidated state due to which the water transport system is losing out on competing with other motorized modes in the city. Currently, the operational frequency of ferry system is very low resulting in lesser mode share and the sub-urban train services also offer less connectivity when compared to metro and bus. Thus the present study considered only the two modes, Metro rail and bus in developing the public transport serviceability index. Kochi Metro starts from Aluva and ends at Pettah covering a length of about 28 km. The Metro project is likely to be extended to SN junction near Thrippunithura soon as the construction is under progress. At present Kochi metro operates from Aluva to Maharajas College Junction. The study focuses on three major metro stations namely, Edappally, Kaloor, Aluva and surrounding bus stops. Edappally metro station witnessed the highest boarding/alighting of about 7000 passengers/day, followed by Aluva station with 5500 passengers and Kaloor station with 2000 passengers per day.

5 Methodology

5.1 General

The core part of the study lies in the formulation of a well-defined methodology that ensures an effective analysis and proposals for formulating a public transport serviceability index for Kochi city. The methodology adopted for the study is as shown in Fig. 1.

A detailed literature survey was conducted to identify the study area in Ernakulam city. The survey locations were selected which were the three metro stations at Kaloor, Edappally and Aluva. These three stations were selected as there was mass movement of people across the influence zone of these three stations when compared to the others. An expert opinion survey was conducted to assign suitable weights to the factors selected for the study. Data were collected from study area by passenger

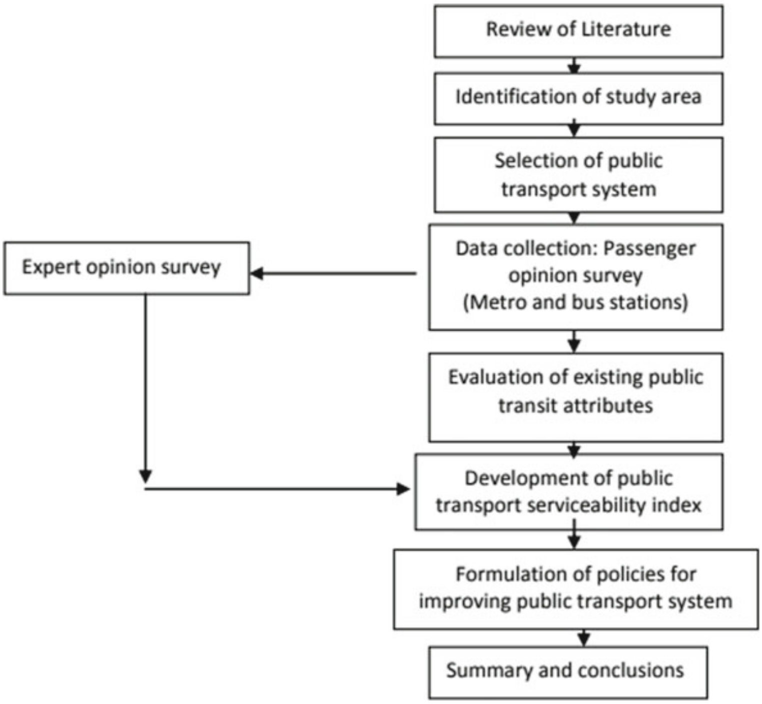


Fig. 1 Study methodology

opinion surveys among the users of car, bus, two wheeler, auto and pedestrians. The passenger opinion survey was conducted at all the three metro stations and bus stops. From passenger opinion surveys the existing public transport attributes are identified. The collected data were analysed, and recommendations were framed.

Sample size depends upon the public transport trips under study. In the city of Kochi, the number of public transport trips per day is 87031. The minimum sample size obtained by taking a confidence level of 95% is 1054.

6 Data Collection

For developing the public transport serviceability index, different data related to the existing public transport system of Kochi city were needed to be collected.

6.1 Expert Opinion Survey

An expert opinion survey was conducted among 30 expert personnel in the field of transportation planning, and they were provided with a list of 14 factors, and were asked to rank the factors according to their significance. The 14 factors were: accessibility, travel cost, transfer time, comfort and convenience, safety and security, parking cost, travel time, reliability, frequency, crew behaviour, passenger information system, travel speed, weather conditions and waiting time.

6.2 Passenger Opinion Survey

The passenger opinion survey was conducted at the metro stations and bus stops across the study area in order to collect the details regarding socioeconomic as well as travel characteristics of commuters. It was done at metro stations and bus stops located at Kaloor, Edappally and Aluva. The survey was conducted from 9:00 AM to 5:00 PM at the study locations covering the peak hours. The socioeconomic characteristics were assessed by considering factors such as age, gender, nature of job, monthly income and vehicle ownership. Travel characteristics included distance from home to office, mode of conveyance, reasons for choosing the mode, waiting time, in-vehicle travel time, cost of travel, etc. A user perception questionnaire was also included to get the commuter ratings about the major travel characteristics such as: travel time, travel cost, waiting time, reliability, transfer time, comfort and convenience, safety and security, frequency, accessibility and in-vehicle time/out vehicle time factor. Thus, weightages were assigned to each of the attributes by analysing the rating provided by the commuter.

7 Preliminary Data Analysis

The collected data was sorted and organized in Microsoft Excel into a format given in the following tables to fit for analysis. The following shows the summary of the characteristics sample of commuters studied. Table 1 gives the summary of the sample of commuters collected by passenger opinion survey.

Sample size had a total of 1100 passengers interviewed out of which male and female participation as 63% and 37%, respectively. Average income of 47% of the commuters using the metro had a value in between Rs. 10,000 and Rs. 20,000. Majority of the commuters opinioned that the metro fare was very high compared to the bus services. It indicated that the lower income people had a low proportion among the commuters travelling in metros. It was found that there would be a 7.8% increase in metro users with a 25% reduction in metro fare. As the income increased, the choice for the commuters widened and the use of private vehicles became predominant in

Table 1 Summary of the sample of commuters collected by passenger opinion survey

Characteristics	Classification	Sample size (%)
Gender	Male	62.8
	Female	37.2
Age	< 18	5.1
	18–25	37.9
	26–40	34.2
	41–60	17.6
	> 60	5.1
Education	Upto 12th Std.	17.7
	Graduate	58.2
	Post graduate	8.3
	Professional	13.1
	Illiterate	2.7
Occupation	Govt. employee	2.5
	Private employee	35.4
	Student	27.4
	Business	10.7
	Housewife	14
	Retired	3.3
	Unemployed	3.1
	Others	3.6
Vehicle ownership	No vehicle	43.4
	Only TW	40.2
	Only car	2.3
	Both TW and car	14.2

their travel choices. Figures 2 and 3 denote the income distribution and trip purpose distribution of the respondents, respectively.

It was observed that 56% of the commuters had an access distance more than 1 km followed by 23 % of commuters having access distance between 250 and 500 m. It was found that majority of the commuters having less access distance to the bus tops or metro stations (i.e. less than or equal to 250 m) chose walking to reach the stations or stops. As the access distance increased, preference for walking decreased and the selection of two wheeler as the mode increased. It was also found that majority of the commuters having less access distance to the destinations from the bus tops or metro stations (i.e. less than or equal to 250 m) chose walking as their mode. As the egress distance increased, the mode shares of car and two wheeler increased.

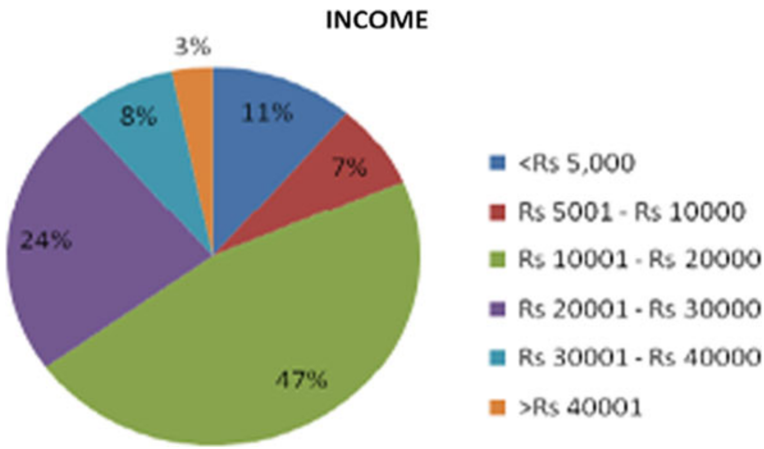


Fig. 2 Income distribution

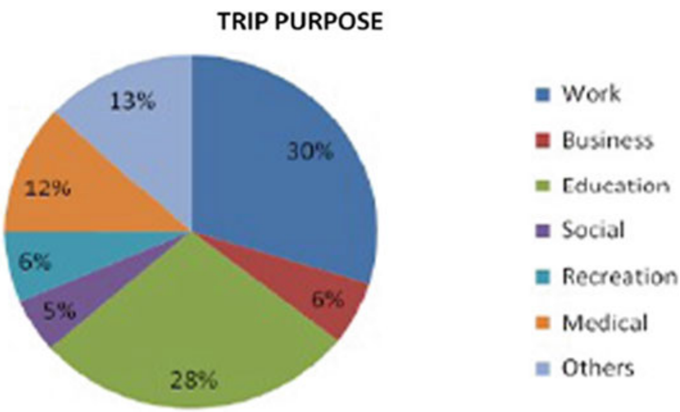


Fig. 3 Trip purpose distribution

8 Estimation of Sustainability Indicators and Sustainability Integration Indices for the Study Area

The final aim of this study is to determine the serviceability index value for Edappally, Kaloor and Aluva in the Kochi city which indicates the existing level of service offered by the public transportation system as well as to compare the service aspects of the public transport system in the three metro stations as well as of the bus stops. An expert opinion survey was conducted among 30 expert personnel in the field of traffic and transportation planning. While conducting expert opinion survey, the commuters were asked to rate the travel attributes on a ten point scale so as to obtain weightages corresponding to their significance in the mode choice decision

Table 2 Weightages for public transport attributes

S. No.	Attributes	Weightages (1–10)
1	Travel time	9.67
2	Travel cost	9.33
3	Waiting time	8.56
4	Reliability	8.33
5	Transfer time	8.67
6	Comfort and convenience	8.22
7	Safety and security	8.33
8	Frequency	8.44
9	Accessibility	8.67
10	In vehicle time/out vehicle time	9.00

of a public transport user. Assigning the weightages to the public transit attributes according to their relevance in a public transport commute is a significant step towards development of public transport serviceability index. Values nearer to 10 indicated that the variable was considered as major factor in the selection of public transport mode by a commuter, whereas values nearer to 1 indicated less significance for the variable. Table 2 shows the weightage assigned to each public transport attribute. The data collected from passengers waiting at metro stations and bus stops was used to estimate the value of serviceability indicators based on the quantitative measures obtained in the survey field and the number of commuters. The summary of indicators is presented in Table 3. The average value of serviceability indicators for each of the metro station was computed by giving equal importance to each of the individual indicators. However, it may not be true that all the indicators have equal importance.

9 Estimation of Public Transport Serviceability Index

The public transport serviceability index value representing the existing level of service provided by public transport system is estimated for all the three metro station locations. The weightage obtained for indicators based on an expert opinion survey as shown in Table 3, and the values estimated for various serviceability indicators through commuter opinion survey are collated and expressed in Table 2. The weighted value for every indicator is calculated by taking the product of average value of the indicator obtained by conducting opinion survey among commuters at metro station and bus stops and weightage of the indicator resulting from commuter rating. The final public transport serviceability index value was then obtained by accumulating the weighted value of all the indicators are shown in Table 4.

From Table 4, it is found that Kaloor metro station and bus stops around the metro station have the highest public transport serviceability index value with the maximum value of serviceability index of 48.86, followed by Aluva with 38.40 and Edappally

Table 3 Summary of serviceability indicators obtained from commuter survey

S. No.	Indicators	Value of public transport indicators					
		Kaloor		Edappally		Aluva	
		Metro station	Bus stop	Metro station	Bus stop	Metro station	Bus stop
1	Travel time	0.38	0.29	0.47	0.43	0.39	0.41
2	Travel cost	0.37	0.34	0.39	0.26	0.26	0.25
3	Waiting time	0.42	0.40	0.39	0.26	0.43	0.32
4	Reliability	0.90	0.40	0.90	0.26	0.9	0.32
5	Transfer time	0.29	0.37	0.27	0.24	0.15	0.23
6	Comfort and convenience	0.28	0.21	0.26	0.17	0.25	0.17
7	Safety and security	0.26	0.22	0.26	0.22	0.25	0.23
8	Frequency	0.8	0.26	0.80	0.29	0.8	0.24
9	Accessibility	0.19	0.16	0.22	0.21	0.26	0.20
10	In vehicle time/out vehicle time	2.09	2.47	1.43	0.31	1.27	1.42

Table 4 Summary of computation of serviceability index for study locations

S. No.	Indicators	Value of public transport indicators		
		KALOOR	EDAPPALLY	ALUVA
1	Travel time	3.24	4.35	3.87
2	Travel cost	3.31	3.03	2.38
3	Waiting time	3.51	2.78	3.21
4	Reliability	5.41	4.83	5.08
5	Transfer time	2.86	2.21	1.65
6	Comfort and convenience	2.01	1.77	1.73
7	Safety and security	2.00	2.00	2.00
8	Frequency	4.47	4.60	4.39
9	Accessibility	1.52	1.86	1.99
10	In vehicle time/out vehicle time	20.52	7.83	12.11
Public transport serviceability index		48.86	35.27	38.40

with 35.27. The public transportation system at Kaloor offers the best level of service among the three locations studied owing to the better physical integration between the metro station and the bus stops at Kaloor. The commuters have bus stops and NMT facilities closer to the metro stations enabling them to make transfers easily. Waiting time and transfer time are comparatively less at the Kaloor location than the other

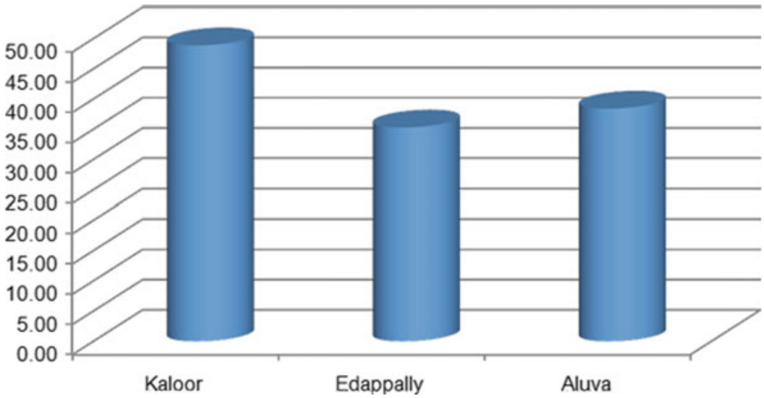


Fig. 4 Estimated sustainability integration index in the study area

two locations. Figure 4 illustrates the graphical representation of the sustainability index values. The less value obtained for Edappally can be explained due to the poor connection between the bus stop and the metro station when compared to the other two locations. Most of the commuters arriving at the Edappally metro station make their way to the Lulu Mall through the flyover and very less wait to make transfers to the bus or NMT facilities.

10 Conclusions

It was observed that 56% of the commuters had an access distance more than 1 km followed by 23% of commuters having access distance between 250 and 500 m. The average ridership of the commuters is ranging from 30,000 to 40,000 commuters per day during normal working days. It was also found that majority of the commuters having less access distance to the bus tops or metro stations (i.e. less than or equal to 250 m) chose walking to reach the stations or stops. As the access distance increased, preference for walking decreased and the selection of two wheeler as the mode increased.

Another observation was that majority of the commuters having less access distance to the destinations from the bus stops or metro stations (i.e. less than or equal to 250 m) chose walking as their mode. As the egress distance increased, the mode shares of car and two wheeler increased. Average income of 47% of the commuters using the metro had a value in between Rs. 10,000 and Rs. 20,000. Majority of the commuters opinioned that the metro fare was very high compared to the bus services. About 7.8% of the commuters expressed willingness to shift to metro with a 25% reduction in metro fare.

From the analysis, it was found that work trip was the predominant trip among the commuters. The most relevant indicators identified from expert opinion survey for using public transport modes were: travel time, travel cost, waiting time, reliability, transfer time, comfort and convenience, safety and security, frequency, accessibility and in-vehicle time/out-vehicle time factor. The public transport serviceability index value representing the existing level of service provided by public transport system (metro and bus) was estimated for all the three metro station locations of Edappally, Kaloor and Aluva in the Kochi city.

It was found that that Kaloor location has the highest public transport serviceability index value with the maximum value of serviceability index of 48.86, followed by Aluva with 38.40 and Edappally with 35.27. The public transportation system at Kaloor offers the best level of service among the three locations studied owing to the better physical integration between the metro station and the bus stops at Kaloor. The commuters have bus stops and NMT facilities closer to the metro stations enabling them to make transfers easily. Waiting time and transfer time are comparatively less at the Kaloor location than the other two locations. The least value obtained for Edappally can be explained due to the poor connection between the bus stop and the metro station when compared to the other two locations. Most of the commuters arriving at the Edappally metro station make their way to the Lulu Mall through the flyover and very less wait to make transfers to the bus or NMT facilities. The serviceability index value at Edappally and Aluva can be improved by enhancing physical integration between the metro and the bus. This can be achieved by proper placement of bus stops enabling convenient transfers for the commuters within the public transportation system.

References

1. Madhu E, Patil KS, Prasad CSRK (2018) Evaluation of integration between public transportation modes by developing sustainability index for India cities. J World Conf Transp Res Soc:1–18
2. Litman T (2017) Developing indicators for comprehensive and sustainable transport planning. Victoria Transport policy Institute, Canada, pp 10–15
3. Jeon C, Amekudzi A (2005) Addressing sustainability in transportation systems: definitions, indicators and metrics. J Infrastruct Syst:31–52

Economic Evaluation of a Highway Project Using HDM-4: A Case Study of Widening of Ahmedabad-Bagodara National Highway

Yash Tahiliani, L. B. Zala, and Pinakin Patel

1 Introduction

Road transport, in a developing country like India, plays an exceptional role in economic development. The construction of roads and operation of traffic on the roads leads to various advantages resulting in benefits to all other sectors of economy. Highway is an important construction of civil engineering, which requires huge amount of expenditure. Economic evaluation of highway also known as highway project appraisal is a method in which the costs and benefits from a road project are computed for a selected time horizon and analyzed by a particular point of reference [1]. Also, economic nalysis serves a number of purposes such as preparing highway plans at various levels, prioritizing various highway schemes, comparison of mutually exclusive projects, determining whether the project is worth investing, analyzing alternative strategies, standards and policies.

Y. Tahiliani (✉)
Transportation Engineering, Birla Vishvakarma Mahavidyalaya Engineering College,
Vallabh Vidhyanagar, Anand, Gujarat, India

L. B. Zala · P. Patel
Department of Civil Engineering, Birla Vishvakarma Mahavidyalaya Engineering College,
Vallabh Vidhyanagar, Anand, Gujarat, India
e-mail: lbzala@bvmengineering.ac.in

P. Patel
e-mail: pinakin.patel@bvmengineering.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_24

1.1 Literature Review

Many research works have been carried out in the field of economic evaluation of highways and other transportation projects using IRC method and HDM-4. Transportation funding must be allocated properly for constructing and maintaining a good transportation system and for providing the infrastructure required for growth and economic development of a region.

Availability of funds sometimes restricts the augmentation of capacity on the transportation network. Economic viability of the project on the region, combined with the impacts of the project to the economy, should be considered whilst making important decisions on allocating transportation funds [2]. As transportation plays and outstanding role in a country's economic growth and huge investments that are required, a thorough economic evaluation of these investments is unavoidable. Use of current methods of economic appraisal, their improvement and incorporation of the LCCA into the investment decision process is studied [3]. Benefits of construction of new road infrastructures in Chandigarh city of Punjab have been evaluated by carrying out benefit–cost analysis of the expenditures involved and returns obtained in terms of economic internal rate of return. Also, sensitivity analysis has been carried out to check the economic viability of the project [4]. Optimum maintenance treatment for the urban road network in Chennai has been determined from the various alternatives recommended in Government of India specifications using HDM-4 [5]. Calibration of the HDM-4 for pavement deterioration models has been conducted for a national highway network located in the Uttar Pradesh and Uttaranchal of India. Data for cracking, ravelling, potholing and roughness have been collected, analyzed and used for calibration of the HDM-4 [6]. HDM model level-1 calibration according to local conditions has been carried out for a state highway in Uttar Pradesh. Factors viz., roughness-age environment, crack initiation and crack propagation have been computed [7]. The purpose of evaluating mutually exclusive highway alternatives is to choose the most beneficial one for project implementation. Different analytical techniques for evaluation of alternatives like cost-effectiveness (C/E) technique, benefit-to-cost ration (B/C) technique, internal rate of return (IRR) technique, payoff period (PP) technique have been explained [8]. HDM-4 software can be used for aiding the highway planners for allocation of funds and also for determining priorities to increase the effectiveness of expenses incurred in the construction and maintenance of pavement. The strategic analysis of a network of selected urban roads of Noida City has been carried out using HDM-4 software for maximizing the NPV and minimizing the costs for gaining a target international roughness index [9].

1.2 Methods of Economic Evaluation

A number of methods have been developed, and the literature on them is voluminous. The various important methods are as follows:

1. Net present value (NPV) method
2. Benefit/cost (B/C) ratio method
3. Internal rate of return (IRR) method.

1.3 Introduction to HDM-4

Economic analysis is important parameter in making decisions related to highway investments. World Bank's HDM-4 software is an important tool to which not only performs economic analysis but is also helpful in planning maintenance strategies for highways and also to perform life cycle cost analysis. The three major applications of HDM-4 are

1. Strategy analysis: It includes planning of strategy for highways like maintenance, reconstruction, rehabilitation, etc.
2. Project analysis: It includes economic analysis for highway projects, estimation of road user costs and benefits, etc.
3. Programme analysis: It includes prioritization of a defined long list of candidate road projects into a one-year or multi-year work programme under defined budget constraints.

2 Research Objective

The main aim of this study is to carry out economic analysis of widening of Ahmedabad-Bagodara national highway using HDM-4 software. The objective of this study is to study various aspects of economic appraisal to determine necessity of widening and to check the economic feasibility of widening of Ahmedabad-Bagodara national highway using IRC SP:30-2009 and HDM-4 along with sensitivity analysis.

3 Methodology

The methodology in this study involves six stages. In the first stage, the stretches have been identified, selected, and the road inventory data have been collected. In the second stage, classified traffic volume count study has been carried out for the stretch for 7 days-24 h. Pavement condition survey has been performed for measuring the surface distresses like pothole, ravelling, patching, edge breaking and surface cracking. Pavement composition and soil CBR have also been determined. In the third stage, secondary data like topographical data and petrol-pump sales data have been collected which serve as an input to economic analysis.

Traffic has been converted from average daily traffic (ADT) to annual average daily traffic (AADT) by using seasonal correction factors. In the fourth stage, capacity analysis has been performed to determine the necessity for widening the existing highway. In the fifth stage, economic analysis has been performed using IRC SP:30-2009 along with sensitivity analysis to find out the economic indicator results. In the sixth stage, economic analysis has been performed using HDM-4 along with sensitivity analysis, and economic viability of project has been determined based on the values of the NPV, B/C ratio and economic internal rate of return (EIRR).

4 Data Collection

4.1 Study Area Profile

The national highway under consideration is a part of NH-47 which connects Rajkot and Ahmedabad cities of India. Study area starts form the outskirts of Ahmedabad, from Changodar village and traverses and connects various cities/villages like Bavla, Bhayala and Bagodara with Ahmedabad. The highway provides connectivity to Jamnagar and Mundra ports with the rest of the nation. These centres are to become crucial given the likely high level of investments getting attracted in the SEZs therein. The total length of the study area is approximately 33.465 km. The widening of the existing project from 4 to 6 laning divided carriageway will not only result in reduced travel time and increase in speed, but will also lead to safe and efficient traffic operation and will accommodate the future growth in traffic.

4.2 Road Inventory

The existing pavement width consists of an average 7.0 m carriageway and 1.5 m shoulder on both sides including median of 2.0 m width. There are around 30 culverts, 10 minor and 1 major road over bridge along the length of highway. Also, there are 2 major junctions and 22 minor junctions which are present at the existing highway.

4.3 Traffic Survey

Classified traffic volume count survey is carried out to determine volume of different categories using roads. In this study, classified traffic volume count has been performed for 7 days, 24 h as suggested in IRC SP:19-2001 [10]. Traffic volume count has been carried for NH-47, stretch starting from Ch.0+000 to Ch.33+465, i.e., from Ahmedabad (Changodar) to Bagodara at Ch.19+465 for both sides.

LHS indicates direction of travel from Ahmedabad to Rajkot.

RHS indicates direction of travel from Rajkot to Ahmedabad (Table 1).

Table 1 Classified traffic volume count

Direction	2W	3W	Car	Goods pickup	BUS	LCV	Truck/HCV	MAV	Tractor	Tractor trailer	Cycle
LHS	1975	673	5717	627	793	447	1972	1017	15	142	23
RHS	2082	624	6517	675	804	748	1320	1248	25	141	24
Total	4057	1297	12,234	1302	1597	1195	3292	2264	40	282	47

4.4 Pavement Condition Survey

Pavement condition survey has been performed to determine various types of pavement distresses. This is an essential survey to be performed as it gives the amount of different distresses present on the existing pavement and it also serves as an input parameter to HDM-4 based on which vehicle operating costs and maintenance requirements are evaluated. Various types of distresses observed on the existing highway are cracking, pothole, ravelling, rutting and edge breaking.

4.5 Pavement Composition and Soil CBR

Flexible pavement comprises of different layers. These layers are granular sub-base (GSB), water bound macadam (WBM), bituminous macadam (BM), bituminous concrete (BC) and semi-dense bituminous concrete (SDBC), etc. The thickness of all these layers needs to be known because it serves as an input parameter in HDM-4 to compute the structural number of pavement (SNP). The main application of California bearing ratio (CBR) is to evaluate the stiffness modulus and shear strength of subgrade. The CBR of the subgrade layer needs to be known because it serves as an input parameter in HDM-4 to compute the structural number of pavement (SNP).

4.6 Topographical Data

Topographical parameters like altitude and rise/fall have considerable effect on the vehicle operating costs (VOC), and thus these parameters need to be given equal importance. The topographical parameters like altitude and rise/fall have been determined with the help of Google Earth.

4.7 Petrol-Pump Sales Data

For conversion of average daily traffic (ADT) to annual average daily traffic (AADT), seasonal correction factors (SCF) are required. These seasonal correction factors are based upon the round-year traffic census, and collection of such data is a relatively difficult task. Hence, petrol-pump sales data have been collected from Arth petroleum located at Ch. 16+280 km.

5 Capacity Analysis

Capacity analysis is fundamental to the planning, design and operation of roads. For the purpose of augmentation of the facilities and up gradation of the project road, the design service volume for 4 lane roads LOS B is 45,000 pcu/day, and for LOS, C is 72,000 pcu/day as per Indian highway capacity manual (2012–2017) [11]. The traffic projection has been made for total ADT by considering a growth rate of 8.00% as suggested by IRC SP:30-2009 [1]. This traffic projection has be compared with the design service volume for each year and the necessity to widen/augment the facility with year has been determined with the help of capacity analysis (Fig. 1).

From the projection and figure of capacity analysis, it can be understood that this indicates a clear necessity for widening of this highway to 6 lanes in order to keep traffic movement continuously plying on the highway with sufficient operating requirements along with safety.

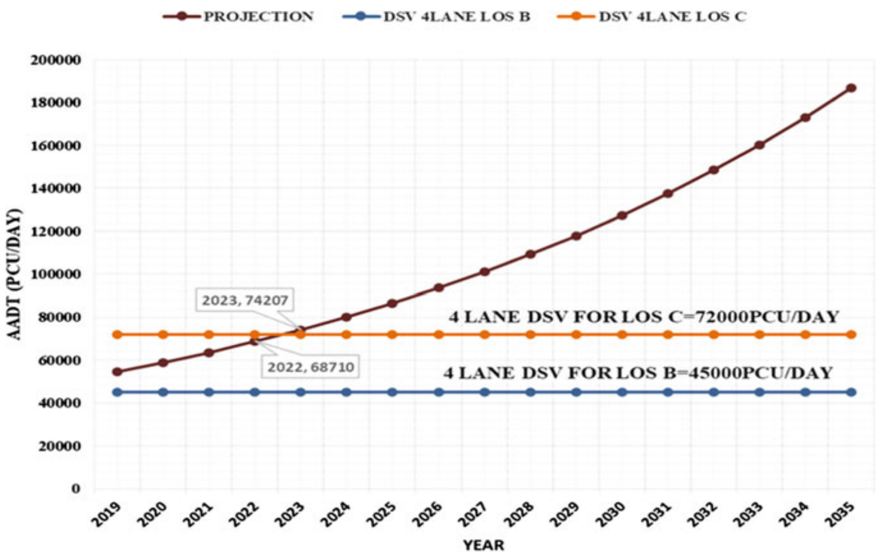


Fig. 1 Capacity analysis

Table 2 PCU values as per Indian highway capacity manual (2017)

S. No.	Vehicle type	PCU value
1	Standard car	1.0
2	Big car	1.45
3	2-wheeler (2W)	0.4
4	3-wheeler (3W)	1.2
5	Light commercial vehicles (LCV)	3.1
6	Two/three axle trucks	4.4
7	Multi-axle vehicles (MAV)	6.6
8	Bus	5.0
9	Tractor with trailer	6.2

6 Economic Evaluation Using IRC SP:30-2009

6.1 Traffic Forecast

Initially, forecast of traffic has been carried out for the base condition, i.e., do minimum (case-1) and proposed condition, i.e., widening to 6 lanes and strengthening the existing 4 lane pavement (case-2). The traffic volume data are expressed in terms of PCUs. The conversion of vehicles into PCUs is carried out by using the conversion Table 2 of Indian highway capacity manual (2017) [11].

The construction period is taken as 2 years, i.e., from 2019 to 2021. The analysis period has been taken 15 years as suggested in IRC SP:30-2009. The traffic forecast has been carried out from the year of 2019–2035 with growth rate of 8.00% for the base case and widening. Also, 10% of total daily traffic is assumed to present the peak hour traffic. The capacity of 4 lane road for the base condition is taken as 4540 pcu/h/direction and that of 6 lane road for widening condition is considered as 6790 pcu/h/direction as suggested by Table 2 of Indian highway capacity manual (2017).

6.2 Congestion Factors

The effect of congestion on VOC can be considered separately for the distance-related and time-related components. Distance-related congestion components are fuel, lubricants, tyre, spare parts and maintenance labour. Time-related congestion components that affect the vehicle operating costs are depreciation, fixed costs, crew wages, etc. Distance-related congestion factors and time-related congestion factors for the same traffic for analysis period of 15 years have been computed.

6.3 Vehicle Operating Costs

The vehicle operating costs (VOC) for different categories of vehicles have been determined with the help of procedure specified in IRC SP:30-2009 [1]. It may be noted that vehicle operating costs are affected by roughness of the pavement surface and rise/fall of the entire highway. In this study, average rise/fall determined with the help of Google Earth is 15. Also, the roughness data obtained from the R&B department, Ahmedabad indicates that the average roughness of road can be taken as 4000 mm/km. Hence, the VOC of all the vehicles has been computed for a RF of 15 and roughness of 4000 mm/km for case-1, i.e., 'do minimum' and roughness value of 2000 mm/km has been adopted for the case-2, i.e., upgraded 6 lane road. The VOC has been corrected for present day costs by using ratio of WPI value [12] of the present year and that of 2009.

6.4 Economic Evaluation

Based on the VOC of different vehicles considering both conditions, benefits have been worked out. Construction cost of highway is Rs. 414 crores as per R&B department, Ahmedabad. As per report of the committee on norms for maintenance of roads in India, MoRTH [13], the maintenance cost for the existing 4 lane pavement is taken as Rs. 59,106,000 per year and that for 6 lane road is taken as Rs. 76,490,000 per year (converted to present day costs). These benefits are then compared to the costs involved in the construction and maintenance of the highway, and thus economic analysis has been performed (Table 3).

The economic analysis results are stated below:

1. B/C ratio for the widened 6 lane highway is 2.14, which is greater than 1.
2. NPV for the highway is Rs. 4341.88 million, which is positive.
3. EIRR value is 28.14%, which is greater than the interest rate.

6.5 Sensitivity Analysis

Sensitivity analysis has been carried out to test the economic strength of the project. The variations in the following parameters have been examined, considering them to be on the conservative side (Table 4).

7 Economic Evaluation Using HDM-4

Economic evaluation has been carried out using HDM-4 based on costs and benefits, by comparing the total net benefits in 'do minimum' situation with 'widening

Table 3 Economic analysis as per IRC SP:30-2009 (all figures in Rs. lakhs)

Year	Case-1: do min.		Case-2: widening		Costs	Benefits	Net benefits	Disc. benefits
	Maint	VOC	Const. and maint	VOC				
2019	591.06	51,710.37	20,700.00	51,710.37	– 20,108.9	0.00	– 20,108.94	– 20,108.94
2020	591.06	57,578.43	20,700.00	57,578.43	– 20,108.9	0.00	– 20,108.94	– 17,954.41
2021	591.06	64,267.62	764.90	55,576.60	– 173.84	8691.02	8517.18	6789.84
2022	591.06	71,928.83	764.90	61,453.47	– 173.84	10,475.36	10,301.52	7332.41
2023	591.06	80,752.27	764.90	68,208.05	– 173.84	12,544.22	12,370.38	7861.60
2024	591.06	90,983.08	764.90	75,969.40	– 173.84	15,013.68	14,839.84	8420.52
2025	591.06	102,945.73	764.90	84,788.10	– 173.84	18,157.63	17,983.79	9111.15
2026	591.06	117,084.86	764.90	94,835.67	– 173.84	22,249.19	22,075.35	9985.77
2027	591.06	130,929.43	764.90	106,315.79	– 173.84	24,613.63	24,439.79	9870.82
2028	591.06	141,403.78	764.90	119,471.15	– 173.84	21,932.63	21,758.79	7846.44
2029	591.06	152,716.08	764.90	134,239.51	– 173.84	18,476.57	18,302.73	5892.99
2030	591.06	164,933.37	764.90	151,123.99	– 173.84	13,809.38	13,635.54	3919.89
2031	591.06	178,128.04	764.90	170,254.50	– 173.84	7873.54	7699.70	1976.32
2032	591.06	192,378.28	764.90	189,373.96	– 173.84	3004.32	2830.48	648.67
2033	591.06	207,768.55	764.90	204,523.88	– 173.84	3244.67	3070.83	628.35
2034	591.06	224,390.03	764.90	220,885.79	– 173.84	3504.24	3330.40	608.45
2035	591.06	242,341.23	764.90	238,556.65	– 173.84	3784.58	3610.74	588.99
						EIRR	28.13	B/C 2.14
							NPV	4341.88 (million)

Table 4 Sensitivity analysis

S. No.	Conditions	EIRR (%)
1	Increase in costs by 20%	23.71
2	Decrease in benefits by 20%	22.77
3	Increase in costs by 20% and decrease in benefits by 20%	18.76

to 6 lanes and strengthening the existing 4 lane pavement' situation. The term 'do minimum' is defined as the base strategy for economic analysis, i.e., without project situation. The term 'widening to 6 lanes and strengthening the existing 4 lane pavement' is defined as widening to 6 lane dual carriageway with paved shoulder configuration of NH-47.

7.1 Sectioning of the Study Area

Taking into account the different conditions of the study area and construction of 6 lane along the length of 33.465 km, the study area has been divided into 14 sub-sections for economic analysis. Details of these sections are given in Table 5.

Table 5 Sections of the study area

Section ID	Chainage (km)		Side	Length (km)
	From	To		
1	0 + 000	5 + 000	LHS	5
2	0 + 000	5 + 000	RHS	5
3	5 + 000	10 + 000	LHS	5
4	5 + 000	10 + 000	RHS	5
5	10 + 000	15 + 000	LHS	5
6	10 + 000	15 + 000	RHS	5
7	15 + 000	20 + 000	LHS	5
8	15 + 000	20 + 000	RHS	5
9	20 + 000	25 + 000	LHS	5
10	20 + 000	25 + 000	RHS	5
11	25 + 000	30 + 000	LHS	5
12	25 + 000	30 + 000	RHS	5
13	30 + 000	33 + 465	LHS	3.465
14	30 + 000	33 + 465	RHS	3.465
Total length of the study area				33.465

7.2 Construction and Analysis Period

The construction period is selected as 2 years. The construction work is expected to be initiated by 2019, and the new facility is expected to come in operation from 2021; therefore, the construction cost phasing has been distributed as 50–50% over the stated 2 years. The analysis period is taken as 15 years.

7.3 Project Cost

In the present study, the construction cost has been taken from the R&B department, Ahmedabad. The financial cost of construction is Rs. 460 crores. Hence, this cost needs to be converted to economic cost for carrying out the analysis.

For this, a conversion factor of 0.9 is taken as suggested by IRC SP:30-2009, and thus the economic cost of the project is taken as Rs. 414 crores.

7.4 Standard Conversion Factors

In economic analysis, the prices of all components used are in economic terms. This is because of all the distortions in prices relating to labour wages, capital market, transfer payments need to be corrected. For the purpose of economic analysis, financial prices have been converted to economic prices using a factor of 0.9, as suggested by IRC SP:30-2009.

7.5 Traffic Composition Data

For each road section, traffic level is specified in terms of average annual daily traffic (AADT) flow. Traffic data have been derived from the primary surveys like classified traffic volume count survey which was conducted for 7 days-24 h at chainage 19 + 465 km. Total AADT for LHS is taken 12,332 and that for RHS is taken 13,069. The traffic composition for the vehicle fleet data is given in Table 6.

7.6 HDM-4 Calibration Parameters

The calibration (level-1) to the road deterioration and work effects has been carried out for the sensitive parameters. The level-1 calibration parameters with their respective values for calibrating HDM-4 model are as given in Table 7.

Table 6 Traffic composition

Categories	LHS composition (%)	RHS composition (%)
2-wheeler	15.53	15.46
3-wheeler	5.30	4.63
Car	42.19	45.37
Goods pickup	4.63	4.70
Bus	5.85	5.60
LCV	3.30	5.21
Trucks/HCV	14.55	9.19
MAV	7.50	8.69
Tractor	1.21	1.15
Total MT traffic (%)	100.00	100.00

Table 7 HDM-4 level-1 calibration parameters

Parameters	Initiation	Progression
All structural cracking	0.9	1.1
Wide structural cracking	1.0	1.0
Transverse thermal cracking	1.0	1.0
Ravelling	1.0	1.0
Pothole	1.0	1.0
Edge break	1.0	
Initial densification	1.0	
Structural deterioration	1.0	
Roughness env. coefficient	1.25	
Roughness progression	1.0	

7.7 Road Network Data

The basic road network data used as input is as given Table 8.

7.8 Vehicle Fleet Data

Vehicle operating costs (VOC) is the major component of economic analysis. The primary inputs required in HDM, to estimate the VOCs, are the prices of vehicles, tyre, petrol, diesel, lubricants, crew cost, maintenance cost, vehicle utilization and other vehicle fleet characteristics. The vehicle fleet data provided as an input in HDM-4 is as given in Table 9.

Table 8 Road network data

	Section ID													
Description	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Existing pavement type	Asphalt mix on granular base													
Average carriageway width (m)	7	7	7	7	7	7	7	7	7	7	7	7	7	7
Average shoulder width (m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Number of lanes	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Rise + fall (m/km)	9.5	9.5	13	13	10	10	8	8	10	10	9	9	5.2	5.2
Altitude (m)	26	26	26	26	25	25	21	21	17	17	13	13	13	13
Avg. roughness (m/km)	4	4	4	4	4	4	4	4	4	4	4	4	4	4
Total area of cracking (%)	0.0036	0.0032	0.0143	0.0047	0.0462	0.0037	0.3751	0.0139	0.2777	0.0171	0.1662	0.0147	0.0462	0.0551
Ravelled area (%)	0.0086	0.0968	0.0023	0.0837	0.1522	0.1867	0.0510	0.0475	0.0037	0.019	0.0017	0.0029	0	0
No. of potholes (no./km)	1	2	1	2	0	1	1	0	1	1	1	1	1	1
Mean rut depth (mm)	1.3	0.7	0	17	1.2	3.8	0	4	0	0.4	0	0.4	0	0

Table 9 Vehicle fleet data

Vehicle type	2-W	3-W	Car	Goods pickup	Bus	LCV	Truck/HCV	MAV	Tractor
Base type	Motor cycle	Auto	Car	Pickup vehicle	Bus	Light truck	Heavy truck	Multi-axle	Tractor
PCSE	0.5	0.70	1	1	1.5	1.3	1.4	1.6	1.9
No. of wheels	2	3	4	4	6	4	6	14	4
No. of axles	2	2	2	2	2	2	2	4	2
Tyre type	Bias-ply	Bias-ply	Radial-ply	Radial-ply	Bias-ply	Bias-ply	Bias-ply	Bias-ply	Bias-ply
Base number of recaps	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Retread cost (%)	15	15	15	15	15	15	15	25	15
Annual kilometre (km)	10,000	40,000	45,000	54,000	146,000	50,000	85,000	86,000	9000
Working hours	550	1460	730	1800	2920	1825	2200	2555	730
Average life (years)	8	6	10	15	8	8	8	10	8
Private use (%)	100	0	100	0	0	0	0	0	100
Passenger	2	5	5	6	56	2	2	2	2
Work-related passenger trips (%)	80	75	75	75	75	0	0	0	0
ESALF	0	0	0	0	1	0.27	4.25	3	1
Operating weight (tons)	0.2	0.8	1.25	1.4	10	4.5	14	18	8

7.9 HDM-4 Configuration

It is essential to configure the model to local conditions in HDM-4. The configuration of the model, therefore, includes traffic flow pattern, speed-flow type and Currency. Inter-urban traffic pattern has been selected for the study area. Speed-flow type for the project has been defined for 4 lane having ultimate capacity of 2000 PCSE/lane/h and 6 lane having ultimate capacity of 2400 PCSE/lane/h.

7.10 Unit Costs for Vehicles

A unit cost database has been prepared for new vehicle price, replacement of tyre, petrol/diesel/engine oil, body-building costs of vehicles, etc., by referring certain guidelines given by HDM-4 itself (Table 10).

7.11 Maintenance Strategy and Cost

Maintenance strategy adopted for the analysis has been given in Table 11.

7.12 Results of Economic Evaluation

Economic analysis conducted using HDM-4 has stated the below mentioned results:

1. NPV for the highway is Rs. 7394.63 million which is positive.
2. EIRR value is 32.1% which is greater than the interest rate (Fig. 2).

7.13 Sensitivity Analysis

Sensitivity analysis has been carried out using HDM-4, considering the following scenarios (Table 12).

8 Conclusions

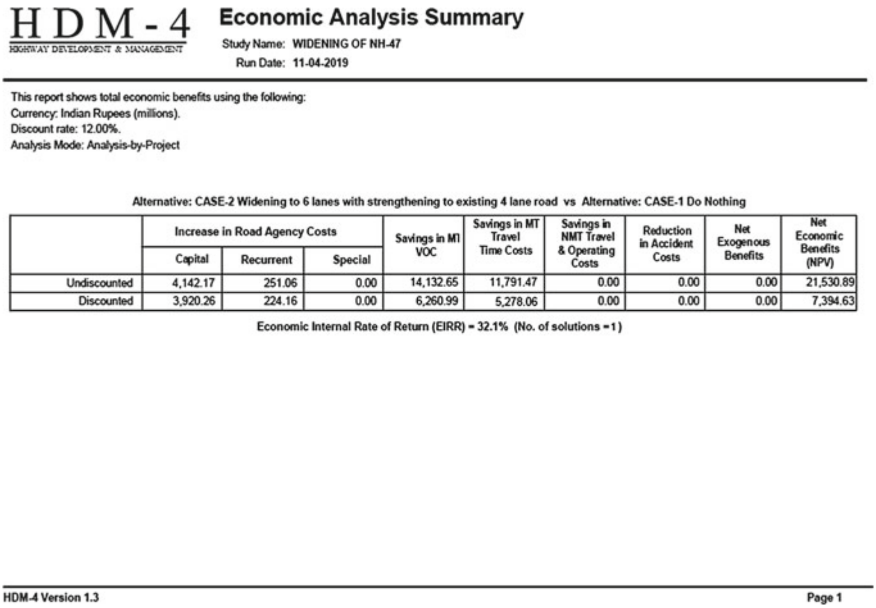
Traffic volume on Ahmedabad-Bagodara national highway NH-47 is 59,726 pcu/day for both directions which is more than the design service volume for LOS B for 4 lane roads. Also, as discussed in capacity analysis, the traffic volume will cross the

Table 10 Unit costs for vehicles

Vehicle type	2-W	3-W	Car	Bus	LCV	Truck/HCV	MAV	Tractor
New vehicle (Rs.)	54,090	230,260	763,220	1,102,520	652,750	1,141,900	1,287,960	616,000
Replacement tyre (Rs.)	990	1675	2950	9980	4620	9360	9360	11,400
Fuel (Rs./l)	28	28	28	28	28	28	28	28
lubricating oil (Rs./l)	87	87	87	180	87	180	180	87
Maintenance labour (Rs./h)	6	3	6	6	2	4	4	3
Crew wages (Rs./h)	0	50	0	80	20	52	52	30
Annual over head	5000	10,000	20,000	200,000	120,000	400,000	400,000	120,000
Annual interest (%)	12	12	12	12	12	12	12	12
Passenger working time (Rs./psngr. h)	40	30	65	40	0	0	0	0
Passenger non-working time (Rs./psngr. h)	2	3	4.5	5	0	0	0	0
Cargo (Rs./veh. h)	0	0	0	2	1	3	5	0

Table 11 Maintenance strategy and cost

Year		Type of maintenance		Cost of maintenance (Rs.)			
Do min.	Widening	Do min.	Widening	Do min.		Widening	
				Economic (m^2)	Financial (m^2)	Economic (m^2)	Financial (m^2)
Periodic maintenance (scheduled for every 5th year)							
2019 2024 2029 2034	2026 2031	Overlay 25 mm BC	Overlay 40 mm BC	334	367	534	588
Routine maintenance (scheduled for every year)							
All years other than stated above		Patching	Patching	104	114	104	114
		Crack sealing	Crack sealing	82	90	82	90



HDM-4 HIGHWAY DEVELOPMENT & MANAGEMENT

Economic Analysis Summary

Study Name: WIDENING OF NH-47

Run Date: 11-04-2019

This report shows total economic benefits using the following:
Currency: Indian Rupees (millions).
Discount rate: 12.00%.
Analysis Mode: Analysis-by-Project

Alternative: CASE-2 Widening to 6 lanes with strengthening to existing 4 lane road vs Alternative: CASE-1 Do Nothing

	Increase in Road Agency Costs			Savings in MT VOC	Savings in MT Travel Time Costs	Savings in NMT Travel & Operating Costs	Reduction in Accident Costs	Net Exogenous Benefits	Net Economic Benefits (NPV)
	Capital	Recurrent	Special						
Undiscounted	4,142.17	251.06	0.00	14,132.65	11,791.47	0.00	0.00	0.00	21,530.89
Discounted	3,920.26	224.16	0.00	6,260.99	5,278.06	0.00	0.00	0.00	7,394.63

Economic Internal Rate of Return (EIRR) = 32.1% (No. of solutions = 1)

HDM-4 Version 1.3 Page 1

Fig. 2 HDM-4 economic analysis summary

Table 12 Sensitivity analysis as per HDM-4

S. No.	Conditions	EIRR (%)
1	Increase in capital cost by 20%	28.3
2	Decrease in traffic growth by 20%	26.7
3	Increase in capital cost by 20% and decrease in traffic growth by 20%	23.3

design service volume for LOS C for 4 lane roads, there is necessity to widen it to 6 lane highway in order to keep traffic movement continuously plying on the highway with sufficient operating requirements along with safety. The results of economic analysis of widening of highway carried out using IRC SP:30-2009 are stated below:

1. B/C ratio for the upgraded 6 lane highway is 2.14, which is greater than 1.
2. NPV for the highway is Rs. 4341.88 million, which is positive.
3. EIRR value is 28.14%, which is greater than the interest rate.

Economic analysis conducted using HDM-4 has stated the below mentioned results:

1. NPV for the highway is Rs. 7394.63 million, which is positive.
2. EIRR value is 32.1%, which is greater than the interest rate.

Also the results of sensitivity analysis in each and every case indicate that the EIRR is favourable. The variation in the results obtained by using IRC SP:30-2009 and

HDM-4 is substantial. This is due to the fact that the equations given in IRC SP:30-2009 are developed for Indian conditions, whereas the equations in HDM-4 model are calibrated for the same. The certain limitations of method of economic analysis using IRC SP:30-2009 are equations of calculating distance-related congestion factor and time-related congestion factor, and vehicle operating costs are given only up to 4 lane highways and not for 6 lane/8 lane highways. Also, the class of vehicles for which DRCF and TRCF equations are given are only limited to 2-wheeler, car, bus, truck/HCV and MAV. There is no provision in the code for other vehicles like 3-wheeler, goods pickup vehicle, tractor. This leads to variation in the results.

Thus, due to above mentioned limitations of IRC SP:30-2009, it cannot be used for multilane highways and other important highway constructions in which large amount of investments is involved and for which there is no any clearly specified provision in the code. HDM-4 covers the above-mentioned limitations of IRC SP:30-2009, given that entire data set required for performing economic analysis is available with the analyst. Thus, from the results of economic evaluation of Ahmedabad-Bagodara national highway, it is concluded that the 6 lane highway is more economical as compared to existing 4 lane highway.

References

1. IRC: SP:30-2009. Manual of economical evaluation of highway projects in India. Indian Roads Congress
2. Schultz GG, Stewart AL (2007) Evaluating economic analysis alternatives in the transportation decision making process. In: International conference on transportation engineering, pp 3133–3139
3. Tomek R, Vitasek S (2016) Improvement of economic effectiveness of road highway projects. Proc Eng 164(164):395–401
4. Mohapatra DR (2015) An economic analysis of improvement of road infrastructure: a case study. Euro Acad Res 2(11):14636–14651
5. Sudhakar R (2009) Pavement maintenance management system for urban roads using HDM-4. Indian Geotechnical Society Chennai, Chap 31–37
6. Jain SS, Aggarwal S, Parida M (2005) HDM-4 pavement deterioration models for Indian national highway network. J Transp Eng 131(8):623–631
7. Bagui S, Ghosh A (2015) Level 1 calibration of HDM-4 analysis with a case study. Malaysian J Civ Eng 27(1):121–143
8. Khasnabis S, Mishra S, Safi C (2012) Evaluation procedure for mutually exclusive highway safety alternatives under different policy objectives. J Transp Eng 138(7):940–948
9. Shah YU, Jain SS, Tiwari D (2016) Adaptation of HDM-4 tool for strategic analysis of urban roads network. Transp Res Proc 17:71–80
10. IRC: SP:19-2001. Manual for survey, investigation and preparation of road projects. Indian Roads Congress
11. CSIR—Central Road Research Institute, New Delhi (2017) Indian highway capacity manual (Indo-HCM)
12. Office of the Economic Adviser, Ministry of Commerce & Industry. www.eaindustry.nic.in/display_data_201112.asp
13. Ministry of Road Transport & Highways (2001) Report of the committee on norms for maintenance of roads in India. Indian Roads Congress, New Delhi

BPNN (ANN) Based Operating Speed Models for Horizontal Curves Using Naturalistic Driving Data

Suresh Nama, Gourab Sil, Akhilesh Kumar Maurya, and Avijit Maji

1 Introduction

Speed is a significant component that governs the geometry of the highway. AASHTO [1] states that the selected design speed determines the geometric elements, such as horizontal alignment, the cross-section dimensions, and vertical alignment. It also recommends that the design speed selected should provide safety and consistent with the drivers' speed.

Krammes [2] stated that the AASHTO design speed policy could not guarantee uniform operating speeds on highways with lower design speeds. McLean [3], Fitzpatrick et al. [4], and Nama et al. [5] observed that the 85th percentile (operating) speeds over curves on rural highways are higher than the curve design speeds. To counteract this gap in design and 85th percentile speeds and improve safety, researchers introduced design consistency. To maintain consistency, the gap between the design and 85th percentile operating speeds should not be more than 10 kmph [6]. However, this method oversees 15 percent of the traffic by considering the 85th percentile speed. Bonneson [7] suggested using other higher percentile speed values in curve design to cover a much wider range of road users. Especially, the Green Book published in 1994 [8] states that for a curve design, the 95th percentile speed of the

S. Nama (✉) · A. K. Maurya
Department of Civil Engineering, IIT Guwahati, Guwahati, India
e-mail: suresh.nama@iitg.ac.in

A. K. Maurya
e-mail: maurya@iitg.ac.in

G. Sil
Department of Civil Engineering, IIT Indore, Indore, India

A. Maji
Department of Civil Engineering, IIT Bombay, Mumbai, India
e-mail: avimaji@civil.iitb.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_25

passenger car traffic stream is the best representative of the design speed. In addition to higher speeds 85th, 95th, the lower speeds like 15th percentile speeds would also help in improving road geometry by making curve travelling more comfortable for the travellers in heavier and slow-moving vehicles.

Hence, rather than considering single speed (85th percentile) in design, it is suggested to incorporate other percentile speeds in design to improve highway safety. Tarris [9] and Fitzpatrick [10] also suggested using the entire speed distribution to develop speed models instead of focusing on a particular percentile to understand speed behaviour and proper geometric design.

Therefore, the objectives of this study were (1) to develop a speed model to predict any percentile speed (V_p) rather than predicting just the 85th percentile speed; (2) to identify the geometric parameters that influence the drivers speed. These percentile speed models (V_p) are further helpful for consistency analysis, design speed selections, design of posted speeds, and studying speed behaviour concerning curve geometry, further considering the growth in infrastructure projects such as constructing four-lane highways yearly. The present study is focused on four-lane highways only.

2 Speed Models

Over the past five decades, researchers started developing speed models to predict the operating speeds on highways. These speed models were started using extensively in the design consistency approach to improve safety. Figueroa and Tako [11] developed the 85th percentile speed model for two-lane highways using panel speed data by applying the ordinary least square regression method. The study showed that the parameters like site distance, degree of the curve, and superelevation influence the speed on a horizontal curve on two-lane highways. Jacob and Anjaneyulu [12] studied the response of the operating speed to the geometric parameters (radius and curve length). They found that the curve length has a negative impact on operating speed. They obtained the 85th percentile speed prediction model with R^2 of 0.80 for passenger cars at the centre of the curve. Gibreel et al. [13] developed regression-based 85th percentile speed models with vertical gradient and obtained the model Root Mean Square (RMSE) accuracy ranging from 5.0 to 7.5. Sil et al. [14] used the backward regression method to develop operating speed models on four-lane highways and obtained an R^2 of 0.72. Maji et al. [15] observed the relationship between the 85th and 98th percentile speeds at the centre of the curve and found that both were highly correlated and depend on the curve's length and deflection angle.

All the works conducted in the twentieth century used regression as an effective technique to develop speed models. However, the regression technique has its limitations, such as the initial assumptions in the modal formulation and defining the relationship between the parameters before starting the modelling. With the advancement of techniques in modelling like artificial neural network (ANN), the 21stcentury researchers in the field of transportation stated implementing ANN models to improve the models' accuracy by overcoming the assumptions and limitations made in linear regression techniques. McFadden et al. [16] compared the operating speed models developed for two-lane highways using ANN and regression methods with independent parameters curve length, degree of the curve, and deflection angle. They concluded that ANN offers predictive power compared with the regression techniques, but ANN does not need any of those linearity assumptions considered in regression methods. Hence, it is suggested to use ANN over regression. A similar study on multilane lane highways conducted in Egypt using the speed data collected from 78 curves [17]. The results proved that the ANN offers better logical results with confidence (better R^2) in comparison with the regression methods.

Studies on speed prediction models on two-lane rural highways indicate several important geometric parameters in determining speed on horizontal curves. Curve radius (R), superelevation (e), deflection angle (Δ), degree of curvature (D), length of the curve (CL), vertical gradient (VG), spiral length (SL), superelevation (SE), and cross-section (CS) are examples of parameters that have been used in the regression equations to predict operating speeds on horizontal curves [17]. Among these geometric parameters, the radius is found out to be the most critical parameter; hence, it has been used as a dominant predictor while deriving the operating speed models using regression methods. However, percentile speed prediction regression has some problems with data because for the same geometry (R, CL, SL, etc.,) it needs to predict different percentile speeds. Due to some limitations (with initial assumptions) in the regression method, the use of the neural network has been increased in the development of speed prediction models. Therefore, in the present study, prediction of percentile speeds, we also focused on using the artificial neural network.

3 Data Collection

In order to develop percentile speed (V_p) model, two kinds of data are needed (i) vehicle speed and (ii) road geometry. A detailed procedure of data collection techniques, processing, and extraction are explained in the following sections.

3.1 Speed Data

To capture the speed data, considered a 45 km rural four-lane highway (NH-40) between Jorabat (Assam) and Nongpoh (Meghalaya). This 45 km stretch covers a wider range of road geometry, such as gradients, curve radius, and curve length as shown in Table 1. Speed and vehicle GPS data were collected using 22 experienced drivers familiar with the selected study area. The vehicles used in data collection are drivers' own passenger cars, ensuring the driver's familiarity with the vehicle and the road. This familiarity with the vehicle and the route further helps the driver to travel naturalistic. All the vehicles used in data collection are equipped with a GPS device with a frequency of 10 Hz (i.e. 1 data point for every 0.1 s). The GPS device collects

Table 1 Geometric data descriptive statistics

	CL	R	DS	SL	SE	VGT	VGC	PTL
Mean	38.63	149.21	44.14	26.51	6.24	0.20	0.20	36.75
St. Dev	32.91	139.15	10.61	11.27	1.40	3.01	3.01	62.58
Minimum	30.00	20.00	20.00	0.00	2.50	−7.00	−7.00	0.00
Maximum	243.57	800.00	80.00	75.00	7.00	9.00	9.00	642.16

Where *CL* Curve length, *R* Radius, *DS* Design speed, *SL* Spiral length, *SE* Superelevation, *VGT* Tangent vertical gradient, *VGC* Curve vertical gradient, *PTL* Previous tangent length

the vehicle position (latitude, longitude) and the corresponding vehicle speed at each of its 0.1 s intervals. The longitudinal vehicle speed is measured with the help of the Doppler Shift technique using three or more satellite data simultaneously. The GPS device used has a speed-accuracy of ±0.1kmph and position accuracy of ±0.5m. Each day, at least one driver was picked among 22 multiple drives during the data collection period and ran the GPS-equipped vehicle over the 45 km stretch. The speed at the centre of the curves needed for model development is extracted from the collected continuous naturalistic data.

3.2 Geometric Data

All the necessary geometric data for this study are extracted from the actual drawings provided by the highway maintenance authority. In addition to the geometric data, the design speed (DS) data is also obtained from the highway authority. Further, these geometric parameters extracted from drawings are cross-verified using Laser Distance Metre (LDM) and field data collection. The descriptive statistics of the geometric parameters are presented in Table 1.

4 Development of ANN (BPNN) Model

The artificial neural network has a significant advantage over linear regression, as there is no limitation of using a specific model with well-defined inputs. Hence, neural networks can solve a wide range of problems with complex data and a large number of data points. The model learns through the dataset, trains itself, and adjusts its parameters according to the given dataset input and output values.

The ANN is structured in layers, namely the input layer, output layer, and hidden layer. Each of these layers consists of a different set of neurons. The input layer feeds the know information through the neurons; usually, the number of neurons in input layer is equal to the number of independent parameters. The output layer consists of neurons equal to the number of dependent or output parameters wanted to know.

Finally, in the hidden layers section, the number of layers is usually defined based on the trial and error process. However, for any continuous function single hidden layer is sufficient, provided with enough neurons [18]. Analysis in ANN starts with assigning randomized weights to the interconnections between different neurons and updating them in a series of iterations until a satisfactory level is achieved in predicting the output. The final weights in the interconnection will depend on the contribution performance of that neuron to the final output in a multi-layer perceptron. One of the commonly used algorithms to update the weights is the backpropagation algorithm. BPNN is the most popular and the oldest supervised learning multi-layer neural network algorithm proposed by Rumelhart et al. [19]. Therefore, in the present study, we used BPNN in order to predict the percentile speeds.

A three-layered neural network with nine nodes in the input layer and one in the output layer architecture is used to predict the percentile speeds. Hornik et al. [18] observed that any continuous function could be approximated with the use of a single hidden layer. Hence, a single hidden layer is considered. Through a trial and error process, the optimum number of neurons is obtained.

In the architecture defined in Fig. 1, the input layer takes the independent variables (CL, R, DS, ST, SE, VGT, and VGC. PTL, and Zp) as inputs. Among these inputs, Zp is the only parameter that varies with percentile speed, whereas the rest of the parameters remains the same for a given curve. Zp is a value obtained from the normality table for percentile (p). The reason behind considering Zp as one of the input variables is that the free-flow speed data follows a normal distribution [5, 20]. The value of the dependent variable percentile speed is obtained from the output layer. In order to train and test the network, the 5415 observation obtained from the

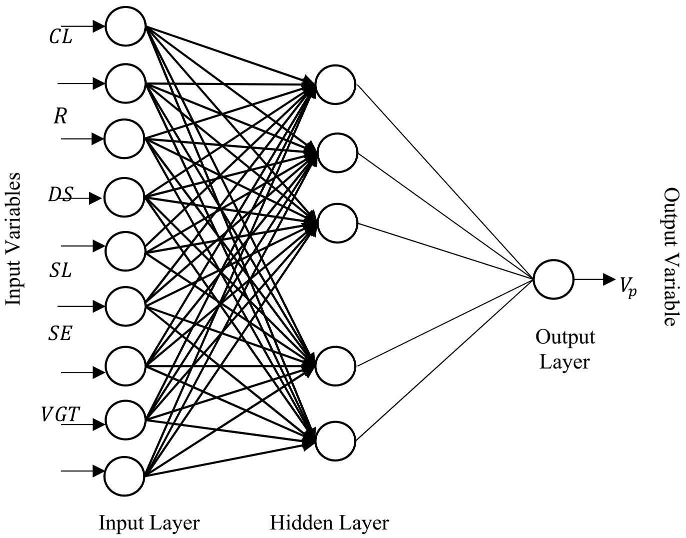


Fig. 1 Neural network architecture for V_p prediction

data generated from 285 curves is used. For each curve, the speed at every five-percentile interval is extracted, such that each curve produces 19-speed values. So, 285 curves produce 5415-speed values. Among this 80%, (4332) points are used to train the data, and the rest 1083 data points are used in testing.

The ANN network is trained using various activation functions (logistic, Tanh, Sigmoid) in various iterations using different batch and hidden layer neurons. The best model is obtained for sigmoid activation function (Eq. 1) at 890 Epochs with a batch size of 32 having 20 neurons in the hidden layer. The trained data gave an R^2 of 0.80 and RMSE of 4.63, and the tested data provided an R^2 of 0.78 and RMSE of 4.93.

$$S(x) = \frac{1}{1 + e^{-x}} \tag{1}$$

where:

x is the input value obtained from weights and before layer neurons.
$S(x)$ is the sigmoid function.

The developed ANN model results are tested for the normality of the speed data. The K-S normality test conformed that the predicted free-flow speed follows a normal distribution (Fig. 2). Hence with all these R^2, RMPS and normality test it can be said that this developed model predicts the Vp with better accuracy.

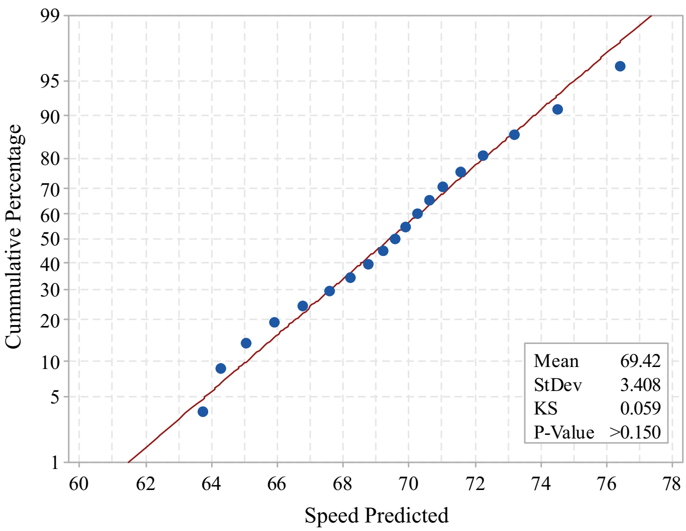


Fig. 2 Normal Q-Q plot for predicted speeds

5 Relative Influence (%) of Geometric Parameters on V_p

Relative influence is an essential analysis for any model to know how the input variables influence on the dependent variable or to say which input parameter dominates the results. In the present, ANN study the parameter influence studied using the 'Weights' method [21]. In the weights method, the actual model weights are considered to define the percentage influence of various input parameters. Here in our case, the ANN architecture consists of three layers. Therefore, using the following set of equations, measured the relative influence percentages of all the weights attributable to the given input variable.

$$Q_{ih} = \frac{|W_{ih}|}{\sum_{i=1}^{ni}|W_{ih}|} for i = 1\, to\, i\ followed\, by\, h = 1\, to\, h \tag{2}$$

where

- i no of neurons in the input layer.
- h no of neurons in the hidden layer.
- W_{ih} Weights in the interconnection between ith and hth neurons.
- Qih Relative weight influence of ith neuron in the interconnection between ith and hth neurons.

In the next step to measure RI %, the following equation is used

$$RI(\%) = \frac{\sum_{h=1}^{nh}(Q_{ih})}{\sum_{h=1}^{nh}\sum_{i=1}^{ni}(Q_{ih})} * 100 \tag{3}$$

where $i = i$th input parameter

The RI (%) of all the parameters is presented in Fig. 3. From this weighted method, it is observed that the DS has the most substantial influence of 16.3% among all the parameters and Z_p is showing the least RI (%) of 3.9%. The obtained RI (%) clearly states that for a unit percentage change in design speed changes, the change in percentile speeds is much higher than the rest of the parameters considered. Next, to design speed (DS), the spiral length (SL) and gradient over the curve (VGC) have higher RI (%) of 14%. The geometric parameters CL, RSE, and PTL have relative influence nearly equal to 10% each. So in designing, to bring the operating speeds closer to the given design speed, it is suggested to change first either SL or VGC. In places like mountainous where the speed is difficult to control by changing the curve geometry such as SL and VGC because of confined spaces, it is suggested to change superelevation (SE). Where superelevation (SE) does not need any additional space for corrections, and it had an RI (%) of above 10%.

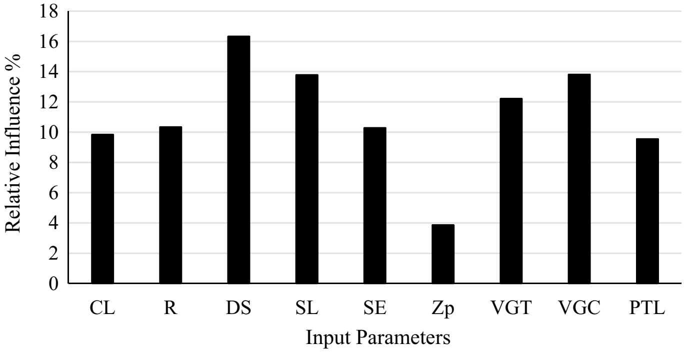


Fig. 3 Parameter RI (%) in predicting percentile speed (Vp). Where: Curve length (CL), Radius (R), Design Speed (DS), Spiral length (SL), Superelevation (SE), Tangent vertical gradient (VGT), Curve vertical gradient (VGC), previous tangent length (PTL)

6 Conclusions

The development of speed prediction models is one of the highly researched areas in highway engineering in order to use in a geometric design. Linear regression is a widely used method for developing speed models. Since linear regression has limitations and needs initial assumptions before modelling, there is a need to explore other methods to develop speed models, which can overcome the limitations of regression. Therefore, the present study attempted to use an artificial neural network method to develop the speed prediction models for four-lane horizontal curves.

A 45 km four-lane divided highway with 285 horizontal curves covering a wide range of geometry is considered in this study. A total of 22 drivers were used in this study to collect the naturalistic speed data using a vehicle-mounted GPS device. The GPS device used has an accuracy of ± 0.1 kmph in speed and ± 0.5 m in position. A total of 5415 data points are generated using the data collected from 285 curves. Out of this, 80% of data was used for neural network training, and 20% of data was used in testing the model.

A 3-layered neural network with nine inputs and one output is used for speed model development. The best results for the model are obtained at 890 epochs with a batch size of 32 having 20 neurons in the hidden layer. The trained data had an R^2 of 0.80 and RMSE of 4.63, and the tested data provided an R^2 of 0.78 and RMSE of 4.93.

To validate the normality of speed data, the predicted values from the model are tested using the K-S test. The K-S test revealed that the predicted speed values are following the normal distribution, satisfying the findings of the earlier studies conducted by Hashim [20].

To find the influence of input parameters on the output, the percentage relative influence values for each parameter are measured using the ANN weighted method. The results show that the design speed has the highest influence of 16% over the percentile speed, and Zp has the least influence of 4% among all the parameters considered for modelling.

One of the advantages of the present study is that with one single model, we can predict any percentile speed values and also say which parameter is influencing much on the percentile speeds. This model can also be used to predict operating speeds (i.e. 85th percentile speeds) without modifying the base model. Therefore, it can be further used to study the consistency of the highway geometry using the predicted 85th percentile speeds.

Further studies can be conducted on four-lane highways to predict the speeds at other locations on the curve, such as tangent, start, and end of the curve. A similar ANN method can also be used to develop a complete speed profile prediction method rather than a specific position speed prediction model.

References

1. AASHTO (2001) A policy on geometric design of highways and streets. American Association of State Highway and Transportation Officials, Washington, D.C.
2. Krammes RA (2000) Design speed and operating speed in rural highway alignment design. Transportation Research Record, Washington, D.C., pp 68–75
3. McLean J (1981) Driver speed behaviour and rural road alignment design. Traffic Eng Control 22(HS-032 471)
4. Fitzpatrick K, Krammes RA, Fambro DB (1997) Design speed, operating speed and posted speed relationships. Inst Transp Eng ITE J 67(2):52
5. Nama S, Maurya AK, Maji A, Edara P, Sahu PK (2016) Vehicle speed characteristics and alignment design consistency for mountainous roads. Transp Dev Econ 2(2):1–11
6. Lamm R, Choueiri EM (1987) Recommendations for evaluating horizontal design consistency based on investigations in the state of New York. Transp Res Record 1122(68):78
7. Bonneson J (2001) Controls for horizontal curve design. Transp Res Rec 1751(1):82–89
8. American Association of State Highway and Transportation Officials (1994) A Policy on Geometric Design of Highways and Streets. American Association of State Highway and Transportation Officials, Washington, D.C.
9. Tarris JP, Mason JM Jr, Antonucci ND (2000) Geometric design of low-speed urban streets. Transp Res Rec 1701(1):95–103
10. Fitzpatrick K, Carlson P, Brewer M, Wooldridge MD (2003) Design speed, operating speed, and posted speed limit practices. In: 82nd annual meeting of the transportation research board, Washington, DC.
11. Jacob A, Dhanya R, Anjaneyulu MVLR (2011) Geometric design consistency of multiple horizontal curves on two-lane rural highways. Procedia Soc Behav Sci 104:1068–1077
12. Gibreel GM, Easa SM, El-Dimeery IA (2001) Prediction of operating speed on three-dimensional highway alignments. J Transp Eng 127(1):21–30
13. Sil G, Nama S, Maji A, Maurya AK (2018) The 85th percentile speed prediction model for four-lane divided highways in ideal free flow condition. In: Transportation Research Board 97th Annual Meeting, Washington, D.C.
14. Sil G, Nama S, Maji A, Maurya A (2020) Modelling 85th percentile speed using spatially evaluated free flow vehicles and backward elimination technique to promote consistent horizontal curve design. J Transp Eng Part A: Syst (ASCE)
15. Maji A, Sil G, Tyagi A (2018) 85th and 98th percentile speed prediction models of car, light, and heavy commercial vehicles for four-lane divided rural highways. J Transp Eng Part A: Syst 144(5)

16. McFadden J, Yang WT, Durrans S (2001) Application of artificial neural networks to predict speeds on two-lane rural highways. Transp Res Rec 1751(1):9–17
17. Semeida AM (2014) Application of artificial neural networks for operating speed prediction at horizontal curves: a case study in Egypt. J Modern Transp 22(1):20–29
18. Hornik K (1991) Approximation capabilities of multi-layer feedforward networks. Neural Netw 4(2):251–257
19. Rumelhart DE, Hinton GE, Williams RJ (1985) Learning internal representations by error propagation (No. ICS-8506). University of California, San Diego
20. Hashim IH (2011) Analysis of speed characteristics for rural two-lane roads: a field study from Minoufiya Governorate, Egypt. Ain Shams Eng J 2(1):43–52
21. Gevrey M, Dimopoulos I, Lek S (2003) Review and comparison of methods to study the contribution of variables in artificial neural network models. Ecol Model 160(3):249–264

Modification of NPV Value Based on Real Option Study

Swapan Kumar Bagui, Atasi Das, and Yash Pandey

1 Introduction

The administration has granted the investment to construct different kind of facilities for civilizing people's lives. Road and rail network schemes need huge investment, extensive repayment period; the government investment for development/upgradation of the infrastructures is a monetary restriction on the management. The Private agencies' participations are the only solution to decrease the administration's financial loads through allowing PPP in the investment, building, function, and preservation of infrastructures [1]. PPP/Build Operate and Transfer (BOT) concord is useful for expansion of profitable infrastructures.

Discounted Cash Flow (DCF) analysis shall be a sound and popular technique which is effectively applied in viability analysis of a project. Therefore, the fiscal capability of BOT projects is determined based on the Net Present Value (NPV), Economic Internal Rate of Return (EIRR), or debt service coverage ratio, which are determined by DCF analysis [2].

DCF approach calculates fiscal capability of BOT/PPP projects with fixed hypothesizes namely requirement future traffic projection, working income, debt-equity ratio, finance cost, working price, building including allowance time. These are predetermined in DCF analysis. Practical variables for DCF analysis are essential to achieve consistent outcome.

Conventionally, a developer finalizes a BOT project based on NPV analysis considering DCF analysis. First step is to define developer's incomes and expenditures. The developer's money outflows of different projects are building costs,

S. K. Bagui
Intercontinental Consultants & Technocrats Pvt. Ltd., Delhi 110016, India
e-mail: swapan.bagui@ictonline.com

A. Das · Y. Pandey (✉)
G. R. Infraprojects Limited, Gurugram 122015, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_26

operations, preservation, and major and minor repair/maintenance costs, and a loan repayment action. Development costs are the first expense to make a BOT project. Funds development expenditures are yearly expenditure necessary to maintain the road in such a way that vehicles move safely. The refund plan summarizes the actual loan component and interest on loan amount of building-associated credit or money taken through bonds. Yearly builder's income begins after the project is opened for traffic. The projected builder's working income is mainly depended on money received from the traffic/road user. The charges for different road users are fixed in the initial concession contract and same shall be revised in future as per condition of the Concession Agreement. Developer's money is discounted to determine the developer's NPV. The selection of money off fee in the NPV calculation analysis is an individual and, therefore, it is tricky in project evaluation. The reduction fee represents the amount of comeback from the project, i.e., it is the risk-adjusted cost. The weighted Average Cost of Capital (WACC) and the Capital Asset Pricing Model (CAPM) are two popular methods which are generally used in the classification of the discount rate for BOT projects.

2 Limitations of NPV Analysis Approach

The traditional NPV analysis has two main restrictions for assessment of PPP projects: (1) the road user unacceptable behavior in the estimation process; and (2) choice of the prejudiced reduction fee. These are not taken care properly to determine the developer's income inflows. The concession period is generally so long, which makes it unfeasible prediction of future traffic. These two demand hazards do not correlate between actual traffic and projected traffic [3]. The adoption of reduction fee is serious for actual assessment of proposal since the project NPV is extremely responsive for changing reduction fee. However, NPV model is a method /approach for finalization of a BOT project. Real option study has been considered to improve NPV analysis.

3 Real Options Analysis

Real option study was developed by Myers and Kazakidis [4]. It was used for claim in economics, business, and finance institute, such as Black and Scholes formula [5], for estimation of non-financial or "actual" savings prospect. Model consists of:

1. Find out key records i.e., price figures related to builder's principal structure, and projected traffic for the concession period;
2. Establish a binomial lattice model to distinguish ambiguity of the projected traffic/cost stream;
3. Make random potential future projected traffic including cost stream using Monte Carlo Simulation Technique;

4. Calculate NPV and income analysis under each arbitrary traffic scenario and prepare the developer's cost-effective threat; and
5. Regulate the proposal of projected traffic considering safety of investment assessment and do again if require.

4 Case Study

A hypothetical case study has been considered for a real option study. The project is the construction of 10 km long Bypass with four lanes configuration. The project cost for construction is Rs. 1000 million. It is assumed that the Concessionaire will invest 70:30 (debt: equity) as mentioned in the Concession Agreement generally. After construction, toll revenue will generate and different expenses are toll's administrative charge, annual, periodic and structural overlay, maintenance of road furniture, etc.

Construction period is one year for small 10 km bypass and operation period is 23 years.

Cost stream is shown in Table 1.

It is assumed that interest rate of debt and return on equity is 12 and 15% respectively.

Discount rate adopted = 12 * 0.70 + 15 * 0.30 = 12.6%.

4.1 *Traffic Uncertainty*

Expected annual growth factor is determined using following equation [6]

$$\mathrm{GW} = \left(\frac{\mathrm{AADT}_J}{\mathrm{AADT}_i}\right) \tag{1}$$

Table 1 Different parameters for real option study

Case	Upstream coefficient	Downstream coefficient	Risk neutral probability	Option value (Rs. Million)	Base NPV (Rs. Million)	Base FIRR (%)	Revised NPV (Rs. Million)	FIRR (%)
Base	1.1337	0.88204	0.973	291.1	−17.52	12.5	273.48	17.1
120% of base	1.1625	0.86017	0.883	308.52			290.98	17.3
80% of base	1.1056	0.90445	1.0	164.49			146.97	14.6

The yearly instability of Annual Average Daily Traffic (AADT) to explain ambiguity in projected road user. Yearly instability (or parameter σ) or standard deviation of the anticipated yearly expansion factor of AADT. Growth factors of different traffic are generally considered to measure the risk of underestimate/overestimate the future projected traffic demand over the concession period. The choice of yearly instability of traffic growth is complex phenomena and this shall be studied properly before taking a BOT/PPP Project.

Three ways are suggested to determine σ in BOT projects:

1. Apply Past traffic records of parallel present road to estimate the instability of the proposed project;
2. Apply the forecasted yearly instability of gross domestic product (GDP) of the project road region where the project is to be constructed; and
3. Adopt the opinions to calculate the yearly instability of the new BOT project or Diesel, petrol consumption of roadside fuel pump.

4.2 Binomial Lattice Model

Binomial Model [7] has been used to determine uncertainty of traffic. Since project cost, cash flow directly proportional to traffic. Therefore, same has been used to determine NPV.

UP stream, downstream coefficients, risk neutral probability are calculated using following equations:

$$u = \mathrm{e}^{\sigma\sqrt{\delta t}} \tag{2}$$

$$d = \mathrm{e}^{-\sigma\sqrt{\delta t}} \tag{3}$$

$$P = \frac{\mathrm{e}^{\sigma r\sqrt{\delta t}} - d}{u - d} \tag{4}$$

where,

u, d, and P are upward coefficient, downward coefficient, and risk neutral probability of traffic/cost; and r is risk free interest rate.

Detail calculation has been presented in Annexure 1 for the proposed case study.

4.3 Leakage Rate

The leakage rate can be determined using following equation as mentioned in Eq. 5. The underlying asset value with delaying investment can be adjusted.

$$Li = \frac{\frac{ri-ei}{(1+rd)^i}}{\sum_{i=1}^{i=n} \frac{ri-ei}{(1+rd)^i}} \tag{5}$$

where,

Li leakage rate in year *i* occurring through the exercise of the option to delay;
rj expected annual operating revenue from operating in year *i*;
ei expected annual operating cost, including the annual sales, maintenance and operating cost, and tax service, when the accumulated earnings indicate a surplus in year *i*; and rd = Project discount rate.

Using above equations, real option value has been determined and detail calculation is presented in Annexure 1. Revised NPV value is determined using following equation:

$$\text{Revised NPV} = \text{Conventional NPV} + \text{Option Value} \tag{6}$$

4.4 Option Value Calculation

Base Scenario

There will be variation of traffic for the BOT Project. It is assumed that annual volatility of AADT is $\sigma = 12.6\%$ (It varies from 10 to 20%).

$u = 2.708^{0.126} = 1.1337$,
$d = 2.708^{-0.126} = 0.88204$,
Risk free interest = 12%,
$\Delta t = 1$ year,
$P = (2.708^{0.126} - 0.88204)/(1.1337 - 0.88204) = 0.973$

Scenario (Sensitivity analysis) increasing and decreasing volatility by 20% have been used and all the financial parameters obtained from this present study are presented in Table 1.

4.5 Monte Carlo Simulation Technique

It is found from Table 1 that there is variation of NPV considering variation of risk. Expected NPV can be determined using Monte Carlo Simulation Technique and expected NPV is found to be Rs. 217.63 million.

5 Analysis of Results

NPV has been determined by different methods as proposed in this present study. It is found that conventional method of NPV calculation estimates lowest NPV of Rs. −17.52 Million with financial internal rate of return 12.35% i.e., lower than discount rate of 12.60%. Therefore, project is infeasible. The case study has been evaluated using real option study; it is found that revised NPV Rs. 273.48 million. FIRR has been determined and found to be 17.1%. Sensitivity analysis has been carried out increasing/decreasing volatility by 20%. NPV and FIRR are calculated and presented in Table 1. From Table 1, it is found that NPV varies from Rs. 146.97 to 290.98 million and subsequently FIRR varies from 14.6 to 17.3%.

6 Conclusion

The traditional Discounted Cash Flow (DCF) study may be generally applied for assessment for building a project. A Build Operate and Transfer (BOT) project has ambiguity in an extensive concession time. The monetary practicality determination of this BOT project is supplementary significant for the private sector. Therefore, the traditional methods that studied earlier have been projected to determine the fiscal feasibility of a BOT project are unsuitable for determine the fiscal practicality of the project. Hence, the proposed method recommended in this present research work is more suitable for possibility of the project to adopt the developer.

The proposed method has the several uniqueness: (1) the traditional methodology allowing for only NPV, the proposed model can more broadly estimate the feasibility of a project adopting not only NPV but also the value of future ambiguity throughout the concession period; and (2) the proposed model can determine more truthfully the operating cost for the BOT project. There is always a hindrance the speculation for the BOT project, the in-service income may reduce due to the reduced duration of operating period.

In this study, a real case study has been carried out to verify the usefulness of the proposed method for road sector. The methodology of the three proposed scenarios observed that the modified NPV Method is better method than the NPV proposed by traditional DCF approach. Therefore, NPV/financial analysis using real option study is better method and may be adopted for BOT Project. From the proposed case study and present research work, following conclusions may be drawn:

- Uncertainty of traffic has been taken care and calculation with risk neutral has been considered.
- Proposed method for determination of financial parameters is a better method and recommend of financial viability of a project.
- The proposed method may be used in place of conventional method of NPV/FIRR calculation.

Annexure 1

Year	Initial investment (Rs. Million)	Actual revenue (Rs. Million)	Discounted revenue (Rs. Million)	Project expanses (Rs. Million)	Discounted project expanses (Rs. Million)	Discounted cash flow (Rs. Million)	Cumulative discounted cash flow (Rs. Million)	Leakage rate
2006	1000							
2007		150	133.21	51.84	46.04	87.18	87.18	0.089
2008		167	131.72	75.52	59.56	72.15	159.33	0.073
2009		184	128.89	86.56	60.63	68.25	227.58	0.069
2010		201	125.04	101.12	62.90	62.13	289.71	0.063
2011		218	120.44	117.04	64.66	55.78	345.49	0.057
2012		235	115.30	128.32	62.96	52.34	397.83	0.053
2013		252	109.81	134.72	58.70	51.10	448.94	0.052
2014		269	104.10	139.76	54.08	50.01	498.95	0.051
2015		286	98.29	147.68	50.75	47.54	546.49	0.048
2016		303	92.48	153.84	46.96	45.53	592.02	0.046
2017		320	86.74	161.20	43.70	43.05	635.06	0.044
2018		337	81.13	167.92	40.42	40.70	675.77	0.041
2019	23	354	75.68	175.92	37.61	38.07	713.84	0.039
2020		371	70.44	183.20	34.78	35.66	749.50	0.036
2021		388	65.43	186.72	31.49	33.94	783.44	0.035
2022		405	60.65	189.52	28.38	32.27	815.71	0.033
2023		422	56.13	203.04	27.00	29.12	844.83	0.030
2024		439	51.85	204.08	24.11	27.75	872.58	0.028
2025		456	47.83	218.32	22.90	24.93	897.51	0.025
2026		473	44.06	224.56	20.92	23.14	920.65	0.024
2027		490	40.54	230.24	19.05	21.49	942.15	0.022
2028		507	37.25	237.44	17.45	19.81	961.95	0.020
2029		524	34.19	209.44	13.67	20.53	982.48	0.021
				NPV = −17.52				
				Option value = 291.00				
			Expanded NPV = 273.48					

UP stream and down stream coefficient		0 Year	1 Year	2 Year	3 Year	4 Year	–	21 Year	22 Year	23 Year
UP	D/S	Leakage Factor	0.089	0.073	0.069	0.063	–	0.022	0.020	0.021
1.133740683	0.882036	990	1022.812*	1074.443	1133.516	1203.840	–	5160.012	5732.178	6363.025
			795.735*	835.903	881.861	936.573	–	4014.424	4459.562	4950.353
Calculation				650.322	686.077	728.642	–	3123.171	3469.483	3851.312
990 * 1.133740683 * (1–0.089) = 1022.812					563.105	598.040	–	2563.376	2847.615	3161.005
990 * 0.882036 * (1–0.089) = 795.735						494.134	–	2118.001	2352.855	2611.795
							–	1762.094	1957.484	2172.912
							–	1471.428	1634.587	1814.479
							–	1230.344	1366.770	1517.188
Sigma	12.6						–	1029.964	1144.171	1270.092
U	1.133741		Risk free Interest		12%		–	864.509	960.369	1066.062
D	0.882036						–	727.193	807.828	896.732
							–	613.308	681.315	756.296
		1.126984	0.245				–	518.548	576.047	639.443
			0.252				–	439.654	488.405	542.155
Risk neutral probality		0.973157					–	373.716	415.155	460.845
							–	318.243	353.532	392.439
							–	271.482	301.586	334.776
							–	232.360	258.125	286.532
							–	199.161	221.245	245.594

(continued)

(continued)

UP stream and down stream coefficient		0 Year	1 Year	2 Year	3 Year	4 Year	–	21 Year	22 Year	23 Year
UP	D/S	Leakage Factor	0.089	0.073	0.069	0.063	–	0.022	0.020	0.021
							–	171.209	190.194	211.125
							–	147.455	163.806	181.833
							–	127.216	141.322	156.875
									122.138	135.580
										117.088

Backward Iteration

0 Year	1 Year	2 Year	3 Year	4 Year	–	21 Year	22 Year	23 Year
291.00	330.31	374.94	425.59	483.08	–	4163.01	4725.09**	5363.03*
213.50	242.34	275.09	312.26	354.46	–	3064.06	3479.07	3950.35*
156.68	177.76	201.68	228.81	259.59	–	2215.86	2513.60	2851.31
117.75	133.65	151.69	172.17	195.40	–	1678.30	1904.43	2161.01
86.65	98.42	111.78	126.96	144.19	–	1250.53	1419.73	1611.80
61.56	70.00	79.60	90.51	102.91	–	908.37	1032.215	1172.912
40.945	46.661	53.171	60.585	69.030	–	628.737	715.625	814.479
23.987	27.440	31.387	35.899	41.056	–	396.785	453.028	517.188
10.672	12.287	14.146	16.285	18.747	–	204.109	234.799	270.092
2.260	2.617	3.031	3.510	4.064	–	49.258	57.045	66.062
0.000	0.000	0.000	0.000	0.000	–	0.000	0.000	0.000

Note *Maximum (6363.025 − 1000.0) = 5363.03, 4950.35 − 1000 = 3950.35
** 5363.03 · 0.973 + 3950.35 (1–0.973) · 0.887 = 4725.09

References

1. Zhang X (2004) Improving concessionaire selection protocols in public-private partnered infrastructure projects. J Constr Eng Manage 130(5):670–679. https://doi.org/10.1061/(ASCE)0733-9364
2. Zhang X (2005) Financial viability analysis and capital structure optimization in privatized public infrastructure projects. J Constr Eng Manage 131(6):656–668. https://doi.org/10.1061/(ASCE)0733-9364
3. Transyt (2007) Evaluation of demand risk mitigation in PPP projects. EIB University Research Sponsorship Program, Transport Research Center
4. Mayer Z, Kazakidis V (2007) Decision making in flexible mine production system design using real options. J Constr Eng Manage 133(2):169–180
5. Black F, Scholes M (1973) The pricing of options and corporate liabilities. J Polit Econ 81(3):637–654
6. Luenberger DG (1998) Investment science. Oxford University Press, Inc., New York
7. Hull JC (2008) Options, futures, and other derivatives (7th Edition) Lebanon, Indiana, U.S.A.: Prentice Hall

Impact of Foot Over Bridge (FOB) Upgradation Over Pedestrian Movement: A Case Study of Guwahati City

Arunabha Banerjee and Akhilesh Kumar Maurya

1 Introduction

The different types of road users, both motorized and non-motorized, constitute the global network of the transportation system. Post the new millennium, even though there has been a significant increase in demand for motorization, yet the way to accommodate motorized vehicles on the streets of a developing nation like India has hardly been considered. Post the sudden rise in vehicular traffic, the higher authorities instead of building safer roads for pedestrians, began to curb down pedestrian sidewalks and crosswalks to make room for the high speeding vehicles. This led to pedestrians using the carriageway or crossing illegally and come in direct contact with the motorized vehicles. As per the N.C.R.B report [1] and Mohan et al. [2], the majority of the pedestrian road accidents took place at urban road crossings in metropolitan cities.

To prevent a direct collision, proper segregation in the form of foot over bridges (overpass) or subways (underpass) is the most feasible solution. Past studies [3, 4] showed that pedestrians preferred to use the overpasses in comparison to the underpasses as the underground facilities were prone to illegal activities (especially at night) as well as waterlogging. Ribbens [5] described overpasses as the pedestrian facilities that allowed separation of pedestrians and vehicular traffic, and thus enhanced safety. Mutto et al. [6] found that in Uganda, 77.7% of pedestrians felt that overpasses were a better option in comparison to at-grade facilities; yet insecurity, high stairs, and extra walking distance played a negative role on their psychology. In Turkey, Rasanen et al. [3] observed that depending on the land use type and presence of escalator, the use of FOBs varied between 6 and 63%. Abojaradeh [7] found that in Jordan, FOB's use or nonuse depended on the speed limit of traffic, walkway width, and presence of guardrail on the median. Saha et al. [4] and Pasha et al. [8] observed

A. Banerjee (✉) · A. K. Maurya
Department of Civil Engineering, IIT Guwahati, Guwahati 781039, Assam, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_27

that in Bangladesh, the pedestrians were forced pedestrians to avoid such overpass facilities and cross illegally at-grade due to presence of vendors/standing pedestrians and inadequate security. A study in Pakistan by Malik et al. [9], showed that female pedestrians were demotivated to use the elevated facilities due to improper security and comfort levels. Oviedo-Trespalacious and Scott-Parker [10] also observed that security played a pivotal role in the choice of pedestrians using such facilities. Researchers [3, 11, 12] felt that the installation of proper elevator/escalator and security improvement would encourage pedestrians to use such facilities.

In India, unplanned foot over bridges (FOBs) are constructed without detailed feasibility studies, and this leads to the pedestrians either being forced to use the facilities or using other illegal at-grade crossings instead. Also, the absence of escalators/elevators, presence of vendors/standing persons on the walkway, and absence of proper security (CCTVs and security personnel) discourage pedestrians to use elevated facilities. The current study tries to highlight the importance of improving the existing non-mechanized FOB facility and providing better-mechanized FOB with adequate available width, security, and lifts for pedestrians to use.

2 Data Collection and Analysis

To understand the pedestrian behavior over elevated pedestrian facilities, data was collected over a non-mechanized FOB (NMFOB) which was later converted into a mechanized FOB (MFOB), using both videography technique as well as questionnaire survey.

2.1 Data Collections Sites

Maligaon is a busy location situated in the heart of Guwahati city and is of mixed land use type (as offices, schools, railway station, shopping markets, and residences are located in the near vicinity). The data was collected over two separate weekdays in the months of October 2016 (for NMFOB) and July 2018 (for MFOB). Videography, as well as questionnaire survey data, were collected for both the non-mechanized FOB (without escalators, elevators, CCTVs, shade, and maintenance) and the mechanized FOB (with elevator, shade, CCTVs, security personnel, proper maintenance, and cleanliness). Figure 1 shows the location of the footbridge in Maligaon (Guwahati).

During videography data collection, the camera was installed on a tripod stand at the height of 12 feet above the ground. A trap length of 10 m was used for both the FOBs. The data were collected between 9 am to 12 pm. During the data collection of NMFOB, no at-grade crossings were available; while in the case of MFOB an illegal median opening was available at a distance of 100 m from the FOB. Figure 2a, b show the NMFOB and MFOB locations.

Fig. 1 Location of Maligaon FOB (*Source* google.com)

Fig. 2 **a** Non-mechanized FOB and **b** Mechanized FOB at Maligaon

The geometric features of the two FOBs are shown in Table 1. It is observed that apart from the installation of elevators (available only to elderly, female, child, and differently-abled pedestrians), the width over stairs and mid-block sections were increased for MFOB. Also, the riser and tread dimensions were modified.

Along with videography data collection, questionnaire survey was also conducted for both the FOBs. The respondents were randomly selected and the ones willing to undergo the entire survey process were surveyed. The participation rate was also

Table 1 Geometric details of FOBs

Facility (FOB)	Length of mid-block (in m)	Effective with of mid-block (in m)	Effective width of stairways (m)	No. of stairways	Height of riser (in cm)	Depth of tread (in cm)	Entry-exit points	Vertical connectivity available
Non-mechanized (NMFOB)	39	1.0	0.9	65	8	30	2	Stairs
Mechanized (MFOB)	39	1.6	1.2	65	13	26	4	Stairs, elevators

considerably low (1 in 20 pedestrians) due to the peak time interval during which the data was collected. In total 81 pedestrians (39 from NMFOB and 42 from MFOB) participated in the survey. A five-point Likert scale (very poor, poor, satisfactory, good, and very good) was used to understand the perception of the pedestrians regarding the current existing situation based on seven perception parameters. The parameters used were available walkway width (over mid-block and stairways), available surface condition (whether broken or slippery), available end connectivity (i.e., entry/exit points and type of vertical connectivity), available safety-security measures (presence/absence of security personnel/CCTV camera), available comfort (presence and maintenance of lighting, shade and guardrail), available walk environment (cleanliness and overall maintenance) and available obstruction (presence/absence of hawkers/vendors/standing pedestrians). Table 2 shows the questions which were put up before the respondents. The three primary sections in the questionnaire survey were demographics of the respondents, current existing features and usability preference (Yes/No). The final section of the sheet was related to the field observations which were noted down by the surveyors.

Table 2 Design of questionnaire

Title	Description
Demographics	Respondents' gender (*Male/Female*), Respondents' age category (<12/13–22/23–45/46–59/ > 60), Respondents' luggage condition (*With/Without*), Respondents' profession (*Student/Self-employed/Service/Business/Retired/Homemaker/Others*), Respondents' purpose of trip (*Education/Work/Mode change/Returning home/Shopping/Jaywalking/Others*), Respondents' frequency of using FOB daily (*First time/Occasionally/Once/Twice/More than twice*)
Current features	*On a rating scale of five* (*Very Poor-Poor-Satisfactory-Good-Very Good*): Available walking width (over mid-block and stairway), Available surface condition (over entire FOB), Available connectivity (horizontal end connectivity and vertical end connectivity), Available safety-security measures, Available comfort (over entire FOB), Available walk environment (over entire FOB) *On a rating scale of three* (*Many-Some-None*)*:* Available obstruction (over entire FOB)
Usability preference	Preference to use FOB more than at-grade crossing (0: *No*, 1: *Yes*)
Characteristics of FOB	Travel length (in m), Width of mid-block (in m), Number of steps, Width of stairway (in m), Tread and riser dimension (in cm), Facility available for vertical connectivity (*Stairway/Escalator/Lift/Ramp*), Available entry-exit points

2.2 Extraction and Analysis of Data

The video data was extracted using a manual extraction technique to get more insight on the impact of age, gender, presence of luggage, use of mobile and group size on the average walking speed. Apart from these factors, average speed, flow rate, density, and space were also calculated for both FOB. Figure 3 shows the speed variation for different pedestrian categories for both NMFOB and MFOB.

Figure 3 indicates that the walking speed over the NMFOB was higher than the MFOB for all the different pedestrian categories. In the case of NMFOB, the average speed was higher due to the narrow available width, which forced the pedestrians to continually keep on moving with the stream speed to avoid any congestion. However, in the case of the renovated and widened MFOB, the pedestrians had the freedom to choose their own speed and not forced to move with the group speed.

Also, from Fig. 3, it is evident that pedestrians in the age category of 23–45 years walked at comparatively higher speeds (by 2–18 m/min) that the younger or the older ones. Moreover, the male pedestrians walked with higher speeds (4–6 m/min) in comparison to females. The group size played a considerable impact on the average speed, as with an increase in group size the average walking speed significantly reduced.

Tables 3 and 4 show the t-test and ANOVA test results for the different pedestrian categories. These tests were conducted in order to check whether there was any significant difference between various pedestrian groups based on gender, luggage, mobile use, age, and group size. A t-test was performed in the case of pedestrian gender group, luggage group, and mobile user group. The t-test is a statistical hypothesis

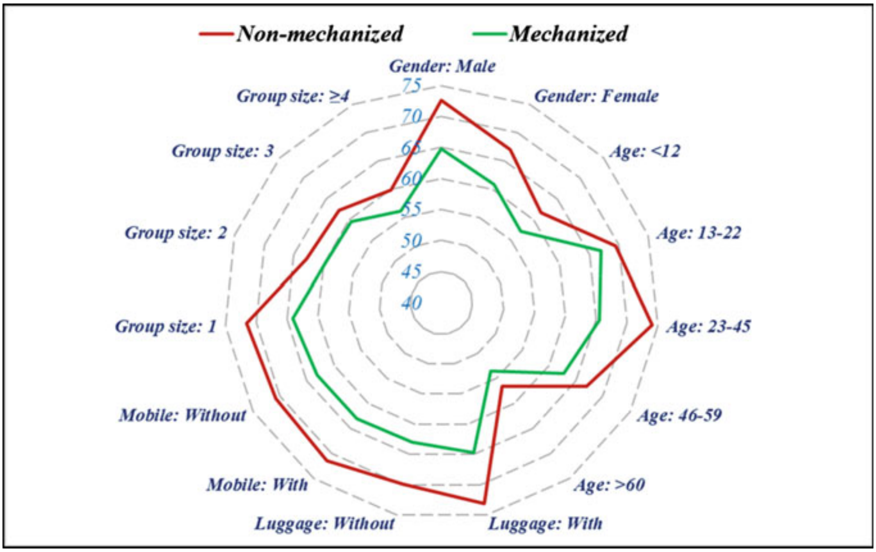


Fig. 3 Variation of speed for different pedestrian categories over NMFOB and MFOB

Table 3 t-test for pedestrian gender, luggage, and mobile use groups

FOB	Type of pedestrian group	t-statistical value	t-critical value	Difference
NMFOB	Gender	7.69	1.96	Significant
	Luggage	4.17	1.96	Significant
	Mobile use	0.21	1.96	Not-significant
MFOB	Gender	5.59	1.96	Significant
	Luggage	2.09	1.96	Significant
	Mobile use	0.06	1.96	Not-significant

Table 4 ANOVA test for pedestrian age groups and different group sizes

FOB	Type of pedestrian group	*F*-statistical value	*F*-critical value	Difference	*P*-value
NMFOB	Age	45.21	2.37	Significant	0
	Group size	21.3	3.00	Significant	0
MFOB	Age	34.51	2.37	Significant	0
	Group size	11.62	2.61	Significant	0

test that is performed to determine whether two sets of data are significantly different from each other or not, and is generally applied when the test statistics follow a normal distribution. Similarly, the ANOVA test was performed for comparing statistical significance for three or more groups (viz. age and group size), and it generalizes the t-test to more than two groups.

It could be seen from Table 3 that a significant difference (t-statistical value > t-critical value) existed for gender and luggage groups. The t-test result showed that mobile use did not impact the overall walking speeds significantly over the two FOBs.

The result of the ANOVA test (refer Table 4) showed that F-statistical value was significant difference existed between different age categories and group sizes.

Table 5 shows the variation of different macroscopic parameters for the two different types of FOBs. The overall mean speed was ~ 7 m/min higher in the case of NMFOB. Moreover, the flow rate and density were lower in the case of MFOB, and such an observation was due to the increased effective width. Further, in the case of MFOB, as the illegal opening was present in close vicinity, the young and middle-aged pedestrians used the at-grade facility instead of using the FOB. This led to lower pedestrian volume over MFOB in comparison to NMFOB.

Table 5 Variation of pedestrian flow characteristics over NMFOB and MFOB

FOB type	Speed (m/min)	Flow rate (ped/min/m)	Density (ped/m^2)	Area module (m^2/ped)
NMFOB	70.75	37.86	0.75	1.52
MFOB	63.44	15.27	0.28	4.24

The questionnaire data was entered into the excel sheet as per the final requirements. IBM SPSS Statistics 20 was used to analyze the data. The proportion of respondents based on age, gender, profession, purpose of the trip and daily frequency of using facility are shown in Figs. 4, 5, 6, 7 and 8.

Figures 4 and 5 show that the majority of the respondents were male pedestrians aged between 23 and 59 years, over both the FOBs.

Figures 6 and 7 show that the highest proportion of pedestrians using the facilities were self-employed, servicemen, or students, who were using the facilities majorly for their work trip, education trip, or mode change.

Fig. 4 Gender specific proportion

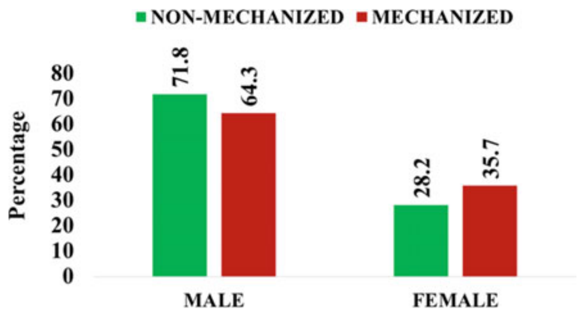


Fig. 5 Age specific proportion

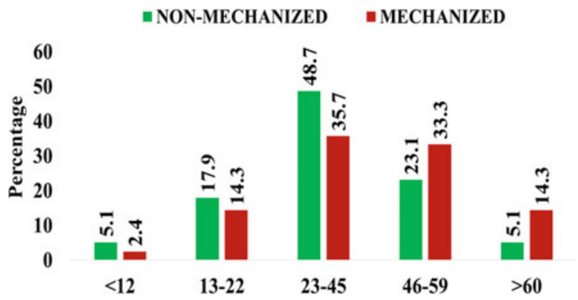
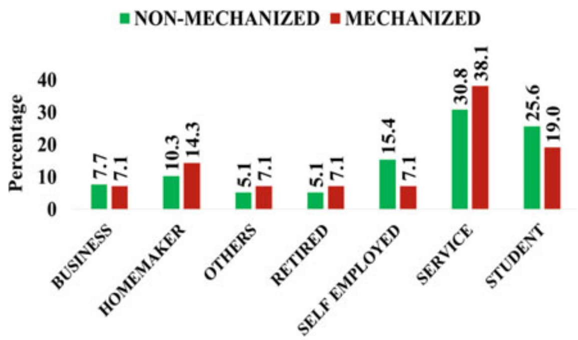


Fig. 6 Profession wise proportion



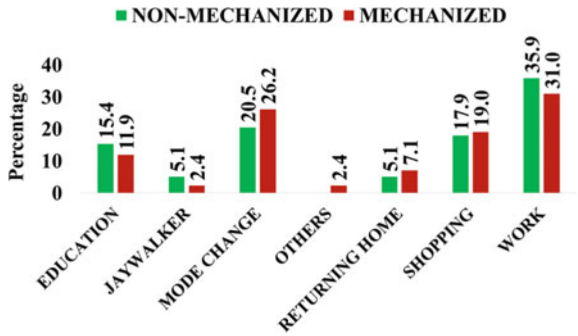


Fig. 7 Trip purpose wise proportion

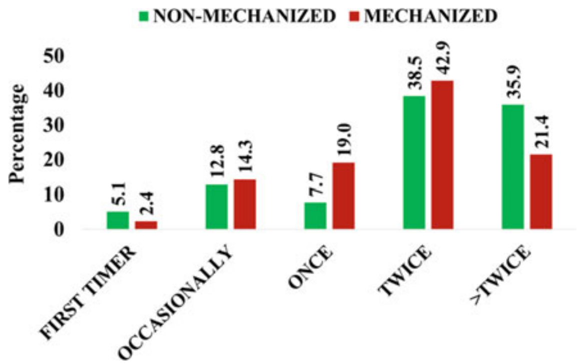


Fig. 8 Daily use-specific proportion

Also, as per Fig. 8, it could be concluded that the highest proportion of pedestrians who responded used the facility twice or more daily, and were highly aware about the current existing condition of the facilities.

A Cronbach's alpha test was performed to check the internal reliability of the Likert scale, and results showed that Cronbach's alpha value was 0.766, and it projected a high level of internal consistency between the seven parameters (available walkway width, available surface condition, available vertical/horizontal connectivity, available safety-security measures, available comfort, available walk environment, and presence/absence of obstruction). "The Cronbach's alpha if item deleted" test was also performed, and results showed that removing any factor resulted in a lower value of Cronbach's alpha (i.e., less than 0.766).

2.3 Model Development

The dependent and independent variables used for predicting the preference of pedestrians using the FOBs under two different circumstances are mentioned in Table 6.

Table 6 Dependent and independent variables used in model development

Dependent variables (Yi)	Independent/predictor variables (Xi)
Preference for using FOB	Respondents' gender (Gen), Respondents' age category (Age), Respondents' luggage condition (Lug), Respondents' profession (Prof), Respondents' purpose of trip (TP), Respondents' daily frequency of facility use (Freq), Available walkable width (Wid), Available surface condition (Sur), Presence/absence of obstruction (Obs), Available horizontal/vertical connectivity (Con), Available Safety-Security measures (SS), Available comfort for travel (Com), Available walk environment (WEnv)

The aim was to develop two separate models for NMFOB and MFOB, to predict the impact of most significant factors.

A Pearson correlation test was conducted to understand the correlation between different factors for non-mechanized and mechanized FOBs. The results showed that the gender variable had a negative correlation with luggage, width, obstruction, safety-security, and comfort (at 5% significance level). This implied that female pedestrians were profoundly affected by safety-security and comfort, due to the presence of obstacles which reduced the accessibility width. Similarly, with age, frequency of travel and connectivity were observed to have a negative correlation. This signifies that the number of times pedestrians used the facility daily and connectivity (vertical as well as horizontal) affected their choice of using the facility.

The forward stepwise logistic regression method was used to predict the preference of pedestrians toward using non-mechanized and mechanized FOBs. The regression model predicting the odds of making one or the other decision is represented by Eq. 1.

$$\ln(\text{ODDS}) = \beta + \beta 1X1 + \beta 2X2 + 1 \ldots \beta nXn \tag{1}$$

where β represents the coefficients, X_1 to X_n represents the parameters influencing the model.

A Hosmer–Lemeshow test was conducted to predict the accuracy of the goodness of fit. The HL test shows the chi-square and p-values. In the case of both the models for non-mechanized and mechanized FOBs, the chi-square and p-values were high (>0.05), which indicated that the models fitted well.

The best models for NMFOB and MFOB were obtained in the 6th and 4th steps, respectively, using forward binary logistic regression. Equations 2 and 3 show the best models used for predicting the preferability of the pedestrians toward using the non-mechanized and mechanized FOBs.

$$\begin{aligned}\ln(\text{ ODDS})\ \text{NMFOB} &= -1.483 + 0.939 * \text{Wid} + 0.345 * \text{Con} + 0.843 * \text{Sur} \\ &\quad + 0.762 * \text{SS} + 0.239 * \text{Gen} + 0.332 * \text{Age}\end{aligned} \tag{2}$$

$$\ln(\text{ ODDS})\ \text{MFOB} = -1.023 + 0.731 * \text{Con} + 0.361 * \text{Wid} + 0.442 * \text{Surf} + 0.277 * \text{Age} \quad (3)$$

The final model for NMFOB (Eq. 2) predicted that width, connectivity, surface, safety-security, gender, and age all had positive coefficients. Width had the highest impact on the preference of using the NMFOB, and thus improving the width over the mid-block and stairways would enhance future usability. Factors such as surface (0.843) and safety-security (0.762) were highly important toward the inclination of pedestrians to use such facilities. Connectivity, age, and gender also had a significant impact on the perception of pedestrians.

The final model related to MFOB (Eq. 3) showed that connectivity was still quite poor, and especially vertical connectivity (in the form of lifts). Improving the vertical connectivity by providing escalators would provide a comfortable movement for pedestrians of different age and gender categories. Also, improving the width and surface of the stairways would encourage more pedestrians to use such facilities. Age was observed to have a positive coefficient as with the increase in age pedestrians were reluctant to climb the stairs.

3 Conclusion and Application

The objective of the current study was to identify the impact of different factors on pedestrian movement behavior and perception. Both videography and questionnaire techniques were used to study the effect of potential factors (gender, age, luggage, mobile use, and group size) on the pedestrian walking speed and also identify the most relevant factors which impacted their perception. The data was collected from a non-mechanized FOB (NMFOB) at Maligaon (Guwahati), which was later reconstructed into a mechanized FOB (MFOB).

The major conclusions from the present study are:

- The pedestrian walking speed over the NMFOB was significantly higher than the MFOB for all the pedestrian categories. The main reason for such an observation was the narrow width as the pedestrians were forced to move with the stream speed to prevent congestion over the facility. In such a case, pedestrians of different age and gender categories were unable to choose their favored walking speed. On the contrary, the NMFOB was widened and pedestrians had the freedom to choose their preferred walking speeds and not be forced to move with the group speed.
- The male pedestrians walked at 4–6 m/min higher walking speed than their female counterparts for both the FOBs. Similarly, the pedestrians in the age group 23–45 years walked with comparatively higher speeds than the other groups.
- The results obtained from the t-test (refer to Table 3) showed that there was no significant difference when pedestrians were using mobile phones or not.

However, there was a significant difference in walking speeds based on gender and luggage.

- From ANOVA test results (refer to Table 4), a significant difference was observed for different age categories and group sizes.
- The flow rate and density were higher in NMFOB due to lower available effective width.
- The results of the questionnaire survey showed that the majority of the pedestrians using such facilities were young (13–45 years) male pedestrians who used the facilities twice or more daily. Moreover self-employed, servicemen, or student pedestrians majorly used the facilities for their work/educational/mode change trips.
- The forward stepwise logistic regression method predicted that before reconstruction, width, surface condition, and safety-security were highly influencing the perception of the pedestrians. Similarly, post-reconstruction, connectivity was the most important which influenced the choice of pedestrians in using such facilities.

The result of the study shows that in order to encourage or attract more pedestrians in using such facilities, it is extremely important to provide proper vertical connectivity (in form of escalators) for all pedestrian categories, improve the width and surface conditions, and also provide appropriate safety-security measures (in form of security personnel deployment, or CCTV camera installation or both).

The current study would provide insight to engineers and planners to consider the most significant factors before reconstructing older FOBs or constructing newer ones.

Acknowledgements This research work was funded by CSIR—Central Road Research Institute (CRRI), New Delhi under the Supra Institutional Network Project (SINP) category as part of the 12th Five Year Plan Period (FYP).

References

1. N.C.R.B. Report (2015) Accidental deaths and suicides in India
2. Mohan D, Tiwari G, Bhalla K (2016) Road safety in India: status report
3. Räsänen M, Lajunen T, Alticafarbay F, Aydin C (2007) Pedestrian self-reports of factors influencing the use of pedestrian bridges. Accid Anal Preven 39:969–973
4. Saha MK, Tishi TR, Islam MS, Mitra SK (2013) Pedestrian behavioral pattern and preferences in different road crossing systems of Dhaka city. J Bangladesh Inst Plann 6:149–160
5. Ribbens H (1996) Pedestrian facilities in South Africa: research and practice. Transp Res Rec 1538:10–18
6. Mutto M, Kobusingye OC, Lett RR (2002) The effect of an overpass on pedestrian injuries on a major highway in Kampala – Uganda. Afr Health Sci 2:89–93
7. Abojaradeh M (2013) Evaluation of pedestrian bridges and pedestrian safety in Jordan. Civil Environ Res 3(1):66–79
8. Pasha M, Rifaat SM, Hasnat A, Rahman I (2015) Pedestrian's behaviour on road crossing facilities. J Teknol 73(4)

9. Malik IA, Alwi SKK, Naiga G (2017) A survey to understand people perception of pedestrian bridges usage on Shahrah-e-Faisal Road, Karachi-Pakistan. New Horizons, Karachi, Pakistan
10. Oviedo-Trespalacious O, Scott-Parker B (2017) Footbridge usage in high-traffic flow highways: the intersection of safety and security in pedestrian decision-making. Transp Res F: Traff Psychol Behav 49:177–187
11. Sabet D (2013) Fear is not enough: testing the impact of risk on pedestrian behavior in Dhaka, Bangladesh, CES Working Paper, pp 1–27
12. Hasan R, Madzlan N (2018) The perception of Malaysian pedestrians toward the use of footbridges. Traffic Inj Prev 19(3):292–297

Effect of Two-Wheeler Proportion on Passenger Car Unit at Urban Signalised Intersections

Sandeep Singh and S. Moses Santhakumar

1 Introduction

Traffic is diverse in India, where the vehicles vary in size, speed, and time for clearing the intersections. Two-wheelers (TWs) are the most common mode of transportation in cities, and they have a considerable effect on traffic flow at signalised intersections [1]. TWs' presence also affect the Passenger Car Unit (PCU) value [2]. The dual heterogeneity in India, i.e. heterogeneity in vehicle types and driver behaviour, limits the use of constant PCU values recommended by the Indian Roads Congress (IRC-93:1985) [3] guidelines. The PCU values vary as the speeds of vehicles clearing the intersection vary with traffic and roadway conditions [4]. Signalised intersections are regarded as a hindrance to continuous traffic flow, despite the fact that they allow for a reduction in the number of conflicts point at approaches. The effect of TWs' should be considered when quantifying PCU values under heterogeneous conditions [5]. The dynamism of PCU values must be assessed because the development of PCU standards for the vehicles is deemed necessary. With this motivation, this study shall evaluate the effects of the proportion of TWs' on other vehicle PCU under mixed traffic.

2 Literature Review

Hossain [6] assessed the traffic and operational characteristics of signalised intersections in the capital city Bangladesh. According to the findings of the study, there

Present Address:
S. Singh (✉) · S. M. Santhakumar
Department of Civil Engineering, National Institute of Technology Tiruchirappalli, Tiruchirappalli, Tamil Nadu, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_28

is a need for a robust methodology to better understand saturation flow rates. Al-Kaisy et al. [7] quantified the impact of heavy vehicles using queue-discharge flow rate. Minh and Sano [8] studied the effect of motorcycles on saturation flow at signalised intersections in Eastern nations. They concluded that motorcycles significantly impact traffic and should be considered in the geometric design and operation of signalised intersections. However, all of these studies and their findings are unworkable for Indian conditions because they were conducted under homogeneous traffic conditions.

In the Indian context, Arasan and Jagadeesh [9] estimated saturation flow at signalised intersections in the existence of mixed traffic. They compared the headway method to the probabilistic method. The researchers noted that the probabilistic method was more precise for the Indian traffic, which carries a variety of vehicles. Chandra and Kumar [10] estimated the dynamic passenger car unit (DPCU) value considering that "the PCU value is directly proportional to the ratio of clearing speed ratio of the standard car and subject vehicle and inversely proportional to the area occupancy ratio of the standard car and subject vehicle". Arasan and Vedagiri [11] built HETEROSIM-based simulation models and estimated the saturation flow under heterogeneous conditions. They investigated the impact of the width of the road on saturation flow. Patil et al. [12] proposed new PCE values for various vehicles using regression techniques to analyse the capacity of signalised intersections.

Arasan and Arkatkar [13] developed a microscopic simulation technique to investigate the change in PCU values of various vehicles prevalent in India by taking into account differences in the volume and roadway. The study's findings show that for vehicles larger than passenger cars, the PCU values decreased as traffic volume increased and vice versa. Asaithambi et al. [14] examined the role of mixed traffic, with a particular emphasis on large volumes of motorised TWs. They developed microscopic traffic simulation models. According to the findings of this paper, the effect of traffic composition on capacity is greater when there is a higher proportion of TWs. Sonu et al. [2] studied the time occupancy concept and estimated the PCU values. Mohan and Chandra [15] estimated the PCE at the signalised intersection using the queue clearance rate method.

Nonetheless, hardly, any research has been conducted that explicitly developed an appropriate PCU model, making it difficult to assess the performance parameters. Furthermore, outlining the review of previous literature, it was evident that there is a lack of research in estimating the PCU using ratio methods. The current study thus focuses on examining the effect of the different proportions of TWs' on PCU values of vehicles at a signalised intersection using a ratio method.

3 Study Methodology

The present research work is primarily concerned with analyses of the effect of the proportion of TWs on PCU values. For this purpose, traffic data are acquired using the videotaping method at a signalised intersection where TWs proportion is higher.

These recorded traffic videos are then imported into the traffic data extractor software for finding the individual vehicles' intersection clearance time to estimate the PCU values. Other traffic-related parameters like the saturation flow, capacity, and v/c ratio were also computed. Finally, VISSIM developed models are used to evaluate the effect of the proportion of TWs on the vehicle PCU. The TWs proportion was varied at 5% as 55, 60, 65, 70, and 75% relative to the standard car.

4 Data Collection

A highly congested signalised intersection, with higher proportions of TWs and the other vehicles, is chosen as the study site. In this research work, a signalised intersection at the Head Post Office (HPO) in Tiruchirappalli, India, is selected. A typical weekday is chosen to collect the traffic for three hours between 04:00 PM and 07:00 PM. Figure 1 illustrates the selected test site.

The road width varied at each approach. The geometric details of the approaches in the North Bound (NB), East Bound (EB), South Bound (SB), and West Bound (WB) directions are shown in Table 1.

The NB and EB approaches are identified to be present with an exclusive lane for a free left turn. Also, the EB approach was identified to be present with a curbside bus stop within 75 m from the stop of intersection for which a suitable bus blockage adjustment factor (f_{bb}) is estimated and applied whilst calculating the saturation flow rate and capacity, which is discussed in the following sections.

Figure 2 represents the directional volume (v_i) of all the vehicles for all four approaches. It can be inferred that most of the vehicles had been taking the straight

Fig. 1 Selected study site Head Post Office Junction

Table 1 Geometric characteristics of the Head Post Office Junction

Geometric characteristic	Approach			
	NB	EB	SB	WB
Approach width, *w* (m)	8.10	13.0	10.5	11.5
Number of lanes, *n* (#)	2	3	2	2
Presence/absence of exclusive lane for a free left turn	Present	Present	Absent	Absent
Presence/absence of curbside bus stop within 75 m from the stop of intersection	Absent	Present	Absent	Absent

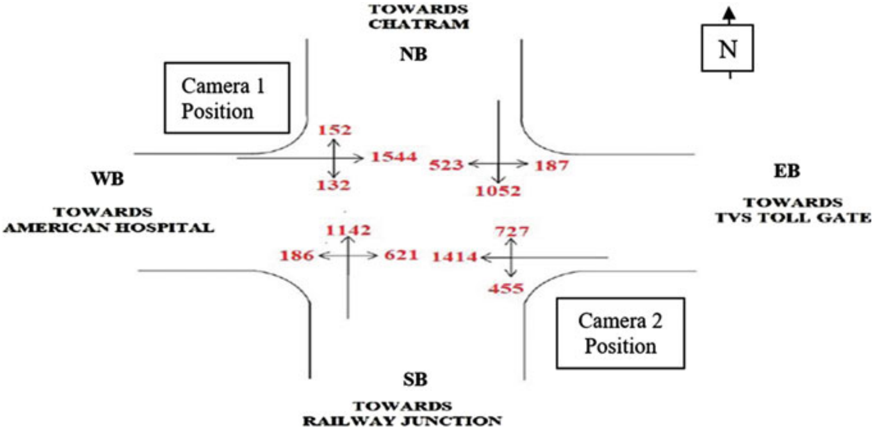


Fig. 2 Approach traffic volume

movement followed by right turning and left-turning movements during the study period. The NB approach and SB approach carried 1762 vehicles and 1949 vehicles, respectively, whilst the EB approach and WB approach carried 2596 vehicles and 1828 vehicles.

From Fig. 2, it can be inferred that the traffic flow from the EB and WB direction was more than the traffic flow from the NB and SB direction. The composition of traffic is also important in the determination of traffic characteristics [16, 17]. It was also noticed that amongst all the four approaches, the TWs carried the highest share. The recorded traffic composition of the eight different vehicle types is presented in Table 2.

The traffic control characteristics measures revealed that the overall cycle length of the HPO junction to be 164 s. The green time, amber time, and red time were also recorded from the signalised intersection with the help of a stopwatch. Table 3 shows the traffic control characteristics.

Table 2 Traffic composition at HPO signalised intersection

Vehicle type	Composition (%)			
	NB	EB	SB	WB
Two-wheeler (TW)	61	68	67	69
Three-wheeler (ThW)	5	14.9	9	11
Small car (SC)	18	8	11	9
Big car (BC)	6	2	8	4
Light commercial vehicle (LCV)	4	2	2	2
Bus (BUS)	4	4	1	3
Heavy commercial vehicle (HCV)	1	1	1	1
Bicycle (BY)	1	0.5	1	1

Table 3 Traffic control characteristics of HPO signalised intersection

Parameter (s)	Approach			
	NB	EB	SB	WB
Green time	30	46	30	46
Amber time	3	3	3	3
Red time	131	115	131	115

5 Data Analysis and Results

5.1 PCU Estimation

In the present study, the PCU value is estimated as a measure of time to clear the intersection entirely in a given direction of the movement group from one approach to another. "The intersection clearance time is defined as the time taken by a vehicle to clear the approach separately for left-movement, straight movement, or right-turn movement" [4]. The time difference between the entry and exit timestamp was taken for the intersection clearance time calculation and to determine the vehicle PCUs. The vehicle PCU is calculated using Eq. (1).

$$\mathrm{PCU}_i = I_i/I_{sc}/A_{sc}/A_i \tag{1}$$

where,

PCU_i PCU of the *i*th vehicle
I_i Mean intersection clearing time of the *i*th vehicle(s)
I_{sc} Mean intersection clearing time of SC(s)
A_{sc} Area of SC
A_i Area of the *i*th vehicle.

Tables 4, 5, and 6 show the PCU values for left turning, straight movement, and right-turning vehicles, respectively.

The PCU values for each turning movement were observed to be slightly different from each other. Hence, the average PCUs were computed and adopted for further

Table 4 Estimation of PCU values for left-turning vehicles

Vehicle type	I_i (s)	A_i (m^2)	I_i/I_{sc}	A_{sc}/A_i	PCU
TW	3.7	1.2	0.7	4.5	0.2
ThW	3.9	4.5	0.8	1.2	0.5
SC	4.8	5.4	1.0	1.0	1.0
BC	5.0	8.1	1.0	0.7	1.4
LCV	5.4	9.5	1.1	0.6	1.8
BUS	6.5	25.8	1.4	0.2	6.9
HCV	6.9	18.0	1.5	0.3	4.9
BY	9.0	0.9	1.9	6.2	0.3

Table 5 Estimation of PCU values for straight movement vehicles

Vehicle type	I_i (s)	A_i (m^2)	I_i/I_{sc}	A_{sc}/A_i	PCU
TW	5.8	1.2	0.9	4.5	0.2
ThW	6.8	4.5	1.1	1.2	0.9
SC	6.1	5.4	1.0	1.0	1.0
BC	6.5	8.1	1.2	0.7	1.7
LCV	5.5	9.5	0.9	0.6	1.5
BUS	8.0	25.7	1.3	0.2	6.5
HCV	8.1	18.0	1.3	0.3	4.4
BY	11.0	0.9	1.8	6.2	0.3

Table 6 Estimation of PCU values for right-turning vehicles

Vehicle type	I_i (s)	A_i (m^2)	I_i/I_{sc}	A_{sc}/A_i	PCU
TW	6.4	1.2	0.9	4.5	0.2
ThW	5.5	4.5	0.8	1.2	0.7
SC	6.9	5.4	1.0	1.0	1.0
BC	7.4	8.1	1.1	0.7	1.6
LCV	7.8	9.5	1.1	0.6	1.8
BUS	8.0	25.8	1.2	0.2	6.0
HCV	7.1	18.0	1.0	0.3	3.4
BY	13.5	0.9	1.9	6.2	0.3

analysis. The average PCU value for TW, ThW, SC, BC, LCV, HCV, BUS, HCV, BY is 0.2, 0.7, 1.0, 1.6, 1.7, 6.5, 4.2, and 0.3, respectively.

5.2 Saturation Flow and Unit Base Saturation Flow Rate Determination

Every signalised intersection at a certain point will attain its saturation flow rate during the peak hour traffic flow. The saturation flow ($\mathrm{SF_i}$) is measured in PCU/hour of the green period [18] and is given by Eq. (2).

$$\mathrm{SF}_i = w * \mathrm{USF}_i * f_{\mathrm{bb}} * f_{\mathrm{br}} * f_{\mathrm{is}} \tag{2}$$

where

w	Width of the approach (m)
USF_o	Unit base saturation flow rate
f_{bb}	Bus blockage adjustment factor for curbside bus stop
f_{br}	Vehicle blockage adjustment factor for the through vehicles waiting for right-turning vehicles turn, $f_{\mathrm{br}} = 1$, if no blockage
f_{is}	Initial surge of vehicles adjustment factor for approach flare and anticipation effect, $f_{\mathrm{is}} = 1$, if no surge flow.

Saturation flow rate under the stated base conditions of intersection relating to traffic, geometric, and control conditions of green is called base saturation flow rate (USF_o). The base saturation flow varies with respect to the approach width as given in Eq. (3) as per the recommendations of Indo-HCM (2017) [18].

$$\begin{array}{ll} \mathrm{USF}_o = 630; & \text{for} w < 7.0\,\mathrm{m} \\ 1140 - 60w; & \text{for} w < 7.0\,\mathrm{m} \\ 500; & \text{for} w > 10.5\,\mathrm{m} \end{array} \tag{3}$$

The $\mathrm{USF_o}$ values estimated using Eq. (3) for NB (8.10 m), EB (13.0 m), SB (10.5 m), and WB (11.5 m) are 654, 500, 510, and 500 PCU/h, respectively.

5.3 Bus Blockage Adjustment

The EB approach was affected by the bus blockage because of a curbside bus stop, which hindered the normal traffic flow. So, f_{bb} was estimated as suggested in the Indo-HCM (2017) method [18]. It is given by Eq. (4).

$$f_{\mathrm{bb}} = \frac{w - 3 * (t_{\mathrm{b}} * n_{\mathrm{b}}/3600)}{w} \tag{4}$$

where,

t_b average blockage time during green (s)
n_b number of buses stopping per hour (buses/h).

The approach width (w) was 13 m, whilst the t_b was 10 s and n_b was 30 buses/h. The f_{bb} was estimated to be 0.98. Hence, the SF_{EB} with the $USF_o = 500$ (for $w >$ 10.5 m) was estimated to be 6370 PCU/h. Similarly, the SF_i for other movement groups are estimated as follows: $SF_{NB} = 5298$ PCU/h, $SF_{SB} = 5355$ PCU/h, and $SF_{WB} = 5750$ PCU/h. These are used for the estimation of the capacity of each movement group.

5.4 Capacity and V/C Ratio Determination

Capacity (C_i) of the movement group is found in this study using Eq. (5), and the effective green time of the movement group is estimated using Eq. (6), which is helpful for the estimation of the v_i/C_i ratio at the HPO signalised intersection.

$$C_i = SF_i * \frac{g_i}{C_o} \tag{5}$$

where,

C_i Capacity of movement group 'i' (PCU/h)
SF_i Saturation flow rate (PCU/h)
C_o Cycle time (s)
g_i Effective green time of the group 'i' (s).

$$g_i = \frac{y_i}{Y} * (C_o - L) \tag{6}$$

where,

Y $\sum (y_i)$
y_i Flow ratio $= v_i/SF_i$
L Lost time $= \sum [I - A] + \sum [l]$ (s)
I Inter-green time (s)
A Amber time (s)
l Starting delay per vehicular phase (s/phase).

The recorded cycle time was 164 s. Using the inter-green time (6 s), amber time (3 s), and starting delay per vehicular phase (1.5 s/phase), the lost time was estimated to be 18 s.

Table 7 Estimated flow ratios, effective green time, and capacity of each movement group

Parameter	Approach			
	NB	EB	SB	WB
Flow ratio (y_i)	0.33	0.41	0.36	0.32
Effective green time (g_i) (s)	34	41	38	33
Capacity (C_i) (PCU/h)	1098	1593	1241	1157
$v_{i/}C_i$ ratio	1.60	1.63	1.57	1.58

Degree of saturation (X_i) measures how much demand (volume) an approach is experiencing compared to its capacity. It is the ratio of approach volume to approach capacity. X_i or volume to capacity ratio (v_i/C_i) of a movement group of each approach was measured. Table 7 shows the flow ratios, effective green time, capacity, and $v_{i/}C_i$ ratio of each movement group.

The maximum effective green time amongst the movement groups is for EB, which is 41 s, which also carries the maximum capacity amongst the other movement groups, which is found out to be 1593 PCU/h. In addition, the EB approach was found to have the highest $v_{i/}C_i$ ratio amongst the other approaches.

5.5 *Simulation Modelling*

For the analysis of the effect of the TWs on the PCU of the vehicles, the microscopic simulation models were developed using VISSIM. The traffic simulation model replicating the real-field conditions is shown in Fig. 3.

Based on the above study mentioned methodology, the PCU values at an incremental rate of TWs were estimated and are shown in Table 8.

6 Conclusion

After simulating the microscopic model, the analysis revealed that the different proportions of TWs significantly affected the vehicle PCUs, as shown in Table 8. These findings are in line with the previous studies, which have also assessed the impact of different traffic conditions on traffic characteristics under mixed traffic [19–23]. The results show that TWs and ThWs PCU reduce consistently with the increase in the TWs' composition from 55, 60, 65, 70, to 75%. In contrast, the PCU of vehicles like BC, LCV, BUS, HCV, and BY were observed to increase with the increase in the TWs' composition. This may be attributed to their relatively more time clearing the intersection and bigger physical area except that for BY, which has a small physical area. Hence, the varying and incremental proportion of TWs significantly influences the vehicle characteristics. The TWs act as an impedance to

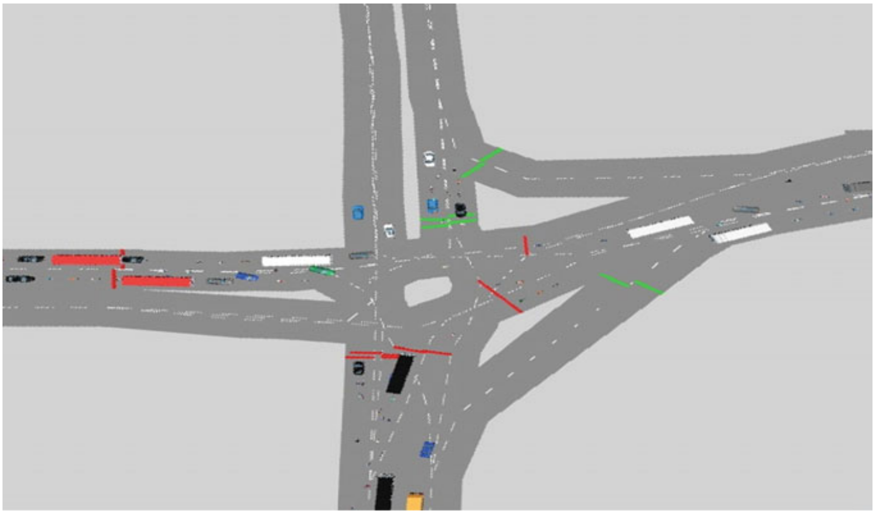


Fig. 3 Snapshot of the microscopic simulation model

Table 8 Vehicle PCUs under the different proportion of TWs

Vehicle type	Proportion of TWs				
	55%	60%	65%	70%	75%
TW	0.24	0.23	0.21	0.20	0.19
ThW	0.83	0.81	0.78	0.75	0.73
SC	1.00	1.00	1.00	1.00	1.00
BC	1.30	1.40	1.50	1.70	1.80
LCV	1.81	1.83	1.84	1.87	1.89
BUS	5.71	5.83	6.0	6.25	6.98
HCV	3.86	3.91	4.13	4.36	4.57
BY	0.26	0.27	0.29	0.30	0.32

the vehicles larger in size than SC, thereby increasing the intersection clearance time and the vehicle PCU at urban signalised intersections.

References

1. Anusha CS, Verma A, Kavitha G (2012) Effects of two-wheelers on saturation flow at signalised intersections in developing countries. J Transp Eng 91(80)
2. Sonu M, Dhamaniya A, Arkatkar S, Joshi G (2016) Time occupancy as measure of PCU at four-legged roundabouts. Transp Lett 1–12. https://doi.org/10.1080/19427867.2016.1154685
3. Indian Roads Congress (IRC): 93 (1990) Guidelines on design and installation of road traffic

signals. Indian Roads Congress, New Delhi, India
4. Singh S, Barhmaiah B, Kodavanji A, Santhakumar SM (2020) Analysis of two-wheeler characteristics at signalised intersection under mixed traffic conditions: a case study of Tiruchirappalli City. Resilience and Sustainable Transportation Systems. In: Proceedings of the 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 35–43. https://doi.org/10.1061/9780784482902.005
5. Singh S, Vidya R, Shukla BK, Santhakumar SM (2021) Analysis of traffic flow characteristics based on area-occupancy concept on urban arterial roads under heterogeneous traffic scenario—a case study of Tiruchirappalli City. In: Mehta YA et al (eds) Trends and Recent Advances in Civil Engineering (TRACE) 2020, Advances in water resources and transportation engineering, Lecture notes in civil engineering, vol 149, Springer Nature, Singapore (2021). https://doi.org/10.1007/978-981-16-1303-6_6
6. Hossain M (2001) Estimation of saturation flow at signalised intersections of developing cities: a micro-simulation modelling approach. Transp Res Rec 35(2):123–141
7. Al-Kaisy A, Hall AF, Reisman ES (2002) Developing passenger car equivalents for heavy vehicles during queue discharge flow. Transp Res Rec 36(8):725–742
8. Minh CC, Sano K (2003) Analysis of motorcycle effects to saturation flow rate at signalised intersection in developing countries. J East Asia Soc Transp Stud 5
9. Arasan VT, Jagadeesh K (1995) Effect of heterogeneity of traffic on delay at signalised intersections. J Transp Eng 121(5):397–404. https://doi.org/10.1061/(ASCE)0733-947X
10. Chandra S, Kumar U (2003) Effect of lane width on capacity under mixed traffic conditions in India. J Transp Eng 129(2):155–160. https://doi.org/10.1061/(ASCE)0733-947X
11. Arasan VT, Vedagiri P (2006) Estimation of saturation flow of heterogeneous traffic using computer simulation. In: Proceedings 20th European conference on modelling and simulation, ECMS, Bonn, Germany
12. Patil RG, Roa KVK, Xu MSN (2007) Saturation flow estimation at signalised intersections in developing countries. In: Proceedings, 86th annual meeting, transportation research board, Washington, DC
13. Arasan VT, Arkatkar SS (2010) Micro-simulation study of effect of volume and road width on PCU of vehicles under heterogeneous traffic. J Transp Eng 136(12):111–119. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000176
14. Asaithambi G, Kanagaraj V, Srinivasan KK, Sivanandan R (2012) Characteristics of mixed traffic on urban arterials with significant volumes of motorized two-wheelers. Transp Res Rec 2317:51–59. https://doi.org/10.3141/2317-07
15. Mohan M, Chandra S (2017) Queue clearance rate method for estimating passenger car equivalents at signalized intersections. J Traf Transp Eng 4(5):487–495. https://doi.org/10.1016/j.jtte.2016.12.003
16. Singh S, Panda RK, Saw DK, Santhakumar SM (2020) Effect of heavy transport vehicles on speed-flow characteristics of mixed traffic on multi lane divided intercity highways, Resilience and Sustainable Transportation Systems. In: 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 176–185. https://doi.org/10.1061/9780784482902.021
17. Singh S, Santhakumar SM (2021) Evaluation of lane-based traffic characteristics of highways under mixed traffic conditions by different methods. J Inst Eng (India) Ser A. 102, 719–735. https://doi.org/10.1007/s40030-021-00549-6
18. Indian Highway Capacity (Indo-HCM) (2017) Manual. Central Road Research Institute (CRRI), New Delhi, India
19. Singh S, Kumar A, Niyas M, Santhakumar SM (2020) Multivariate analysis of freeways speed and time headway under mixed traffic streams. In: 2020 international conference on COMmunication Systems & NETworkS (COMSNETS). Institute of Electrical and Electronics Engineers (IEEE), Bengaluru, India, pp 116–121. https://doi.org/10.1109/COMSNETS48256.2020.9027497
20. Singh S, Shukla BK, Santhakumar SM (2020) Infra-red sensor-based technology for collecting speed and headway data on highways under mixed traffic conditions. In: 2020 7th international

conference on Signal Processing and Integrated Networks (SPIN). Institute of Electrical and Electronics Engineers (IEEE), Noida, India, pp 607–611. https://doi.org/10.1109/SPIN48934.2020.9070829
21. Singh S, Santhakumar SM (2021) Platoon-based impact assessment of heavy-duty vehicles on traffic stream characteristics of highway lanes under mixed traffic environment. Int J ITS Res. https://doi.org/10.1007/s13177-021-00268-z
22. Khan A, Dhamaniya A, Arkatkar S (2021) Effect of two wheeler proportion on passenger car units at roundabout in Indian urban scenario. Transp. Lett 1–12. https://doi.org/10.1080/19427867.2021.1919455
23. Singh S, Santhakumar SM (2021) Empirical analysis of impact of multi-class commercial vehicles on multi-lane highway traffic characteristics under mixed traffic conditions. Int J Transp Sci Tech. https://doi.org/10.1016/j.ijtst.2021.07.005

Influence of Platooning of Heavy Transport Vehicles Operation on Traffic Flow Mix of Intercity Highways

Sandeep Singh and S. Moses Santhakumar

1 Introduction

The increased motorization on rural highways with a considerable share of Heavy Transport Vehicles (HTVs) is supplemented by insufficient infrastructure and lack of traffic management [1]. Vehicles with different physical dimensions behave differently. The traffic speed and flow on the intercity highways are disrupted due to the uncertain placement and movement of the HTVs. The HTVs' presence affects the speed of the neighboring vehicles [2]. Under such scenarios, from the operational point of view, a vehicle being blocked by another slower moving HTV follows the latter for a certain distance until it finds an opportunity to overtake [3]. Let us consider the same situation being followed by all the backed-up vehicles with the generation of slow-moving vehicles as platoon leaders. It leads to the formation of platoons. Generally, moving bottlenecks are caused by these slow-moving HTVs. So, the average speed of all the vehicles behind it on the highways reduce [4]. Hence, platoons are important to be considered in the speed-flow analysis because they impact the traffic speed, flow, and capacity.

Considering this, the development of distinct speed-flow models to determine the capacity of the different lanes for slow-moving vehicles would be accurate in representing the phenomenon of platooning on intercity highways. The capacity suggested in the Highway Capacity Manual (HCM) [5] for other developed, and developing countries cannot be adopted in these conditions because of the differences in the geometric characteristics, traffic characteristics, and driver behavior prevailing in Indian traffic. Additionally, the dynamism of Passenger Car Unit (PCU) values must be assessed because the development of PCU standards under such platoon conditions is deemed necessary. The aim of the study is to evaluate the effects of

S. Singh (✉) · S. M. Santhakumar
Department of Civil Engineering, National Institute of Technology Tiruchirappalli, Tiruchirappalli, Tamil Nadu, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_29

HTV types on highway lanes' speed and flow characteristics under mixed traffic conditions.

2 Literature Review

Alecsandru et al. [6] estimated truck Passenger Car Equivalent (PCE) for different traffic demand and composition levels. They calibrated VISSIM simulated models. The outcomes of this research work showed that PCEs for trucks increased as the compliance ratio increased. Demarchi and Setti [7] examined the influence of overweight vehicles on grades, using PCEs for different types of trucks. Similarly, Al-Kaisy et al. [8] studied the influence of traffic flow, the percentage of the truck, and the type of truck, on PCU. The conclusions drawn were that these variables influence the PCU because of a higher proportion. Chang and Kim [9] attempted to consider the headway of vehicles for capacity estimation. The study detailed the use of different statistical techniques for determining the best confidence level to represent the traffic capacity.

Li et al. [10] modeled the headway distribution, considering the formation of platoons, and estimated the capacity. The study concluded that Gamma distribution was the best fit for the headway distribution of vehicles. Zhou et al. [11] carried out a study to determine PCEs and found that the PCEs are affected by the high proportion of trucks. However, all of these studies and their findings are unworkable for Indian conditions because they were conducted under homogeneous traffic conditions.

In the Indian context, Bharadwaja et al. [12] investigated the impact of the traffic composition on the capacity for intercity expressways. They noticed that the capacity is influenced by the traffic mix composition and emergency lanes. Rahman and Nakamura [13] estimated PCU for non-motorized vehicles based on reducing passenger car speeds. Dhamaniya and Chandra [14] determined the speed of different vehicle types to estimate the PCU of vehicles in India. Recently, Singh and Santhakumar [15] compared the different methods of PCU estimation and determined the capacity values for different lanes of the four-lane highways.

Mallikarjuna and Rao [16] have attempted and proposed an area occupancy-based traffic-concentration representation model to describe the traffic condition on no-lane disciplined-based traffic streams. Gunasekaran et al. [3] determined the platoon flow period for each confidence interval of the headway and obtained different values of traffic capacity. Chandra et al. [17] investigated the effect of traffic mix on highway capacity by proposing a generalized equation to determine capacity. Biswas et al. [18] carried out research to study and model traffic volume and composition for separate vehicle speed and PCE. Recently, Raj et al. [19] laid down a methodology to compute vehicle PCU using the effective area approach.

Although much literature is available for modeling highway characteristics under homogeneous conditions, the platoon modeling and behavior of vehicles under heterogeneous conditions on highways is yet to be explored. From the literature review, it can be summarized that there is a lack of research concerning the platooning phenomenon on highways.

3 Data Collection

The collection of traffic data across the intercity highway near Chennai city was carried out for 38 h, continuously capturing peak hour and off-peak hour traffic covering weekends (Saturday and Sunday) and weekdays (Monday). The four-lane divided NH-45 is 325 km long, which connects Chennai city, and Trichy city is selected as the study site because severe traffic with various vehicle types operates on the highways. Figure 1 illustrates the selected test section.

The traffic data was collected using an Infra-Red (IR) traffic detector system named Transportable Infra-Red Traffic Logger (TIRTL). Figure 2 illustrates the transmitter (TX) and receiver (RX) of the TIRTL.

The traffic measurement was possible with 95 and 96% accuracy for speed and time headway. Further details on the working of the TIRTL can be found in the studies by Singh et al. [20, 21]. The highway consisted of eleven different vehicle classes, as shown in Fig. 3.

Fig. 1 Selected study section

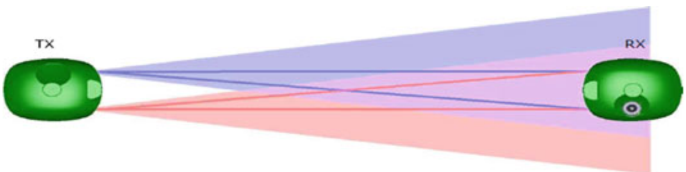


Fig. 2 Schematic representation of the TX and RX

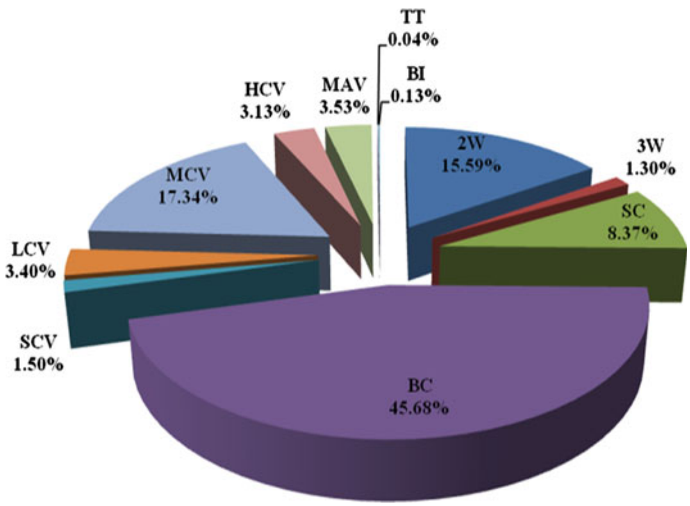


Fig. 3 Traffic composition at NH-45

A total of 36,987 vehicles were observed over 38 h of the analysis period.

4 Data Analysis and Results

4.1 Speed Statistics of Vehicles

The speed data of various vehicle categories recorded by TIRTL are extracted and used for performing descriptive statistical analysis. The different descriptive statistical parameters of vehicle speeds like the minimum speed, maximum speed, different percentiles, Standard Deviation (Std Dev), Skewness, Kurtosis, Time Mean Speed (TMS), and Space Mean Speed (SMS) are determined, which is shown in Table 1. TMS (v_t) and SMS (v_s) are the arithmetic mean and the harmonic mean vehicle speeds as given by Eqs. (1) and (2), respectively.

$$vt = \frac{1}{n}\sum_{i=1}^{n} vi \tag{1}$$

$$vs = \frac{n}{\sum_{i=1}^{n}\left(\frac{1}{vi}\right)} \tag{2}$$

From Table 1, it can be inferred that the TMS of standard/small cars to be the highest with 86.4 kmph followed by big cars with 84.6 kmph. Bicycles were observed to have the lowest average speed of 21.5 kmph amongst the vehicles. Meanwhile, the

Table 1 Summary statistics of the speed of vehicles

Vehicle type	Minimum	Maximum	15th %ile	50th %ile	85th %ile	Std Dev	Skewness	Kurtosis	TMS	SMS
2W	51.4	126.8	52.6	52.5	60.0	7.2	0.9	2.1	55.4	53.5
3W	46.7	72.4	48.5	51.6	57.5	5.4	0.9	2.8	52.0	50.8
SC	53.5	166.5	55.2	78.7	89.2	6.1	0.2	0.9	86.3	80.1
BC	56.8	153.2	58.3	74.8	87.6	8.0	0.7	1.2	84.5	82.0
SCV	49.6	76.7	51.5	55.6	63.8	12.3	0.1	0.1	57.2	54.1
LCV	51.2	79.9	55.8	64.2	76.6	4.2	0.3	1.0	70.4	67.2
MCV	43.5	71.0	46.3	54.7	68.5	9.7	0.6	0.6	65.0	62.7
HCV	38.3	64.5	40.2	49.2	62.4	6.5	0.5	1.7	51.8	49.4
MAV	36.9	61.9	38.4	44.7	55.6	4.6	0.1	2.2	48.2	44.5
TT	24.3	45.5	25.6	27.5	40.7	3.7	0.2	1.0	29.2	21.6
BI	10.6	28.2	11.7	19.3	25.4	1.4	0.1	0.1	21.5	17.8

TMS and SMS of all the vehicles were plotted against the time (in hours) as shown in Fig. 4.

The speed distribution profile of all the vehicle classes against frequency is represented in Fig. 5.

Figure 5 shows that the vehicle speeds in the traffic stream for the study site under mixed traffic conditions are normally distributed.

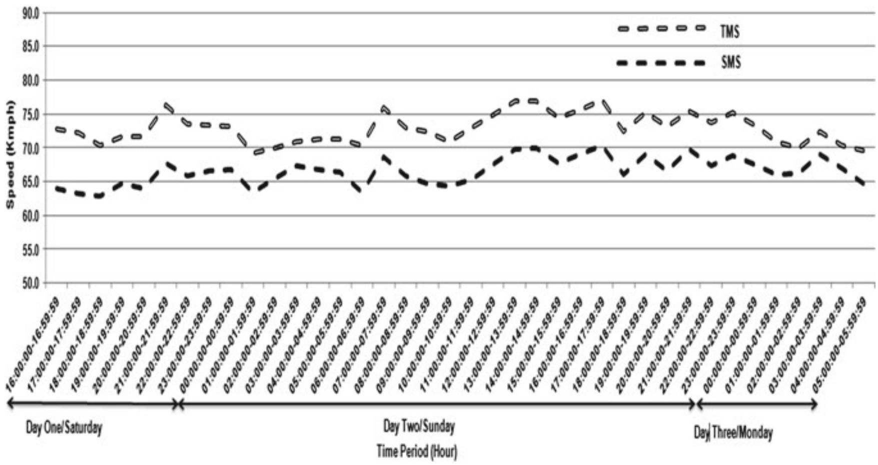


Fig. 4 Trend of TMS and SMS

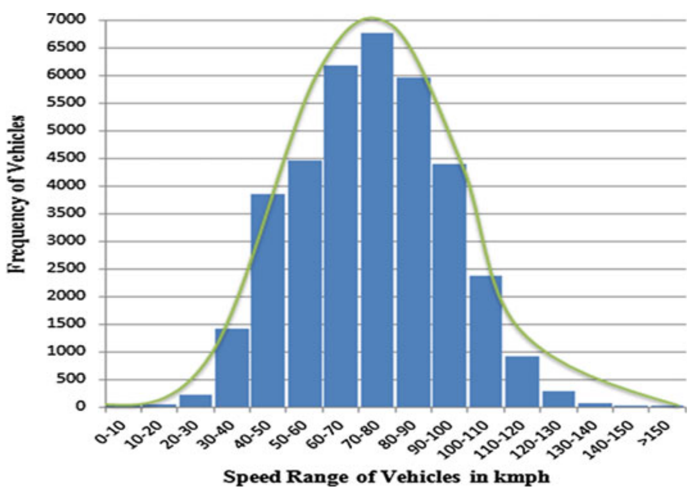


Fig. 5 Speed distribution profile

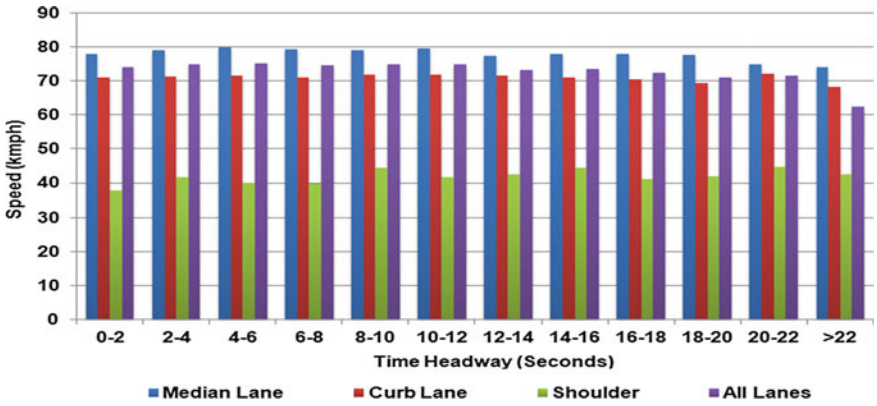


Fig. 6 Speed versus time headway plot

4.2 Speed Versus Time Headway Analysis

The lane-based speed versus time headway analysis is carried on the acquired data. It is observed that the speed of vehicles over the incremental ranges of time headway is the highest on the median lane compared to the curb lane and the shoulder. The vehicle speed versus time headway analysis shows that the vehicles maintained an average speed of 78 kmph on the median lane, 71 kmph on the curb lane, and 42 kmph on the shoulder. The overall average speed on all the lanes is found to be 73 kmph. This is depicted in Fig. 6.

4.3 Passenger Car Unit (PCU) Estimation

The traffic on Indian highways is defined by the high degree of heterogeneity. In the present study, the PCU of the vehicle is estimated using Eq. (3) as suggested by the Indo-HCM [22]. The estimated vehicle PCUs are shown in Table 2.

$$\mathrm{PCU}_i = (V_{sc}/V_i)/(A_{sc}/A_i) \tag{3}$$

where,

PCU_i	PCU of the *i*th vehicle
V_i	Speed of the *i*th vehicle (km/h)
V_{sc}	Speed of SC (km/h)
A_{sc}	Area of SC (m^2)
A_i	Area of the *i*th vehicle (m^2).

Table 2 Estimation of PCU values for different vehicles

Class of vehicle	Speed (Kmph)	Area (m^2)	PCU
2W	55.4	1.08	0.27
3W	52.0	3.41	0.91
SC	86.3	6.18	1.00
BC	84.5	8.50	1.40
SCV	57.2	6.23	1.52
LCV	70.4	8.84	1.75
MCV	65.0	18.14	3.90
HCV	51.8	20.39	5.55
MAV	48.2	25.13	7.25
TT	29.2	11.06	5.29
BI	21.5	0.60	0.38

4.4 Temporal Variation of Traffic Flow

The 5-min aggregated DPCU values for every hour of each vehicle class are multiplied to the respective vehicles/hour/direction (vphpd) to homogenize the traffic flow into pcu/hour/direction (pcuphpd). The peak traffic flow values observed over the analysis period are 1826 vphpd between 17:00:00 h to 17:59:59 h and 2951 pcuphpd between 04:00:00 h to 04:59:59 h. This is depicted in Fig. 7.

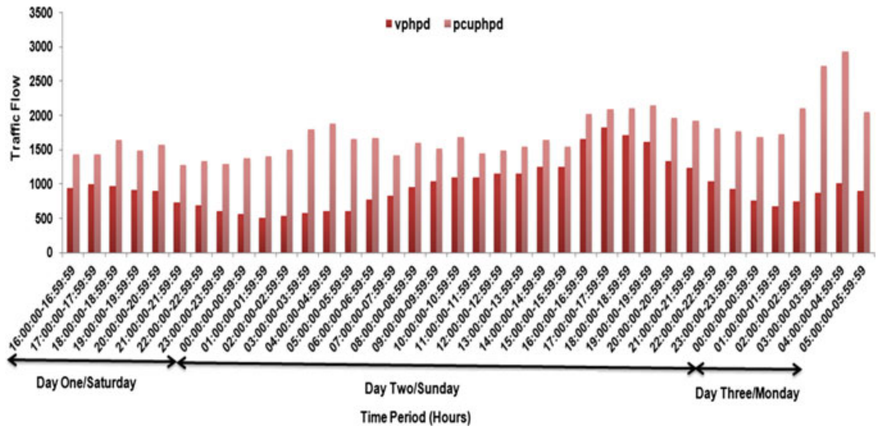


Fig. 7 Temporal variation of traffic flow

4.5 HTVs' Platooning Operation in Different Lanes

In this study, platoon size is represented using a 1-min flow rate (PCU/minute). The vehicles detected 1-min before the arrival of the HTVs, at the IR sensor detector point are used to represent no/minimal platoon formation. In comparison, those vehicles detected one minute after the arrival of the HTVs at the IR sensor detector point represent the platoon formation [23]. The primary reason for this kind of platoon characterization, i.e., one minute before and after the arrival of HTVs is taken under consideration for platoon analysis because it meaningfully describes the platoon formation and no/minimal platoon formation phenomenon in which the speed-flow characteristics are found to be representing the field characteristics [24].

This analysis was done separately for Median Lane (ML) and Curb Lane (CL) traffic during the peak hours between 3 and 9 PM. Though the highest peak flow of 2951 pcuphpd is observed between 4 and 5 AM, the time between 3 and 9 PM was deemed to represent platoons. This is done because the vehicle-to-vehicle interaction was more when compared to the former time period. Since in the former time, the peak flow of 2951 pcuphpd has been due to the higher number of large-sized vehicles whose PCU is more than the fact that maximum flow occurs in that time. Another point to note here is that both ML and CL are separately chosen for analysis to show that the influence of HTVs on platoon formation is more when these HTVs operate in the ML when compared to that in the CL, which is represented by the speed-flow characteristic curves.

The speed-flow relationships are established for the proposed concept of platooning phenomenon for HTVs operating separately on ML and CL. One minute before the HTVs' arrival on ML and CL and one minute after the HTVs' arrival on ML and CL are considered to define the no/minimal platoon formation and platoon formation, respectively. This is shown in Figs. 8, 9, and 10.

Figure 8. depicts a significant drop of 24.39%, i.e., from 102 to 82 PCU/min in the flow values after the MCVs arrival on ML compared to that before the MCVs

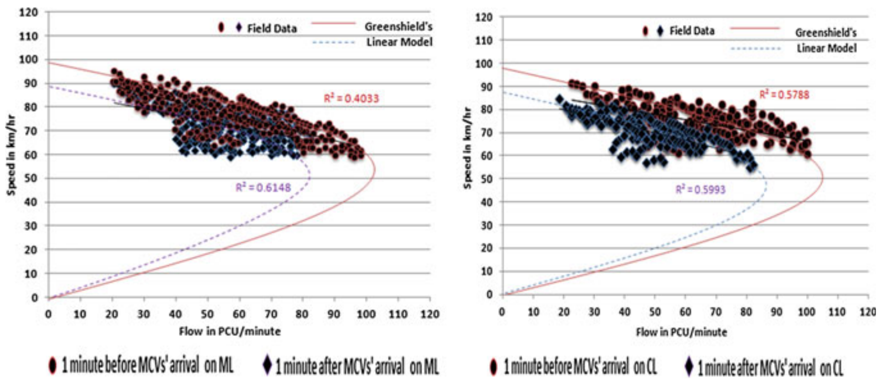


Fig. 8 Speed-flow curves for MCVs as platoon leader

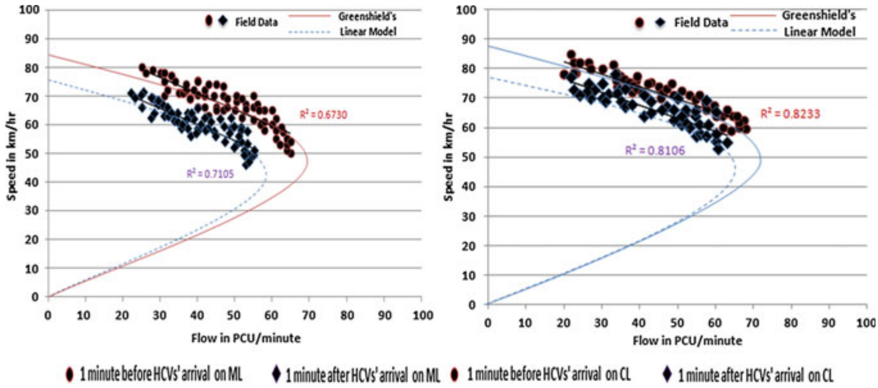


Fig. 9 Speed-flow curves for HCVs as platoon leader

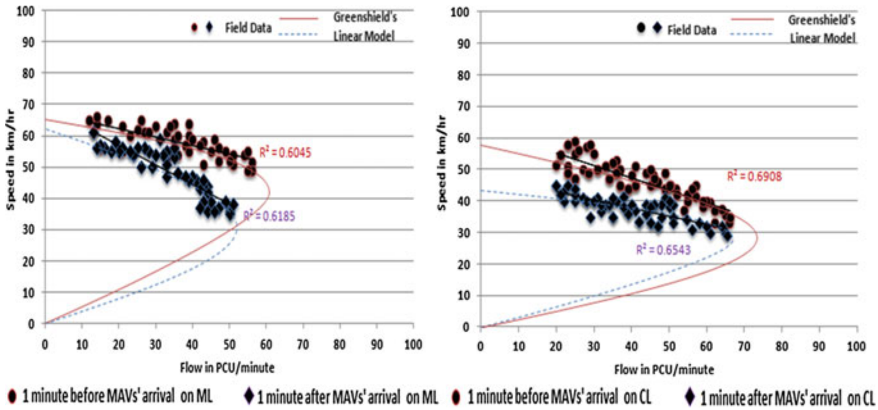


Fig. 10 Speed-flow curves for MAVs as platoon leader

arrival on ML. Likewise, the KL flow for MCVs operation reduced from 105 to 87 PCU/min, which is 20.69%. This shows that the speed and flow of all other vehicles operating on the traffic stream are disrupted by MCVs movement on the ML and CL, thereby forming a platoon where the MCVs act as platoon leaders. Also, it can be inferred that the percentage drop in the speed-flow values after the MCVs arrival on ML is considerably more when compared to that after the MCVs arrival on CL. Hence, the MCVs in the ML with lower operating speeds reduce the vehicle speeds, which eventually reduced the capacity of the intercity highways.

Figure 9 depicts a significant drop of 18.64%, i.e., from 70 to 59 PCU/min in the flow values after the HCVs arrival on ML compared to that before the HCVs arrival on ML. Likewise, the KL flow for HCVs operation reduced from 71 to 65 PCU/min, which is 9.23%. This shows that the speed and flow of all other vehicles operating on the traffic stream are disrupted by HCVs movement on the ML and CL, forming a platoon where the HCVs act as platoon leaders. Also, it can be inferred that the percentage drop in the speed-flow values after the HCVs arrival on ML is considerably more when compared to that after the HCVs arrival on CL. Hence, the HCVs in the ML with low operating speed reduce the vehicle speed, which eventually reduced the capacity of the intercity highways.

Figure 10 depicts a significant drop of 17.31%, i.e., from 61 to 52 PCU/min in the flow values after the MAVs arrival on ML compared to that before the MAVs arrival on ML. Likewise, the KL flow for MAVs operation reduced from 75 to 68 PCU/min, which is 10.29%. This shows that the speed and flow of all other vehicles operating on the traffic stream are disrupted by MAVs movement on the ML and CL, thereby forming a platoon where the MAVs act as platoon leaders. Also, it can be inferred that the percentage drop in the speed-flow values after the MAVs arrival on ML is considerably more when compared to that after the MAVs arrival on CL. Hence, the MAVs in the ML with lower operating speeds reduce the vehicle speeds, which eventually reduced the capacity of the intercity highways.

5 Conclusion

The present study examined the effects of the various HTVs operation on the different lanes of the highways under mixed traffic conditions. Since different HTV types can result in different traffic conditions leading to abrupt vehicle movement and lane changing, interpreting the speed-flow relationship of different HTVs is essential for improving highway safety. The results showed that the speed-flow characteristics decrease consistently with the slow speed operation of the HTV's in the high-speed lanes. The findings of this study can provide valuable insights for traffic engineers and decision-makers to develop potential strategies to manage highway traffic. Future research may include an analysis of similar effects of HTVs for the urban arterials during peak hours.

References

1. Singh S, Vidya R, Shukla BK, Santhakumar SM (2021) Analysis of traffic flow characteristics based on area-occupancy concept on urban arterial roads under heterogeneous traffic scenario—a case study of Tiruchirappalli City. In: Mehta YA et al (eds) Trends and Recent Advances in Civil Engineering (TRACE) 2020. Advances in water resources and transportation engineering, Lecture Notes in Civil Engineering, vol 149. Springer Nature, Singapore. https://doi.org/10.1007/978-981-16-1303-6_6
2. Singh S, Panda RK, Saw DK, Santhakumar SM (2020) Effect of heavy transport vehicles on speed-flow characteristics of mixed traffic on multi lane divided intercity highways, Resilience and Sustainable Transportation Systems. In: Proceedings of the 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 176–185. https://doi.org/10.1061/9780784482902.021

3. Gunasekaran K, Kalaanidhi S, Gayathri H, Velmurugan S (2015) A concept of platoon flow duration in data aggregation for urban road capacity estimation. In: Proceedings of the 14th international conference on computers in urban planning and urban management. MIT, Cambridge, MA
4. Singh S, Barhmaiah B, Kodavanji A, Santhakumar SM (2020) Analysis of two-wheeler characteristics at signalised intersection under mixed traffic conditions: a case study of Tiruchirappalli City. Resilience and Sustainable Transportation Systems. In: Proceedings of the 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 35–43. https://doi.org/10.1061/9780784482902.005
5. Highway Capacity Manual (2010) Transportation research board. National Research Council, Washington, DC
6. Alecsandru C, Ishak S, Qi Y (2012) Passenger car equivalents of trucks on four-lane rural freeways under lane restriction and different traffic conditions. Can J Civ Eng 39:1145–1155. https://doi.org/10.1139/l2012-098
7. Demarchi SH, Setti JR (2003) Limitations of passenger-car equivalent derivation for traffic streams with more than one truck type. Transp Res Rec 1852(1):96–104
8. Al-Kaisy A, Jung Y, Rakha H (2005) Developing passenger car equivalency factors for heavy vehicles during congestion. J Transp Eng 131(7):514–523. https://doi.org/10.1061/(asce)0733-947x(2005)
9. Chang M, Kim Y (2006) Development of capacity estimation method from statistical distribution of observed traffic flow. In: Transportation research circular E-C018 presented at the 4th international symposium on highway capacity
10. Li Y, Lu H, Yu X (2011) Traffic flow headway distribution and capacity analysis using urban arterial road data. Present Int Conf Electr Technol Civil Eng (ICETCE) 1:1821–1824. https://doi.org/10.1109/ICETCE.2011.5775326
11. Zhou J, Rilett L, Jones E, Chen Y (2018) Estimating passenger car equivalents on level freeway segments experiencing high truck percentages and differential average speeds. Transp Res Rec 2672:44–54. https://doi.org/10.1177/0361198118798237
12. Bharadwaj N, Mathew S, Pani A, Arkatkar S, Joshi G, Ravinder K (2018) Effect of traffic composition and emergency lane on capacity: a case study of intercity expressway in India. Transp Lett 10:316–332. https://doi.org/10.1080/19427867.2016.1265237
13. Rahman M, Nakamura F (2005) Measuring passenger car equivalents for non-motorized vehicle (Rickshaws) at mid-block sections. J East Asia Soc Transp Stud 6:119–126 (2005). https://doi.org/10.11175/easts.6.119
14. Dhamaniya A, Chandra S (2016) Conceptual approach for estimating dynamic passenger car units on urban arterial roads by using simultaneous equations. Transp Res Rec 2553(1):108–116
15. Singh S, Santhakumar SM (2021) Evaluation of lane-based traffic characteristics of highways under mixed traffic conditions by different methods. J Inst Eng India Ser A. 102, 719–735. https://doi.org/10.1007/s40030-021-00549-6
16. Mallikarjuna C, Rao KR (2006) Area occupancy characteristics of heterogeneous traffic. Transportmetrica 2:223–236. https://doi.org/10.1080/18128600608685661
17. Chandra S, Mehar A, Velmurugan S (2016) Effect of traffic composition on capacity of multilane highways. KSCE J Civ Eng 20:2033–2040. https://doi.org/10.1007/s12205-015-0479-9
18. Biswas S, Chandra S, Ghosh I (2018) An advanced approach for estimation of PCU values on undivided urban roads under heterogeneous traffic conditions. Transp Lett 00:1–10. https://doi.org/10.1080/19427867.2018.1563268
19. Raj P, Sivagnanasundaram K, Asaithambi G, Ravi Shankar AU (2019) Review of methods for estimation of passenger car unit values of vehicles. J Transp Eng 145:04019019. https://doi.org/10.1061/JTEPBS.0000234
20. Singh S, Kumar A, Niyas M, Santhakumar SM (2020) Multivariate analysis of freeways speed and time headway under mixed traffic streams. In: 2020 international conference on COMmunication Systems & NETworkS (COMSNETS). Institute of Electrical and Electronics Engineers (IEEE), Bengaluru, India, pp 116–121. https://doi.org/10.1109/COMSNETS48256.2020.9027497

21. Singh S, Shukla BK, Santhakumar SM (2020) Infra-red sensor-based technology for collecting speed and headway data on highways under mixed traffic conditions. In: 2020 7th international conference on Signal Processing and Integrated Networks (SPIN). Institute of Electrical and Electronics Engineers (IEEE), Noida, India, pp 607–611. https://doi.org/10.1109/SPIN48934.2020.9070829
22. Indian Highway Capacity (Indo-HCM) (2017) Manual. Central Road Research Institute (CRRI), New Delhi, India
23. Singh S, Santhakumar SM (2021) Platoon-based impact assessment of heavy-duty vehicles on traffic stream characteristics of highway lanes under mixed traffic environment. Int J ITS Res. https://doi.org/10.1007/s13177-021-00268-z
24. Singh S, Santhakumar SM (2021) Empirical analysis of impact of multi-class commercial vehicles on multi-lane highway traffic characteristics under mixed traffic conditions. Int J Transp Sci Tech. https://doi.org/10.1016/j.ijtst.2021.07.005

Determination of Service Quality of Bus Transit System Using SERVQUAL Method Based on User's Perceptions and Expectations

Soumik Sarkar, Mokaddes Ali Ahmed, and Saikat Deb

1 Introduction

In India, the urban population in 2017 was approximately around 34% [35] and it is a monotonically increasing trend since 1960. The population of India is slated to grow to 1.53 billion by 2030 [24] showing significant jump from the 2011 levels of 1.21 billion. The combination of these two factors is going to put lot of pressure on the transportation infrastructure in the urban areas in years to come. Also, India is experiencing tremendous growth in number of vehicles in urban areas with increasing economic development causing transit users to switch to private modes. The per capita income of India for the financial year of 2018–19 is ₹ 1,25,397 showing a rise of 11.10% from the previous year [22]. The vehicle density (no. of vehicles per 1000 population) has risen from 8 in 1981 to 72 in 2005 [33]. The spatial distribution of vehicle ownership in India shows much variability; in 2005, the vehicle density in the 23 million-plus population cities was 215 vehicles/1000 population while the vehicle density was only 72 vehicles/1000 population when country as a whole was considered [33]. Due to this, urban areas are facing problems such as traffic congestions, poor air quality. As per a study conducted on 10 roads of Kolkata city revealed that ₹ 74,078 is lost in only two hours (i.e. 9–10 am and 6–7 pm) per day due to congestion on the roads [2]. It is estimated that in 2005, the 23 million-plus cities account for consumption of one-fourth of the transport fuel consumed in the country leading to emissions of about 15.29 million tonne of CO_2 in 2005 [33]. It is observed that poor air quality leads to 1.7 times higher prevalence of respiratory symptoms [26].

S. Sarkar (✉) · M. Ali Ahmed
Department of Civil Engineering, NIT Silchar, Silchar, Assam, India

S. Deb
Civil Engineering Programme, Assam Down Town University, Guwahati, Assam, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_30

In order to reduce the detrimental effects caused by increase in number of vehicles in urban areas, the public transport service needs to be made available and attractive to the users. The bus transit system is the most available form of transit service in most Indian cities. Bus transit service is effective in meeting higher demands of the urban population cost effectively as it requires less capital investment to introduce the service compared to rail-based transit systems. Also, about 25% of the urban population in India live below the poverty line [21] whose choice of mode depends on affordability. The bus transit system is affordable as well as provides more accessibility to the users. However, due to lack of resources, poor maintenance, inefficiency in operation, overcrowding and undependable serviceability of public bus service, dissatisfied users in the cities are increasingly switching to private cars or two wheelers [30].

To prevent the mode shift of transit users to private vehicles the service quality offered by existing public bus transit system needs to be determined regularly and amends are to areas where service is lagging behind. To determine the service quality, the Ministry of Urban Development (MoUD) has introduced Service Level Benchmarks, 2009 [25] for measurement of performance of transit systems based on benchmarking techniques. It measures the Level-of-Service (LOS) offered based on six parameters like presence of organized service, availability of service, coverage of service, average waiting time, level of comfort based on load factor and percentage of fleet as per specification. On the basis of the six parameters, the LOS scale is divided into four categories from LOS 1 representing best possible service to LOS 4 representing worst service. The MoUD, 2009, suggested LOS scale was developed based on expert opinion and concentrates only on quantitative performance indicators. The scale therefore lacks to represent the user's perception about the service.

The Transit Capacity and Quality of Service Manual, 2nd Edition [34] defines the LOS as "The overall measured or perceived performance of transit service from the Passenger's point of view" and highlighted the importance of user's perception in measuring service quality. Many researchers in recent years highlighted the importance of user's perception in measuring the service quality [4, 13, 28]. However, the user perception towards the service, however, differs from person to person based on factors like socio-economic characteristics, personal needs of the user, demographic values, previous experience of the service, etc. [37]. Researchers have used different techniques to measure service quality of the transit system using user's perception, as summarized by Güner [15]. Some of the statistical techniques are structural equation modelling [7], factor analysis [6, 17, 22, 27], discrete choice logit models [8, 11, 12, 16, 29, 31], combination of factor analysis and structural equation model [10, 36], multi-criteria decision-making techniques like the Analytic Hierarchy Process (AHP) [15, 20], Artificial Neural Networks (ANN) [14, 19], Law of Successive Interval Scaling [5], etc.

In most of the aforementioned studies, the authors identified the factors that affect the transit user perception about the service and have quantified the service quality based on those factors. The studies provide a good measure of the existing service but do not take into account the expectations of the users from the service in calculating the service quality of the system. The desired service quality requirement changes with expectation of the transit users. SERVQUAL is one such method which takes transit users perception and expectation from the service into account for calculation of service quality. SERVQUAL method was developed as a tool to measure service quality offered by the service sector like bank, retail shops, insurance company, etc. [28]. However, the method has been widely applied to various fields including transport-related studies. SERVQUAL method has been applied to measure service quality of the public bus transit system in Kumasi, Ghana [32]; metro service in Montreal, Canada [1], airline service quality in Taiwan [3], etc. SERVQUAL is used in this study as it is able to measure perceived and expected service quality and also because of its simplicity transit users as well as operators are able to interpret and implement its findings. The objective of this study is therefore to determine the LOS of the bus transit service based on the user's perception and expectations.

2 Case Study Area and Selection of Transit System

Kolkata is the state capital of West Bengal, India and is the largest metropolitan city on the Eastern region of India. The city is situated on the banks of the Hooghly River. Due to presence of riverine port and presence of very good connectivity with different parts of the country, the city is a major centre of trade and culture. The land-use pattern of Kolkata is such that only 6% of the city area is available for transport infrastructure [18], due to this the city faces with the problem of traffic congestion. Also, the city is oriented in North–South direction and due to this city structure the major arterial roads, the rail-based transit system like the Local Rail Transit system and the Metro Rail Transit system are aligned along the North–South direction. The bus transit system therefore plays a vital role as feeder service as well as in providing accessibility to the transit users. In Kolkata, about 87% of the passenger-km was through public transit systems in 2005 [33]. The mode share of bus transit system was 55.75% on an average week day [9]. The vehicle density in Kolkata is 64 vehicles/1000 population which is below the national average of 72 vehicles/1000 population [33], due to the presence of extensive public transport network of the city. Therefore, it becomes imperative to study the service quality offered by the bus transit service to prevent the shift of transit users to private vehicles.

The bus transit service of Kolkata comprises of buses operated by the State Transport Undertaking (STU) i.e. the West Bengal Transport Corporation (WBTC) and buses operated by private operators. The WBTC was formed in 2017 by merging the three STUs (CSTC, CTC and WBSTC) operating in the KMA. However, before the merger of the three STUs, all the three STUs were suffering from drop in ridership due to problems like use of old rickety fleet, unreliable service, poor route choices, etc. However, the state government puts capital investment in STUs for fleet modernization under the JNNURM scheme and now the WBTC operates a fleet of Non-AC, AC and Volvo buses which are BS-IV compliant. The WBTC is therefore the single largest operator of buses in Kolkata; it operates buses on approximately 145 routes in the KMA. For the present study, the service quality of buses operated by the WBTC measured as findings of the study have better chances of applicability because of its centralized administrative nature while the privately operated buses are decentralized and are very slow to adapt to changes.

3 Methodology

The objective to this study is to determine the service quality offered by the bus transit system based on the user perception and expectation from the service. Also, the public bus transit authority very rarely conducts studies to determine the LOS being delivered, due sluggish nature of the authority as well as due to insufficient funds or skilled workforce. So, for any research method to be readily applied by the authority, needs to be simple, easy to perform, require less number of personnel to perform and the data analysis should also not require complex calculations as well as costly software packages. SERVQUAL method is one such method which is easy and simple.

SERVQUAL method is used in the present study. SERVQUAL method has 5 service quality dimensions (Reliability, Assurance, Tangibility, Empathy and Responsiveness) and 22 questions focusing on different parameters of the service. To make the questions and service quality dimensions more easy to interpret and relatable to the service being surveyed; some of the questions and dimensions are modified. A 5-point Likert scale is used in this study to get user responses. SERVQUAL method measures the gap in service delivery by expected service quality values from the perceived values. A zero gap score indicates that existing service quality is at par with user expectations while a negative gap score indicates lag in service delivery.

Data collection involves two steps one is preparation of the questionnaire survey and second is collecting responses from the transit users. The questionnaire survey form consists of three sections, first is SERVQUAL-based questions, second is the overall rating of the existing service and third section contains some demographic questions to determine the socio-economic characteristics of the respondents. The survey was conducted from 22nd October to 5th November, 2018 in two shifts from 08:00 to 13:00 and 15:00 to 20:00 h. The survey was conducted across 10 locations in Kolkata as depicted in Fig. 1. The locations were selected such that most of the trips performed by users ended in those locations such as important changeover points like Howrah Railway Station Bus Terminus, Ultadanga, Shyambazar 5-point crossing, Park Circus where users switch from bus to rail transit services; office areas and commercial centres like Esplanade, Gariahat, New Town Bus Stand, Karunamoyee, etc. The surveyors handed the questionnaire survey form to willing respondents who were regular users of the service. A total of 414 responses were collected, however 350 responses were found valid after reviewing the filled-up forms for any missing data as well as presence of absent-minded responses.

The collected data was analysed using IBM SPSS software (version 20). Paired Sample t-test was performed on the collected sample to establish the alternate hypothesis i.e. the perception and expectation dataset differ from each other and their means

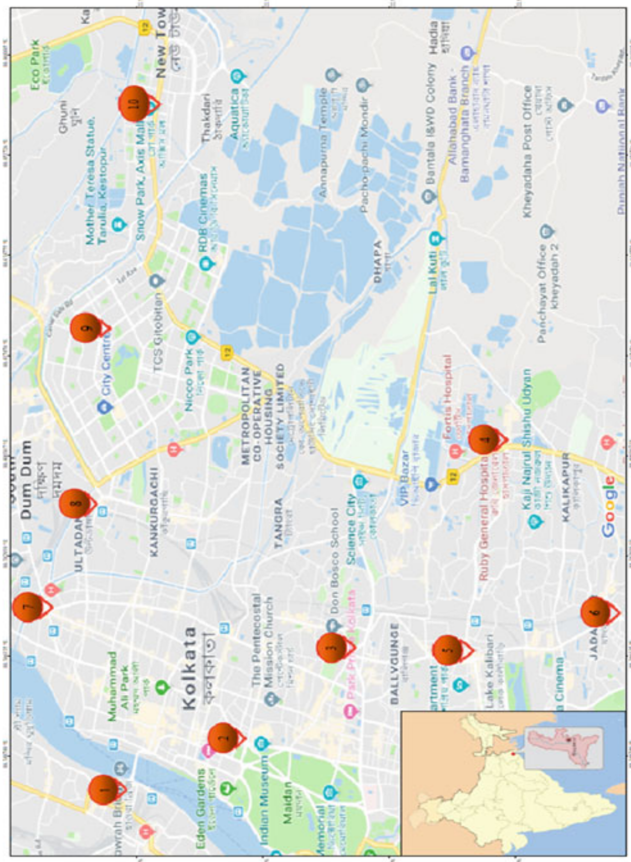


Fig. 1 Map showing survey locations in Kolkata. (1) Howrah Railway Station Bus Terminal, (2) Esplanade, (3) Park Circus, (4) Ruby Hospital Bus Stand, (5) Gariahat, (6) Jadavpur 8B Bus Stand, (7) Shyambazar 5-point crossing, (8) Ultadanga, (9) Karunamoyee and (10) New Town Bus Stand

Table 1 LOS category ranges with description

Percentage deviation (%)	Designated LOS level	Description
0–20	LOS 1	Best possible service, respondents are happy with the performance of the service, bus transit service is efficient
20–40	LOS 2	Respondents are happy with maximum service indicators, bus service is good
40–60	LOS 3	Respondents are somewhat happy with the service, some service indicators are not performing up to the user's expectations
60–80	LOS 4	Majority of the respondents are not happy with the service, bus transit service is performing poorly
80–100	LOS 5	This is the worst service level, bus transit system major operational review

can be compared. To check for the scale reliability, Cronbach's Alpha value is calculated. The LOS offered by the existing service is determined by calculating the percentage deviation of user perception values from their expectations from the service, the percentage deviation is calculated with the help Eq. (1). In this study, LOS scale developed by [6] is used. In this LOS scale, the percentage deviation (ranging from 0 to 100%) is divided into five equal intervals. The scale is presented in Table 1 with brief discussion of each category. Further, multiple regression analysis is performed to determine the coefficient weights of each service quality dimension towards overall satisfaction of the transit users. With the coefficient weights known, the transit authority could prioritize on improving the service quality based on the importance of the factors.

$$\text{Percentage deviation} = \frac{\text{expectation} - \text{perception}}{\text{expectation}} \times 100 \tag{1}$$

4 Results and Discussion

Out of the 350 respondents, 233 (66.57%) respondents were Male and 117 respondents were Female (33.43%). Around 60.57% of the respondents belonged to the 21–30 years age-group and 50.29% respondents were on work trip. The socio-economic characteristics of the collected data are presented in Table 2.

Paired Sample t-test is conducted to verify whether the user perception and expectation values are statistically different. There are two hypotheses in paired sample t-test i.e. Null Hypothesis (the paired population means are equal) and Alternate Hypothesis (the paired population means are not equal) only when the alternate

Table 2 Socio-economic characteristics of collected Sample

Socio-economic characteristics of the respondents		Percentage of total responses (%)
Gender	Male	66.57
	Female	33.43
Age	15–20	5.43
	21–30	60.57
	31–40	22.29
	41–50	6.86
	Over 50	4.86
Monthly income	Less than 5000	23.14
	5000–10,000	13.43
	10,000–15,000	13.14
	15,000–20,000	12.00
	20,000–30,000	16.86
	30,000–40,000	6.57
	40,000–50,000	6.57
	More than 50,000	8.29
Purpose of the performed trip	Work trip	50.29
	Educational trip	20.29
	Shopping trip	10.29
	Recreational trip	9.43
	Business related trip	9.71

hypothesis is valid the means can be compared. For the paired sample t-test, a cut-off value of 0.05 is taken representing a confidence level of 95%, it is the most widely used by the researchers. The results of the paired sample t-test are presented in Table 3 along with the mean and standard deviation values of the 22 factors considered.

From Table 3, it is observed that significance value is less than 0.05, so, the alternate hypothesis is valid and the means can be compared. Gap in service delivery is calculated by deducting the expectation score from the perception score for each factor. From the gap score, it is observed that factors Q5 i.e. “Passenger belongings are safe” and Q3 i.e. “Buses follow schedule” have the largest gap between user perception and expectations while the factor Q19 i.e. “Bus staff assist disabled persons and minors in boarding and alighting” has the lowest gap. Also, from the overall performance rating of the existing service, it is observed that the mean of all the collected ratings is 3.183 which correspond to a rating of “Average”.

Cronbach’s Alpha value is calculated to check the reliability of the scale. Cronbach’s Alpha values are calculated for each perception and expectation datasets of the service quality dimensions and are presented in Table 4.

Table 3 Paired sample t-test output

Dimension	Factors	Code	Perception		Expectation		Gap score	Sig
			Mean	Std. Dev	Mean	Std. Dev		
Reliability	Buses are punctual	Q1	3.129	0.983	4.783	0.413	−1.654	0.00
	Buses never breakdown on road	Q2	3.746	0.826	4.771	0.507	−1.026	0.00
	Buses follow schedule	Q3	2.817	0.958	4.800	0.435	−1.983	0.00
	Bus service has convenient operating hours	Q4	3.766	0.897	4.840	0.375	−1.074	0.00
Safety	Passenger belongings are safe	Q5	2.837	0.942	4.837	0.377	−2.000	0.00
	Buses are driven safely	Q6	3.329	1.034	4.823	0.390	−1.494	0.00
	Bus drivers are well trained	Q7	3.577	0.945	4.849	0.359	−1.271	0.00
	Buses are easy to get on and off	Q8	3.251	0.876	4.783	0.427	−1.531	0.00
Comfort	Buses are modern and are equipped with modern amenities	Q9	3.100	1.086	4.826	0.423	−1.726	0.00
	Buses have adequate capacity and number of seats	Q10	2.889	1.077	4.786	0.425	−1.897	0.00
	Bus staff appear neat and clean	Q11	3.240	0.869	4.731	0.457	−1.491	0.00
	Bus staff behaviour is polite	Q12	3.151	0.897	4.734	0.474	−1.583	0.00
	Bus tickets are easily available and no issue with change	Q13	3.509	0.939	4.789	0.409	−1.280	0.00
	Buses are neat and clean	Q14	3.020	0.977	4.783	0.420	−1.763	0.00

(continued)

Table 3 (continued)

Dimension	Factors	Code	Perception		Expectation		Gap score	Sig
			Mean	Std. Dev	Mean	Std. Dev		
Responsiveness	Bus staff communicate with passengers clearly	Q15	3.320	0.915	4.754	0.431	−1.434	0.00
	Bus staff are responsive to passenger requests	Q16	3.343	0.913	4.737	0.478	−1.394	0.00
	Bus staff are able to solve minor issues	Q17	3.297	0.817	4.746	0.443	−1.449	0.00
	Bus staff are willing to attend passenger requests	Q18	3.160	0.913	4.737	0.490	−1.577	0.00
Empathy	Bus staff assist disabled persons and minors in boarding and alighting	Q19	4.029	0.869	4.751	0.471	−0.723	0.00
	Bus driver wait for any passenger running towards the bus	Q20	3.463	0.938	4.520	0.614	−1.057	0.00
	Bus service looks after the interests of passengers	Q21	3.043	0.847	4.511	0.575	−1.469	0.00
	Bus service information are accessible to passengers	Q22	1.040	1.040	4.511	0.580	−1.331	0.00
Overall Performance rating of the existing service				**Mean**		**3.183**		
				Std. Dev		**0.961**		

The cut-off value for the Cronbach's Alpha is taken as 0.7 as stated by Parasuraman et al. [28]. But, from Table 4, it can be seen that Cronbach's Alpha value for perception dataset of Responsiveness and Empathy dimension is less than 0.7 and inclusion of these dimensions will make the scale unreliable. To improve the reliability of the scale either the dimensions are excluded from final scale or factors can be reshuffled. In

Table 4 Cronbach' Alpha values of service quality dimensions

S. No.	Service quality	Perception dataset	Expectation dataset
1	Reliability	0.718	0.741
2	Safety	0.708	0.781
3	Comfort	0.820	0.868
4	Responsiveness	0.688	0.812
5	Empathy	0.608	0.764

Table 5 Cronbach's Alpha value of service quality dimension datasets after reshuffling of factors

S. No.	Service quality	Perception dataset	Expectation dataset
1	Reliability	0.718	0.741
2	Safety	0.708	0.781
3	Comfort	0.787	0.845
4	Responsiveness	0.710	0.774
5	Empathy	0.705	0.747

this study, trial has been made to improve the reliability by reshuffling of the factors. The factor Q12 i.e. "Bus staff behavior is polite" has been shifted from Comfort to Empathy dimension, as it has similarities with both dimensions. Similarly, factor Q15 i.e. "Bus staff communicate with passengers clearly" is moved to Empathy dimension and factor Q21 i.e. "Bus service looks after the interests of passengers" is moved to Responsiveness dimension. The Cronbach's Alpha values after reshuffling are presented in Table 5.

After reshuffling of the factors, the Cronbach's Alpha values of both perception and expectations datasets of the service quality dimensions are greater than 0.7. Although a value of more than 0.9 is desirable but due to heterogeneous nature of the human perception, Cronbach's Alpha value greater than 0.7 can be deemed satisfactory.

The LOS of the existing service is determined by the percentage deviation of perceptions from the expectation values. To calculate the percentage deviation, the perception and expectation scores of each service quality dimension are used. The perception and expectation scores of each dimension are calculated by taking the average of the factor scores in each dimension. The results are presented in Table 6.

From Table 6, it is observed that the bus transit service in Kolkata provides LOS 2 category of service which can be categorized as a "Good" service. However, there are areas where the service needs to improve. Comfort and Safety are the areas where the service needs improvements.

Regression Analysis is performed to determine the weights of each service quality dimension towards the overall satisfaction of the users. Two regression analysis is performed, one to establish the Satisfaction equation and another to establish the Dissatisfaction equation. For the Satisfaction equation, analysis is performed by

Table 6 LOS offered by the existing service

Service quality dimension	Perception (Mean)	Expectation (Mean)	Percentage deviation (%)	LOS category
Reliability	3.364	4.799	29.8	LOS 2
Safety	3.249	4.823	32.6	LOS 2
Comfort	3.151	4.783	34.0	LOS 2
Responsiveness	3.211	4.683	31.1	LOS 2
Empathy	3.429	4.654	26.3	LOS 2

considering the perception scores of the dimensions as independent variable and the overall performance rating as the dependent variable. The Dissatisfaction equation is established to determine the effect of user expectations on the overall performance rating. For the Dissatisfaction equation, the normalized deviation scores of user perception from their expectation are taken as the independent variable and the overall performance rating as the dependent variable. The normalized deviation is calculated with the help of Eq. 1. The outputs of the multiple linear regression analysis are presented in Tables 7 and 8.

Table 7 Regression output for the satisfaction equation

Model R^2	Model	Un-standardized coefficients		Standardized coefficient	Sig
		B	Std. error	β	
0.650	(Constant)	−1.231	0.186		0.000
	Reliability	0.359	0.065	0.252	0.000
	Safety	0.203	0.074	0.145	0.006
	Comfort	0.350	0.067	0.266	0.000
	Responsiveness	0.253	0.069	0.166	0.000
	Empathy	0.185	0.080	0.122	0.021

Table 8 Regression output for the Dissatisfaction Equation

Model R^2	Model	Unstandardized coefficients		Standardized coefficient	Sig
		B	Std. Error	β	
0.591	(Constant)	5.127	0.094		0.000
	Reliability	−1.647	0.333	−0.239	0.000
	Safety	−0.966	0.374	−0.138	0.010
	Comfort	−1.796	0.348	−0.280	0.000
	Responsiveness	−0.950	0.333	−0.139	0.005
	Empathy	−0.883	0.390	−0.117	0.024

The R^2 value for the Satisfaction equation is 0.65 or 65%, which represent a 65% fit. Also, the significance values are less than 0.05, which means that the output is statistically significant. From Table 7, it is observed that the Reliability dimension has the largest coefficient weight while the Empathy dimension has the lowest coefficient weight. While for the Dissatisfaction equation, the R^2 value is 0.591 or 59.1%, which is a satisfactory fit. Also, significance values are less the 0.05 denoting that output is statistically significant. From Table 8, it is observed that the Comfort dimension has the largest coefficient weight and Empathy has the lowest coefficient weight. From the output of both the regression analysis, it can be concluded that Reliability and Comfort are the most important factors to the transit users of Kolkata. While Empathy is the least important factor as it has lowest coefficient weight in both the regression analysis outputs. However, an interesting fact can be observed that the Safety dimension which has the second highest percentage deviation (see Table 2) but does not have much coefficient weight in both the equations which is little bit contradictory. This may be due to the fact that Safety is not a major issue in some of the routes that were surveyed or may be because the fact that the factors put under Safety dimension were gender neutral.

5 Conclusion

The purpose of this study was to determine the service quality of the existing bus transit system in Kolkata based on user's perceptions and expectations. It is observed that the bus transit system offers LOS 2 category service, which can be categorized as a "Good" service. However, it still lags behind the user expectations on all the 22 factors that were considered. Comfort and Safety are the areas where the service lags the most. Based on the gap scores of the factors, factor such as "Passenger belongings are safe", "Buses follow schedule", "Buses have adequate capacity and number of seats", "Buses are neat and clean" are the areas where users feel that the service lags behind their expectations the most. While, the users feel that the service is somewhat up to their expectations on the factors such as "Bus staff assist disabled persons and minors in boarding and alighting", "Buses never breakdown on road" and "Bus service has convenient operating hours". Also, based on the overall performance ratings provided by the users, the service can be classified as an average service.

Regression Analysis was performed to determine the user's priorities about the service. It is observed from the regression analysis that Reliability and Comfort dimensions of the service quality are the most important factors for the transit users. While, Empathy plays the least important role in overall satisfaction of the users. So, improvements brought to improve the operational reliability of the service and comfort and convenience of the users will help to improve the user satisfaction.

To improve the reliability of the service, the schedule adherence of the buses needs to be improved for that bus drivers shall be instructed to maintain strict arrival and departures schedules from the terminal bus stops. Also, a significant amount of time is lost at the bus stops due to cash-based ticketing so, smart card-based ticketing may be introduced to reduce the dwell time. Recently, the buses have been installed with GPS-based live bus tracking systems, the GPS data may be analysed to identify the bottlenecks in the city road network and routes can be modified to improve schedule adherence. To improve the comfort of the users, the best way is to reduce the overcrowding. Some routes have very high overcrowding during the peak hours, so more buses can be introduced on those routes. Also, to improve safety of the users against theft, the bus drivers shall be instructed to keep the bus doors shut in between the bus stops. As it has been observed that most of the pick-pocketing occurs near the bus doors as pick-pockets find it easy to escape. The cleanliness of the buses shall be ensured by the transit authority.

References

1. Awasthi A, Chauhan SS, Omrani H, Panahi A (2011) A hybrid approach based on SERVQUAL and fuzzy TOPSIS for evaluating transportation service quality. Comput Indus Eng 61(3):637–646
2. Chakrabartty A, Gupta S (2015) Estimation of congestion cost in the city of Kolkata—a case study. Curr Urban Stud 03(02):95–104
3. Chou CC, Liu LJ, Huang SF, Yih JM, Han TC (2011) An evaluation of airline service quality using the fuzzy weighted SERVQUAL method. Appl Soft Comput J 11(2):2117–2128
4. Das S, Pandit D (2013) Importance of user perception in evaluating level of service for bus transit for a developing country like India: a review. Transp Rev 33(4):402–420
5. Das S, Pandit D (2015) Determination of level-of-service scale values for quantitative bus transit service attributes based on user perception. Transp A: Transport Sci 11(1):1–21
6. Deb S, Ali Ahmed M (2018) Determining the service quality of the city bus service based on users' perceptions and expectations. Travel Behav Soc 12(July 2017):1–10
7. de Oña J, de Oña R, Eboli L, Mazzulla G (2013) Perceived service quality in bus transit service: a structural equation approach. Transp Policy 29:219–226
8. dell'Olio L, Ibeas A, Cecín P (2010) Modelling user perception of bus transit quality. Transp Policy 17(6):388–397
9. Dey T, Bhaduri S (2012) Changing profile of state transport undertakings in mass transport services: a case of Kolkata city. J Arts Sci Commer 2(1):45–56
10. Eboli L, Mazzulla G (2007) Service quality attributes affecting customer satisfaction for bus transit. J Public Transp 10(3):21–34
11. Eboli L, Mazzulla G (2008a) Willingness-to-pay of public transport users for improvement in service quality. 38:107–118
12. Eboli L, Mazzulla G (2008) A stated preference experiment for measuring service quality in public transport. Transp Plan Technol 31(5):509–523
13. Eboli L, Mazzulla G (2011) A methodology for evaluating transit service quality based on subjective and objective measures from the passenger's point of view. Transp Policy 18(1):172–181
14. Garrido C, De Oña R, De Oña J (2014) Neural networks for analyzing service quality in public transportation. Expert Syst Appl 41(15):6830–6838
15. Güner S (2018) Measuring the quality of public transportation systems and ranking the bus transit routes using multi-criteria decision making techniques. Case Stud Transp Policy 6(2):214–224
16. Hensher DA, Stopher P, Bullock P (2003) Service quality—developing a service quality index in the provision of commercial bus contracts. Transp Res Part A: Policy Pract 37(6):499–517

17. Hu K, Jen W (2006) Passengers' perceived service quality of city buses in Taipei: scale development and measurement. Transp Rev 26(5):645–662
18. IDFCL (2008) Comprehensive mobility plan: back to basics, Kolkata Metropolitan Area. Infrastructure Development Finance Corporation Ltd., Kolkata
19. Islam MR, Hadiuzzaman M, Banik R, Hasnat MM, Musabbir SR, Hossain S (2016) Bus service quality prediction and attribute ranking: a neural network approach. Public Transport 8(2):295–313
20. Mahmoud M, Hine J (2013) Using AHP to measure the perception gap between current and potential users of bus services. Transp Plan Technol 36(1):4–23
21. MoHUPA (2009) India: urban poverty report. Ministry of Housing and Urban Poverty Alleviation, Government of India, New Delhi. Available at: https://www.undp.org/content/dam/india/docs/india_urban_poverty_report_2009.pdf
22. Morton C, Caulfield B, Anable J (2016) Customer perceptions of quality of service in public transport: evidence for bus transit in Scotland. Case Stud Transp Policy 4(3):199–207
23. MoSPI (2019) First advance estimates of national income, 2018–19. Ministry of Statistics and Programme Implementation, Government of India, New Delhi. Available at: http://pib.nic.in/newsite/PrintRelease.aspx?relid=187267
24. MoSPI (2017) Youth in India. Ministry of Statistics and Programme Implementation, Government of India, New Delhi. Available at: http://mospi.nic.in/sites/default/files/publication_reports/Youth_in_India-2017.pdf
25. MoUD (2009) Service level benchmarks for urban transport at a glance. Ministry of Urban Development, Government of India, New Delhi. Available at: http://mohua.gov.in/upload/uploadfiles/files/Service_level.pdf
26. Nongkynrih B, Gupta S, Rizwan S (2013) Air pollution in Delhi: its magnitude and effects on health. Indian J Commun Med 38(1):4–8
27. Paquette J, Bellavance F, Cordeau J-F, Laporte G (2012) Measuring quality of service in dial-a-ride operations: the case of a Canadian city. Transportation 39(3):539–564
28. Parasuraman A, Zeithaml V, Berry L (1988) SERVQUAL: a multiple-item scale for measuring consumer perceptions of service quality. J Retail 64(January):12–40
29. Prioni P, Hensher D (2000) Measuring service quality in scheduled bus services. J Public Transp 3(2):51–74
30. Pucher J, Korattyswaroopam N, Ittyerah N (2004) The crisis of public transport in India: overwhelming needs but limited resources. J Public Transp 7(4):1–20
31. Román C, Martín JC, Espino R (2013) Using stated preferences to analyze the service quality of public transport. Int J Sustain Transp 8(1):28–46
32. Sam EF, Hamidu O, Daniels S (2018) SERVQUAL analysis of public bus transport services in Kumasi metropolis, Ghana: core user perspectives. Case Stud Transp Policy 6(1):25–31
33. Sudhakara Reddy B, Balachandra P (2012) Urban mobility: a comparative analysis of megacities of India. Transp Policy 21:152–164
34. TRB (2003) Transit capacity and quality of service manual: TCRP Report 100, 2nd edn. Transportation Research Board, Washington, D.C. Available at: http://www.trb.org/Main/Blurbs/153590.aspx
35. World Bank Website data on Urban Population, https://data.worldbank.org/indicator/SP.URB.TOTL.IN.ZS?locations=IN. Last accessed on 27 May 2019
36. Yaya LHP, Fortià MF, Canals CS, Marimon F (2015) Service quality assessment of public transport and the implication role of demographic characteristics. Public Transp 7(3):409–428
37. Zeithaml VA, Berry LL, Parasuraman A (1993) The nature and determinants of customer expectations of service. J Acad Mark Sci 21(1):1–12

Impact of Street Design and Operational Factors on Level of Satisfaction of Green Transport Users in a Large Metropolitan Area

Eeshan Bhaduri, Dipanjan Nag, and Arkopal Kishore Goswami

1 Green Transport Scenario

1.1 Background and Need for the Study

The concept of 'green transportation' focuses on the efficient and effective use of resources (read fossil fuel), improvisation of transport infrastructure (e.g., non-motorized infrastructures, electric vehicles, green materials), and promulgating healthier travel choices [1]. Non-motorized transport (NMT) modes are prime examples of 'green transport' as those have zero pollution effect except in production state and play a key role in diminishing congestion effect. This paper focuses on two such NMT modes i.e., walking and cycle/e-rickshaws.

Research has shown that most Asian cities lie in the lower half of transport-related energy consumption curve [2, 3], which may be attributed to their high urban density resulting from compact land use pattern. Indian cities, being no exemption, have been developed more organically [4], reflected in its urban modal share which indicates almost one-third of the total trips are walking trips [5, 6]. Although recent studies suggest that its modal share is declining, [7] and is predicted to slide further with the policy focus majorly on improving the motorized travel experience by building capital-intensive flyovers, bridges, and subway transits [8]. The conflict between pedestrians and motorized modes has been profound in the Indian context where vehicular population in urban areas are rising at 10.07%, which is more than the yearly average growth in urban population (3.2%) within 2001–2015 [9]. It results

E. Bhaduri (✉) · D. Nag · A. Kishore Goswami
Ranbir and Chitra Gupta School of Infrastructure Design and Management, Indian Institute of Technology Kharagpur, Kharagpur 721302, India
e-mail: eeshanbhaduri@iitkgp.ac.in

A. Kishore Goswami
e-mail: akgoswami@infra.iitkgp.ac.in

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_31

in the growing perception that road edge space in unsafe for pedestrians [10] which in turn plays a vital role in petering out the propensity of walking in Indian cities.

Another non-motorized mode of transport that is prevalent in India but has not received much formal attention in urban transport planning is the cycle rickshaw. Apart from its various transport advantages, it also provides instant means of livelihood, which in turn plays a crucial role in socioeconomic dynamics of developing countries [11]. However, it has got entangled in the vicious web of illegality and got ignored for decades by transport planners in the presumption that it hampers vehicular traffic flow. Some studies have investigated its sustainability, modal choice, and impact on heterogeneous traffic flow in subcontinent context [12]. However, there is limited literature that analyzes its role as a feeder and in assessing its demand [8].

Studies have suggested that a better understanding of users' perception can be useful in the development of strategies for providing a comfortable and safe environment [13]. As such, our research attempts to assess the context-sensitive solutions and design flexibility for the above-mentioned NMT modes i.e., walking, and cycle/e-rickshaw, based upon the necessity to understand user needs and incorporate them in the planning and design process.

2 Current Status in NMT Research

In this section, a brief appraisal of prevailing literature has been elicited focusing on two aspects—(1) theoretical construct of how users' satisfaction analysis has aided NMT policy-making; and (2) empirical findings from modeling the influence of street design and traffic operations on the (dis) satisfaction of pedestrian and cycle-rickshaw users.

2.1 Application of Users' Satisfaction Analysis in NMT Decision-Making Process

The scientific integration of policy-level decision-making and user satisfaction analysis is a comparatively unexplored proponent in NMT domain as most of the earlier studies had ignored emotional perceptions of user satisfaction [14–16]. The term 'user satisfaction' denotes the accomplishment of customers' expectation, measured in terms of the expectations actually fulfilled [17]. The study by St-Louis et al. [18] highlights the growing dissatisfaction of NMT users compared to other modes, which include automobile, bus, metro, and commuter train. Some studies done in western countries like Canada and Sweden show contrasting scenario as those suggest that users of active transportation are the most satisfied, with high satisfaction scores being assigned to cyclists followed by pedestrians [19, 20]. Previous study indicates that perception factors have a greater influence on satisfaction [21]. This analogy is somewhat exemplified in Sydney, Australia, where pedestrians or bicyclists in the inner city core enjoyed their commute more than private vehicle users while going to work or study [22]. Another study originating from Scotland reported higher travel satisfaction for active modes, public transport, and multimodal trips relative to car-only trips made in between 1997 and 2010 [23].

2.2 Modeling User Satisfaction for NMT Users

Modeling methodologies have two major facets—(1) survey design to collect user perception; and (2) statistical modeling techniques to extract underlying causal mechanism. It has been found that researchers most of the time have used field intercept surveys and closed route surveys for measuring the perception of the level of satisfaction by the NMT users [24, 25]. It is worth mentioning that the survey design process is derived from the modeling approaches which can be broadly divided into two categories—(1) aggregate approach and (2) disaggregate approach. Aggregate approaches are comparatively more expensive and incapable of estimating the variation in the behavioral choice of users as it depended mainly on the statistical relationship between variables at an aggregate level [26]. Therefore, a disaggregate travel behavior modeling approach was adopted for this study. Disaggregate models can be estimated with two data types—(1) revealed preference (RP), and (2) stated preference (SP) survey data. Various researchers have observed that RP data has higher acceptability than SP data as revealed preferences are a closer reflection to the true market behavior as opposed to the hypothetical situations presented in SP scenario, which may have a higher probability of not being consistent with actual behavior [27].

Studies have found that the combined approach of considering both qualitative (convenience, security, safety, comfort, etc.) and quantitative measures (pedestrian speed, flow, and density) is more representative for assessing the user perception in any particular transport mode [28]. Further, it has also been established that the assessment methods should not be solely based on quantitative data, as they may not present a comprehensive analysis, and qualitative aspects like perception ratings may be incorporated [29]. Several studies have used multiple linear regression and ordered logit method [28, 30, 31] to model the user perception of non-motorized modes.

The recent research findings show that several infrastructural paucities that lead to NMT users' dissatisfaction include discontinuous pedestrian facility; ill-designed pedestrian infrastructure; and absence of proper markings, to name a few [15, 32]. Research has shown that perceptional attributes such as the absence of convenience, comfort, safety, and shade [33], and design factors such as the presence of property entrances, bus shelters, and the amount of vehicular lanes [34] impact negatively on the level satisfaction for pedestrians. In other words, human-centric designs can upgrade pedestrians' satisfaction level and perception of a walkable community [35]. In the Indian context, there has been a focus on developing a pedestrian level of service, as perceived by users, where different researches identify prime factors like safety, security, mobility, etc. that influence the walking environment [36]. Few other studies have analyzed non-motorized users' perception in the context of particular aspects of an infrastructural facility rather than an inclusive approach [37].

As various Indian cities continue to grow, and their transport patterns involve, there is a need to integrate NMT modes with both the personal vehicular modes as well as the public/mass transportation modes. In this regard, the study presented in this paper is a novel effort to ascertain how satisfied are the users who are availing the services offered by the cycle rickshaw NMT mode, and subsequently predict the factors, which if improved, could enhance their satisfaction. The study simultaneously compares the cycle-rickshaw users' satisfaction to that of the pedestrians' satisfaction in order to gain a holistic understanding of the NMT users' satisfaction. The study is carried out in two distinctly different neighborhoods of a metropolitan area to understand the similarities and differences among the NMT users. Ultimately, the findings of this study would benefit any metropolitan area authority to gauge the users' perception about the NMT facilities and services and would also help the authorities in implementing strategies that are likely to enhance the user satisfaction.

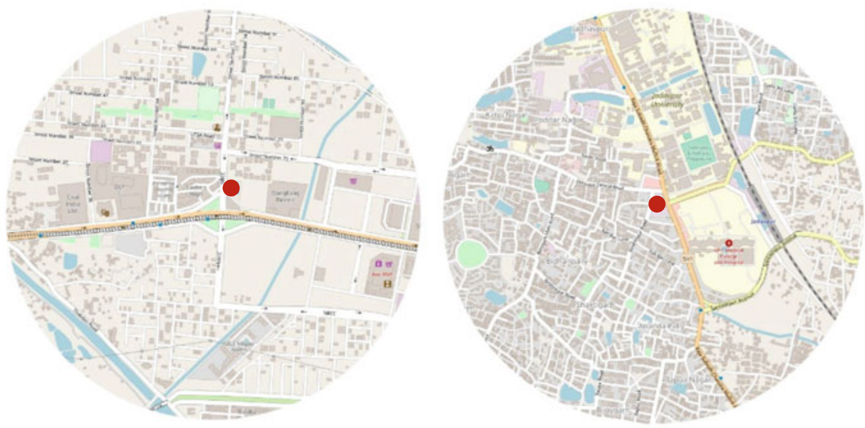
3 Study Design

3.1 Scope

This study designs an RP survey using an essential set of street design and traffic operation attributes influencing NMT users' choice decisions in two different Indian contexts—one newly planned and another old organically developed urban areas. For modeling purpose, ordered logistic regression (OLR), a well-established users' satisfaction modeling tool in the transportation domain [38–40] has been selected. In OLR, the dependent variables are of discrete nature (e.g., categorical variables, count data, qualitative rankings) [37]. Mostly, different studies use the scoring system [41, 42], which is not readily applicable for assessing the perception. Although methods like analytical hierarchy process (AHP) and structural equation modeling (SEM) have been used, researchers are skeptical about using those for a survey with a large sample size as 'cold-called' respondents might show a propensity to provide arbitrary answers, resulting in a very high degree of inconsistency [43]. In contrast, different seminal works done in this domain have shown the considerable explanatory ability of logistic regression [37, 44]. Simultaneously, ordinal logit model shows biasedness mostly with relatively small-sized sample and when several outcome categories are relatively large, and some categories are rare [45]. However, our study being free of such paucity is an ideal case where OLR could have been carried out. It ensures better modeling as it can analyze the responses collected in the Likert scale during the survey. Likert scales were developed in 1932 as the familiar five-point bipolar response that most people are familiar with today [46]. It can be observed that even scale creates the problem of extreme bias toward positive or negative opinions, whereas the actual scenario represents people are of more on a neutral stand-point [47, 48]. Furthermore, different researchers have utilized Likert scale analysis for similar perception assessment [42, 49]. Finally, the effect of heterogeneity concerning two different urban locations on user perception was examined. Moreover, odds ratio analysis (ORA), another technique used for futuristic analysis [50], has been conducted in order to evaluate a set of hypothetical scenarios.

3.2 Study Area

Data was collected in Newtown and Jadavpur, two prominent urban areas in the vicinities of largest metropolitan of eastern India i.e., Kolkata, with varying urban, transport, and socio-demographic characteristics. These two different urban areas are selected to assess the similarities and differences in user perception within a city and its implication on future policy decisions. Newtown was created in the late 1900s in the eastern outskirts of Kolkata to cater to the dual purposes of establishing the new central business district as well as increasing housing stock supply. Hence, the area developed following some of the well-established planning guidelines like grid iron pattern (refer Fig. 1a). Whereas history of Jadavpur dates back 1860s as a quiet suburban retreat and later development test-bed for various educational institutes like Indian Association for the Cultivation of Science and Bengal Technical Institute (later known as Jadavpur University). The development charts more organic pattern



(a) (b)

Fig. 1 Survey area Maps **a** Buffer around Newtown bus stop (Area ‘A’); and **b** Buffer around Jadavpur 8B bus stop (Area ‘B’) (Buffer radius 2 km with centroid shown in red dot, *Source* OpenStreetMap) (refer Fig. 1b) as Jadavpur witnessed huge refugee influx and subsequent squatting (forcible occupation of others' land) during the initial years of independence and it is only in 1984 when Kolkata Municipality included these areas by extending its boundary.

About our survey, the centroids of such zones with 2 km buffer radius (average of walking and rickshaw trip length threshold) were considered for survey purpose similar to the study done by Nag et al. [51]. This also helps to elicit context-specific influence of local infrastructure on user perception. Newtown (within Rajarhat Gopalpur municipality, North 24 Parganas) (referred to as 'Area A' henceforth) has been developing as a planned satellite urban area with census 2011 population estimated as 235,946, whereas Jadavpur (within Kolkata municipal corporation, Kolkata) (referred to as 'Area B' henceforth) is a well-established part of the Kolkata with census 2011 population estimated as 1,595,746. In both cases, the survey links (or stretches) have been selected to be in a mix of residential, commercial, and terminal land uses with access to all kinds of motorized as well as NMT modes. The road network of two survey locations has been shown in Fig. 1a, b which provide an idea about the varying urban form of those two.

4 Methodology

The research methodology consisted of two major parts:

1. The first part concentrates on modeling the user satisfaction of existing NMT-built environment and operational factors. This exercise was done based on the ratings provided by NMT users. The model also considers any possible impact of inherent pedestrian behavior such as pedestrian/rickshaw trip frequency and distance/travel time related to trip purposes on individual ratings.
2. The second part focuses on predicting user satisfaction for hypothetical future scenarios which can be attained upon the improvement of (statistically) significant factors for NMT users. The exercise was done with the help of odds ratio analysis whose results provide a tool to assess probable policy implications. It is emphasized herein that the hypothetical future scenarios were only used for statistical analysis purposes and were not shown to the users to elicit their stated preferences of satisfaction.

4.1 Selection of Attributes Affecting Overall Satisfaction of Individual NMT Users

Factors impacting overall satisfaction of users. Nag and Goswami [51] identify 31 parameters from existing literature that influence a users' perception of the pedestrian walking environment internationally (e.g., volume of pedestrian, safety, speed of vehicular traffic, obstructions, connectivity) The study also goes on to rank the most relevant factors for Indian conditions, and the resulting attributes have been taken forward in this study. These 14 attributes are identified as potential motivators or deterrents for the choice of the walking mode, and as such pedestrians' perceptions regarding these factors have been collected. These attributes have been presented in Table 1.

Factors affecting overall satisfaction of individual cycle/e-rickshaw users. When it came to deciding the factors that might affect the satisfaction of rickshaw users, the study outlined the probable factors based on the research team's past experience of using rickshaws in Kolkata. These factors were further vetted by transport experts in academia. This was necessitated as the body of literature on rickshaws is rather lean. A total of 9 attributes for cycle/e-rickshaw users are identified as potential motivators or deterrents for the choice of this mode, and as such rickshaw users' perceptions regarding these factors have been collected. These attributes have been presented in Table 2.

4.2 Survey Design and Data Collection

A travel intercept survey was executed to collect the responses where the sample size was decided based on the census demographic data (the year 2011) which estimates the total population in Newtown and Jadavpur as 18,31,692. It is worth mentioning that the neighborhood chosen were of comparable size. The minimum sample size was calculated based on the population of study area [52]. The mathematical expression for calculating sample size has been mentioned in Eq. (1).

$$S(\text{Sample size}) = \left(\chi^2 * N * P * (1 - P)\right) / \left(d^2 * (N - 1) + \chi^2 * P * (1 - P)\right) \tag{1}$$

where,

N = Population size

d = Degree of accuracy expressed as a proportion (assumed 0.05)

P = Population proportion (assumed 0.5 for maximum sample size).

Moreover, chi-square value is calculated for the degree of freedom 1 for a requisite statistical confidence level.

In total, approximately 700 users combining both the locations were approached, out of which 18.30% of the surveys were either incomplete or non-responses which leaves with about 570 valid responses for the combined population size of approximately 1.83 million. Based on Eq. (1), the minimum sample requirement for any city with more than 10 lakh population is about 384 which indicates acceptability of the final sample size for our study. It is worth mentioning that the revealed preference survey was conducted over seven days in each city, including both, weekdays and

Table 1 Summary of attributes impacting satisfaction of pedestrians

Serial no	Attributes	What does it mean?	Measurement indicator	Nature of variable
1	Quality of footpath	Footpath surface material, texture and condition	Uneven or slippery surfaces, presence of potholes etc.	Motivator
2	Continuity	Availability of footpath throughout the journey	Grade separated footpath for study stretch	Motivator
3	Shade cover	Shading along the footpath both natural or man-made	Use and frequency of shading devices	Motivator
4	Directional signs	Symbols and emblems for wayfinding	Use and frequency of signage	Motivator
5	Air pollutant levels	Exposure to poor air quality while walking	Air quality index like PM level along study stretch	Deterrent
6	Street illumination	Availability and functioning of street lights along the footpath	Frequency and working condition of street lights along study stretch	Motivator
7	Lateral separation	Presence of barrier between the footpath and adjoining road	Length and height of separators like railings or boulders along study stretch	Motivator
8	Vehicular speed	Speed of vehicles on the adjoining road	Average and design speed of vehicles along study stretch	Deterrent
9	Comfort in walking	Presence of obstructions along the footpath	Frequency of obstructions like light post, hawkers etc. along study stretch	Deterrent
10	Crowdedness	Presence of other pedestrians along the footpath	Width of footpath	Deterrent
11	Footpath grade	Presence of upslope or downslope along the footpath	Gradient of footpath	Motivator/deterrent
12	Intersection quality	Zebra crossings at signalized or unsignalized intersections	Numbers of intersections having zebra crossing	Motivator

(continued)

Table 1 (continued)

Serial no	Attributes	What does it mean?	Measurement indicator	Nature of variable
13	Safety during daytime	Sense of security felt during daytime	Numbers and working condition of surveillance devices installed along study stretch and Numbers of accidents and anti-social events during daytime	Motivator
14	Safety during nighttime	Sense of security felt during nighttime	Numbers and working condition of surveillance devices installed along study stretch and Numbers of accidents and anti-social events during nighttime	Motivator

weekends, as well as peak and non-peak hours. Figure 2 shows a couple of images, which are representative of the pedestrian and NMT environment of the case study areas 'A' and 'B', respectively.

Satisfaction ratings of individual infrastructural parameters as well as of overall situation were collected, respectively, for pedestrians and rickshaw users on a scale of 1–5.

where,

1 = Highly unsatisfied,

2 = Moderately unsatisfied,

3 = Neutral,

4 = Moderately satisfied, and.

5 = Highly satisfied,

Data was also collected about trip frequency and distance/travel time related to diverse trip purposes such as work, education, recreation, etc. As stated earlier, the respondents are asked to rate the different parameters (as shown in Tables 1 and 2) based on their perception of satisfaction. In addition, an overall satisfaction level was also recorded. Furthermore, the internal consistency of the questionnaire was calculated to assess the reliability of the responses. Cronbach's alpha (α) was used to measure the internal consistency which was found to be 0.67, an acceptable value to indicate reliability.

Table 2 Summary of attributes impacting satisfaction level of cycle/e-rickshaw users

S. No.	Attributes	What does it mean?	Measurement indicator	Nature of variable
1	Pavement quality	Condition of pavement surface	Uneven or slippery surfaces, presence of potholes etc	Motivator
2	On road traffic volume	Presence of a large number of vehicles along the road	Traffic count along study stretch	Deterrent
3	Passenger load	Sense of comfort felt due to the number of passengers in the cycle/e-rickshaw	Numbers of passenger on cycle/e-rickshaw	Deterrent
4	Pick-up and Drop-off points	The proximity of the rickshaw stands from origin/destination	Field Distance of cycle/e-rickshaw stops from origin/destination	Motivator
5	The physical condition of cycle/e-rickshaw	The operational state of cycle/e-rickshaws	Years of operation	Motivator
6	Availability of cycle/e-rickshaw	Availability during the time of day	Numbers of cycle/e-rickshaw in designated stops	Motivator
7	Behavior of drivers/rickshaw pullers	Relationship of users and service providers	Number of arguments related to fare and travel in a weekly scale	Motivator
8	Safety during travel	Presence of high-speed traffic along the road	Average and design speed of vehicles along study stretch	Motivator
9	Fare	Amount paid to avail service	Average fare to be paid traveling unit distance	Deterrent

(a) **(b)**

Fig. 2 Study stretches. **a** Newtown (Area 'A'); and **b** Jadavpur (Area 'B')

4.3 Methodology for Statistical Analysis

Modeling use responses—Ordered logistic regression (OLR). In Ordered logit models, the dependent variable is categorical in nature, e.g., count data, a series of qualitative rankings. OLR can consider multi-category responses making it appropriate for analyzing the ordered responses recorded in this study.

The logged odds are unified under a linear form of the independent variables, and the general form of the OLR model is given by Eq. (2).

$$L_j = \beta 0 + \sum_{j=1}^{p} \beta_j X_j \tag{2}$$

where,

L_j = Logged-odds; X_j = independent variables in the model; the p = number of variables; and, J = total number of ordered response.

Odd's ratio analysis. The Odd's ratio (OR) is a ratio between the odds of success to the failure of an event. This quantity is used in understanding the impact in the outcome variable when change is made by one-unit of the independent variable keeping other independent variables constant. A reference (base) case needs to be identified for capturing this change. This analysis is vital in comprehending the independent attributes' impact on the dependent variable.

$$\mathrm{OR} = \frac{Xe/Xne}{Ye/Yne} \tag{3}$$

where,

X_e = Occurrence of an event due to the presence of factor of interest; X_{ne} = Non-occurrence of the event due to presence of factor of interest; Y_e = Occurrence of an event in the absence of factor of interest;

Y_{ne} = Non-occurrence of the event in the absence of factor of interest.

The calibrated OLR model was used to identify statistically significant factors which impacted in influencing users' satisfaction level. Following this, the OR analysis was done to gauge various improvement scenarios.

5 Descriptive Findings from Collected Responses

As mentioned in Sect. 3.2, two urban areas in Kolkata—Newtown (Area 'A') and Jadavpur (Area 'B') have been selected for the survey. Descriptive statistics of sociodemographic attributes for both the areas have been presented in Table 3.

5.1 *Assessment of Travel Characteristics for Pedestrians*

The average walking distance was observed to be 1.4 km (i.e., average 18 min). With increasing distance, the number of people walking declined and 75% of the respondents walked a distance of 1.5 km. 35% of the respondents walked daily (35%), but there were also cases, where 32% respondents undertook walking trips only once a week. Majority (31%) of trip purpose was related to shopping, followed by

Table 3 Socio-demographic attributes from household survey data

Classification		Pedestrian		Rickshaw users	
		Area 'A'	Area 'B'	Area 'A'	Area 'B'
Gender					
	Male	135	147	103	110
	Female	54	69	44	39
Age (Years)					
Age 1	< 15	5	7	5	5
Age 2	15–25	39	27	34	25
Age 3	25–35	34	27	36	32
Age 4	35–50	27	27	34	25
Age 5	> 50	78	84	80	71
Monthly household income (INR)					
Income 1	< 5000	5	5	7	2
Income 2	5000–15,000	19	14	12	19
Income 3	15,000–30,000	14	19	16	12
Income 4	30,000–50,000	37	42	35	33
Income 5	> 50,000	110	98	105	96
Vehicle ownership (Cars and two-wheelers)					
	0	56	51	49	47
	1	70	66	59	75
	2	37	33	28	42
	3	14	19	21	16
	> 3	2	5	0	2

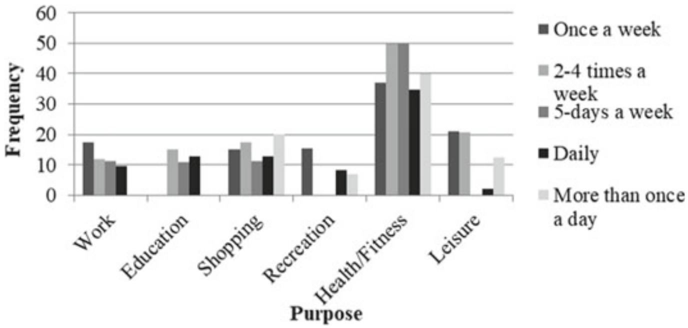


Fig. 3 Distribution of pedestrian trips by purpose

health/fitness (27%). About 26% walked for non-voluntary purposes like work and education, and 14% for recreation/leisure. It is worth mentioning that the survey questionnaire was designed in order to collect NMT trip information related to different trip purposes like work, shopping, etc. Hence, multiple trip purposes include first and last-mile NMT trips which could not be segregated. Income was seen to be instrumental in influencing waking trips as the share of people walking to work was observed to decrease with rising income levels. Share of people walking for health/fitness is higher among higher-income groups.

Travel pattern for pedestrians. Figure 3 presents the distribution in trips by purpose. Observations from this figure show that health/fitness is the dominant trip purpose for which respondents, on an average, walk more frequently and for a longer duration. The next dominant trip purpose was seen to be shopping. However, the average time taken for shopping was observed to be less than that required for health/fitness.

Satisfaction level of individual pedestrian. Table 4 presents the major satisfaction ratings that pedestrians perceived for each of the factors at both the study locations. A majority of the respondents provided ‘average’ satisfaction rating to 9 out of the 14 attributes in Area A, in spite of the location being a planned and newly developed area of Kolkata. It is noteworthy that none of the factors in Area A received a ‘poor’ or ‘very poor’ rating, while the remaining 5 factors were rated as ‘good’. The overall pedestrian environment was rated ‘average’ by majority of the respondents (56%), whereas about 25% rated it ‘good’ or ‘excellent’. 18% felt the overall pedestrian environment of the location was ‘poor’.

In case of Area B, majority of the respondents felt ‘average’ satisfaction with respect to 8 out of the 14 attributes. However, 4 other attributes were either rated ‘poor’ or ‘very poor’ ratings and only the remaining 2 attributes received a majority satisfaction rating of ‘good’. The overall pedestrian environment was rated ‘average’ by majority of the respondents (55%), whereas about 30% rated it ‘poor’ or ‘very poor’. 15% felt the overall pedestrian environment of the location was ‘good’.

Figure 4 shows how the satisfaction ratings are distributed for the 14 factors in Areas A and B. Higher dissatisfaction can be attributed to shade cover, continuity, and

Table 4 Respondents' satisfaction levels toward pedestrian attributes

Attributes	Satisfaction (mode value) Area A	Satisfaction (mode value) Area B
Quality of footpath	4	3
Continuity	3	3
Shade cover	3	2
Directional signs	3	2
Air pollutant levels	3	1 and 2
Street illumination	4	4
Lateral separation	3	3
Vehicular speed	3	3
Comfort in walking	3	3
Crowdedness	3	3
Grade of footpath	3	3
Intersection quality	4	3
Safety during daytime	4	4
Safety during nighttime	4	2
Overall	3	3

vehicular traffic in Area A. Respondents are mostly satisfied with street illumination, intersection, and safety during daytime and nighttime. In Area B, more than 25% of the respondents are dissatisfied with the quality of footpath, continuity, shade cover, directional signage, air pollutant levels, comfort, and crowdedness on facilities. Highest dissatisfaction is attributed to the air pollution levels experienced by them which amounted to 32% of them rating this attribute as 'very poor'.

Heterogeneity in responses. Table 5 depicts the variances in overall satisfaction amongst different socioeconomic subgroups. It can be found by Kruskal Wallis test that except trip purposes, all other groups do not show variability.

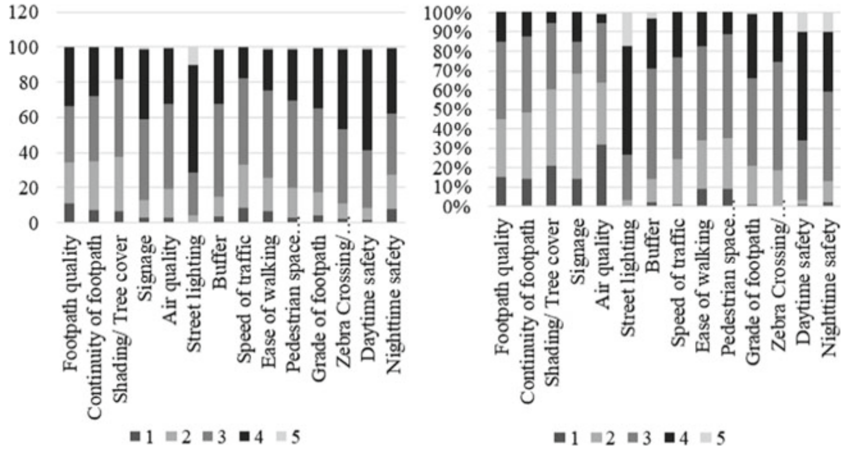


Fig. 4 Percentage of users' with satisfactions levels for **a** Area A; and **b** Area B

Table 5 Subgroups which showed evidence of heterogeneity on overall pedestrian satisfaction

ID	Sub groups	Difference exists (95% CI)	Significance level (Threshold 0.05)
1	Income (1–5)	No	0.603
2	Age (1–5)	No	0.496
3	Gender (0–1)	No	0.849
4	Trip purpose (1–6)	Yes	0.035

5.2 Assessment of Travel Characteristics for Cycle/e-Rickshaw Users

The average distance which people traverse on a cycle/e-rickshaw was observed to be 2.2 km (i.e., average of 16 min). Respondents using cycle/e-rickshaw decreased drastically as distance increased, with approximately 84% of the instances are up to 2 km, whereas the maximum distance traveled is 5 km. Observations on the frequency of using cycle/e-rickshaw showed that a majority of the respondents used these modes once a week (60%) followed by the users who traveled 2–4 times a week (25%). A distinctive majority of the respondents used cycle/e-rickshaw trip for shopping (52%), followed by leisure (16%) purposes. Only about 12% took a cycle/e-rickshaw to work, and 12% for recreational activities. Similar to walking, it was also noted that the share of respondents using cycle/e-rickshaw to work decreases with rising as income levels.

Travel pattern for cycle/e-rickshaw users. Figure 5 shows the distribution in rickshaw trips by purpose. Respondents were observed to use cycle/e-rickshaw more frequently for shopping, followed by leisure trips.

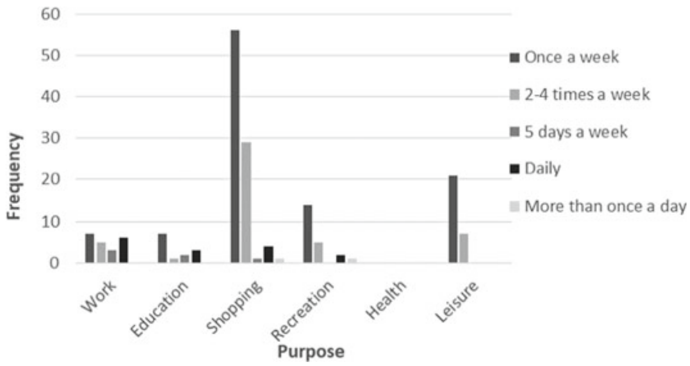


Fig. 5 Distribution in cycle/e-rickshaw trips by purpose

Satisfaction level of individual cycle/e-rickshaw user. Table 6 presents the major satisfaction ratings that the cycle/e-rickshaw users perceived for each of the attributes at the study locations. A majority of the cycle/e-rickshaw users provided an 'average' satisfaction rating to 7 out of the 9 factors in Area A, in spite of the location being a planned and newly developed area of Kolkata. It is noteworthy that none of the factors in Area A were rated as 'poor' or 'very poor', and the remaining 2 factors receiving a 'good' rating. The overall cycle/e-rickshaw environment was rated 'average' by majority of the respondents (41%), whereas about 26% rated it either 'good' or 'excellent'. 10% felt the overall cycle/e-rickshaw environment of the location was

Table 6 Respondents' satisfaction levels toward cycle/e-rickshaw attributes

Attributes	Satisfaction (mode value) Area A	Satisfaction (mode value) Area B
Pavement quality	3 and 4	3
On road traffic volume	3	3
Passenger load	3	3
Pick-up and Drop-off points	3	2
Physical condition of cycle/e-rickshaw	4	3
Availability of cycle/e-rickshaw	4	4
Behavior of drivers/rickshaw pullers	3	3 and 4
Safety during travel	3	3
Fare	3	3
Overall	3	3

‘poor’. In case of Area B, majority of the respondents felt ‘average’ satisfaction with respect to 7 out of the 9 factors, which was similar to observations in Area A. However, only 1 (i.e., pick-up/drop-off point) out of the 9 factors were either rated ‘poor’ or ‘very poor’. Majority of the cycle/e-rickshaw users (38%) recorded an “average” overall satisfaction, 19% either rated with ‘poor’ or ‘very poor’ overall satisfaction, and 27% had a satisfaction rating of ‘good’.

Figure 6 shows how the satisfaction ratings are distributed for the 9 factors in the study locations. Higher dissatisfaction can be observed for factors such as safety during travel, pick-up/drop-off points, and rickshaw fare in Area A. Respondents are satisfied with attributes such as behavior of rickshaw pullers, physical condition of rickshaw, and availability of cycle/e-rickshaw. 25% of the respondents in Area B are dissatisfied with pavement quality, on-road traffic volume, passenger load, safety during travel, and physical condition of the rickshaw. 90% of respondents are highly dissatisfied with the pavement quality that they experience while using bicycle/e-rickshaw in the area and provided a rating of ‘average’ to ‘very poor’.

Heterogeneity in responses. Table 7 indicates the variation in overall satisfaction across different socioeconomic subgroups which is very similar to the pedestrian scenario. It can be found by Kruskal Wallis test that except trip purposes, all other groups do not show variability.

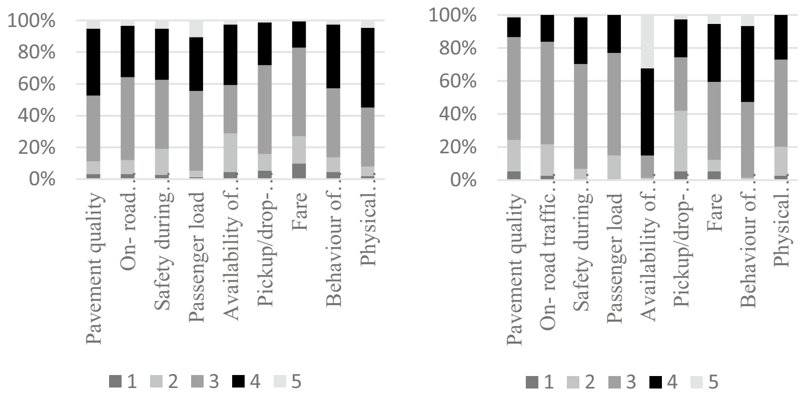


Fig. 6 Percentage of bicycle/e-rickshaw users’ with satisfactions levels for **a** Area A; and **b** Area B

Table 7 Subgroups which showed evidence of heterogeneity on overall rickshaw users’ satisfaction

ID	Sub groups	Difference exists (95% CI)	Significance level (Threshold 0.05)
1	Income (1–5)	No	0.603
2	Age (1–5)	No	0.496
3	Gender (0–1)	No	0.849
4	Trip purpose (1–6)	Yes	0.035

6 Modeling NMT User Satisfaction

6.1 Attributes Impacting Overall Pedestrian Satisfaction

Modeling for pedestrians. 80% of the data collected from both the locations was used for calibrating the OLR model while the remaining 20% used for validation. Table 8 shows the variables significantly (statistically in 95% confidence interval) affecting users' overall satisfaction with the existing pedestrian environment. Those include 'Lateral separation', 'Comfort in walking', 'Intersection quality', 'Safety during Night-time', and 'Continuity'. 'Location' was also found to be significant attribute from the model which indicates that contexuality of the urban fabric influences user satisfaction of pedestrian facilities. In addition to that, the overall satisfaction is significantly lesser in Area B when compared to Area A. The model also depicts that 'Comfort in walking' has the highest impact on overall pedestrian satisfaction, followed by 'Safety during Night-time' and 'Intersection quality'. It is worth mentioning that the OLR model was estimated considering overall pedestrian satisfaction 'Level 5' as the base level which corresponds to the null value for its level-specific constant/intercept value (Refer Eq. 1). Subsequently, the constant/intercept values of each level with respect to 'Level 5' have been indicated in first four rows of Table 8.

The model fit statistics are presented in Table 9 which highlight that the attributes listed in Table 8 help in explaining the overall satisfaction levels and the pseudo-R^2

Table 8 OLR model for pedestrian satisfaction

	Estimate	Std. Error	Sig	95% confidence interval	
				Lower bound	Upper bound
[Satisfaction level while walking = 1.00]	−13.057	1.150	0.000	−10.804	−15.311
[Satisfaction level while walking = 2.00]	−9.094	0.946	0.000	−7.239	−10.949
[Satisfaction level while walking = 3.00]	−5.491	0.811	0.000	−3.902	−7.080
[Satisfaction level while walking = 4.00]	−1.786	1.006	0.076	−0.186	−3.759
Buffer	0.310	0.182	0.021	−0.046	0.667
Ease of walking	0.778	0.177	0.000	0.433	1.124
Zebra crossing	0.506	0.182	0.000	0.379	1.091
Night-time safety	0.509	0.140	0.000	0.217	0.765
Continuity of footpath	0.299	0.158	0.044	−0.073	0.547
Location	−0.701	0.288	0.018	−1.281	0.122

Overall pedestrian satisfaction 'Level 5' acts as base level

Table 9 Model fitting information for pedestrian OLR model

Model	−2 Log likelihood	Chi-Square	Degree of freedom	Significance
Intercept only	412.983			
Final	298.060	114.923	5	0.000
Goodness-of-fit characteristics $R^2 = 0.34$				
Pearson	1296.98	380	0.000	
Deviance	220.08	380	1.000	

values were estimated to be 0.34, which is acceptable for a choice model [46]. In fact, different studies assessing similar perception values have reported R^2 value to be around 0.3 [37, 47] which affirms the explanatory nature of this model. Furthermore, another overall test of fit index ρ^2 comes to be approximately 0.28, which indicates a moderate fit compared to intercept only model [48].

Modeling for cycle/e-rickshaw users. 80% of the data collected from both the locations were used for calibrating the OLR model while the remaining 20% used for validation. Table 10 shows the variables significantly affecting users' overall satisfaction of cycle/e-rickshaw users and the estimation results are largely consistent with previous findings [49]. They include 'On road traffic volume', 'Passenger load', 'Pick-up and Drop-off points', and 'Physical condition of cycle/e-rickshaw', which are all statistically significant at 95% confidence interval. Similar to the previous results presented, 'Location' turned out to be a significant predictor affecting overall satisfaction which reiterates the role of contexuality in an urban setting. The model

Table 10 OLR model for cycle/e-rickshaw users' satisfaction

	Estimate	Std. Error	Sig	95% confidence interval	
				Lower bound	Upper bound
[Satisfaction level while riding = 1.00]	−13.900	1.362	0.000	−11.231	−16.570
[Satisfaction level while riding = 2.00]	−10.431	1.213	0.000	−8.054	−12.809
[Satisfaction level while riding = 3.00]	−6.808	1.074	0.000	−4.702	−8.914
[Satisfaction level while riding = 4.00]	−4.929	1.062	0.000	−2.847	−7.011
On road traffic volume	0.588	0.222	0.008	0.152	1.024
Passenger load	0.543	0.231	0.019	0.090	0.996
Pick-up and Drop-off points	0.552	0.170	0.001	0.219	0.886
Physical condition of cycle/e-rickshaw	0.925	0.191	0.000	0.551	1.299
Location	−0.928	0.315	0.003	0.310	1.545

Overall cycle/e-rickshaw users' satisfaction 'Level 5' acts as base level

Table 11 Model fitting information for cycle/e-rickshaw users' OLR model

Model	−2 Log likelihood	Chi-square	Degree of freedom	Significance
Intercept only	384.932			
Final	291.636	93.297	5	0.000
Goodness-of-fit characteristics $R^2 = 0.32$				
Pearson		678.96	371	0.000
Deviance		231.79	371	1.000

also depicts that 'Physical condition of cycle/e-rickshaw' has the highest impact on overall satisfaction, followed by 'On road traffic volume', 'Pick-up and Drop-off points', and 'Passenger load', respectively. It's worth mentioning that the OLR model was estimated considering overall pedestrian satisfaction 'Level 5' as the base level which corresponds to the null value for its level-specific constant/intercept value (Refer Eq. 1). Subsequently, the constant/intercept values of each level concerning 'Level 5' have been indicated in the first four rows of Table 10.

The model fit statistics are presented in Table 11 which highlight that the above-listed attributes help in predicting the overall satisfaction levels, and the pseudo-R^2 value is estimated to be 0.32 which is acceptable for a choice model. Furthermore, another overall test of fit index ρ^2 comes to be approximately 0.26, which indicates a moderate fit compared to intercept only model [48].

Predicted overall satisfaction levels were compared with the actual satisfaction levels and an accuracy of 61% is reported for pedestrian facilities. A cross-validation procedure was also performed by randomly selecting 80% of the total dataset and the accuracy was found to range between 59 and 68%. Though it is worth mentioning that individual prediction rates for 'Level 2', 'Level 3', and 'Level 4' stay 90–110% but for 'Level 1' and 'Level 5', rates fall sharply to 40–50% which deteriorates overall prediction accuracy. The sharp fall may be attributed to the acute paucity of such responses in the dataset as both even when added account for only 0.02% of total responses. Hence, the model, despite being low in overall accuracy, inspires confidence to a certain degree.

6.2 Improving Level of Satisfaction—ORA Exercise

ORA exercise for the pedestrian environment

The improvement scenarios are ascertained by the predictions in the overall satisfaction generated from the estimated model. Individual independent variables as well as their combinations were modified (i.e., improved) and the change to the overall satisfaction was recorded. It is worth mentioning that the changes in ordinal scale should be perceived in a way that unit change in higher-order is harder to achieve compared to the lower one i.e., it will be easier to achieve a change from 3 to 4 when compared to 4 to 5.

Results for the area 'A'—newly planned urban area. The base case values for this exercise were the user satisfaction ratings received (mode value), as shown in Table 4, for the significant factors. Fifteen (15) different scenario combinations of scenarios were tested against the base case to predict the impact on overall satisfaction. The model predicts (refer Table 12) that in 14 out of 15 possible scenarios, improvement to any particular of the 4 out of 5 factors identified in Sect. 6 i.e., 'lateral separation', 'Comfort in walking', 'Intersection quality', and 'Safety during nighttime' results in increased user satisfaction (change from the current level of 3–4). However, for the case of 'continuity', improvements made by one-unit increase in the user satisfaction is not likely to bring about any improvement in the overall satisfaction. Simultaneous improvements in multiple attributes were also noted in this analysis in order to record the combination of attributes which is likely to improve the overall level of satisfaction of the users. Interestingly, the multiple improvements bring similar change to user satisfaction, as do individual improvements.

Results for area 'B'—old and organically developed urban area. Table 13 suggests that synchronized enhancement to only 2 out of 5 factors i.e., 'Lateral separation', and 'comfort in walking' provides increased overall user satisfaction. Other scenarios of individual changes in user satisfaction were improbable in bringing about a change to the overall satisfaction.

Table 12 Area A: predicted improvement overall satisfaction (cases only where changes in predicted overall satisfaction have been identified)

Combination	Lateral separation	Comfort in walking	Intersection quality	Safety during nighttime	Continuity	Predicted
Base case	3	3	4	4	3	3
1	4	3	4	4	3	4
2	3	4	4	4	3	4
3	3	3	5	4	3	4
4	3	3	4	5	3	4
5	4	4	4	4	3	4
6	4	3	5	4	3	4
7	4	3	4	5	3	4
8	4	3	4	4	4	4
9	3	4	5	4	3	4
10	3	4	4	5	3	4
11	3	4	4	4	4	4
12	3	3	5	5	3	4
13	3	3	5	4	4	4
14	3	3	4	5	4	4

Table 13 Area B: predicted improvement overall satisfaction (cases only where changes in predicted overall satisfaction have been identified)

Combination	Lateral separation	Comfort in walking	Intersection quality	Safety during nighttime	Continuity	Predicted
Base case	3	3	4	4	3	3
1	4	4	4	4	3	4

Table 14 Area A: predicted improvement overall satisfaction (cases only where changes in predicted overall satisfaction have been identified)

Combination Id	On road traffic volume	Passenger load	Pick-up and drop-off points	Physical condition of cycle/e-rickshaw	Predicted
Base case	3	3	3	4	3
1	3	3	3	5	4
2	4	4	3	4	4
3	4	3	4	4	4
4	4	3	3	5	4
6	3	4	4	4	4
7	3	4	3	5	4
8	3	3	4	5	4

ORA exercise for cycle/e-rickshaw environment

Results for area 'A'—newly planned urban area. The model predicts (refer Table 14) that individual improvement to only 1 i.e., 'Physical condition of cycle/e-rickshaw' out of 5 factors identified in Sect. 6 is likely to result in increased user satisfaction, (change from the current level of 3–4). However, for this to happen, the user satisfaction of the physical condition of cycle/e-rickshaw has to improve to 5, which may be difficult to achieve. Hence, synchronized improvement combinations that result positively in increasing overall satisfaction level maybe implemented in Area B.

Results for area 'B'—old and organically developed urban area. The model predicts (refer Table 15) that the scenario of cycle/e-rickshaw users are slightly different from pedestrians in Area B as only 2 out of 4 individual factor improvements seem able to better the overall satisfaction. It is observed that 4 out of 6 possible synchronized improvement combinations provide increased overall user satisfaction.

Findings of the Research

This research was able to create an integrated two-step method, which—(a) predicts the level of satisfaction of NMT users; and (b) determines the impacts of individual

Table 15 Area B: predicted improvement overall satisfaction (cases only where changes in predicted overall satisfaction have been identified)

Combination Id	On road traffic volume	Passenger load	Pick-up and drop-off points	Physical condition of cycle/e-rickshaw	Predicted
Base case	3	3	3	4	3
1	4	3	3	4	4
2	4	3	4	4	4
3	3	4	3	4	4
4	3	4	4	4	4
6	3	3	3	5	4
7	3	3	4	4	4

attributes and their combination thereof, that lead to an enhancement in overall satisfaction levels. The study finds that the NMT trips in Kolkata range from anywhere between 1.4 and 2.2 km, with a majority of users availing NMT modes for shopping or fitness trip purposes. Presented below are the mode-specific findings:

Pedestrian satisfaction findings. A majority of pedestrians in Kolkata walk for health reasons, and they do so almost every day of the week. When it comes to pedestrians' satisfaction of the existing conditions, it was observed that users in the newer study area (Newtown) are slightly more satisfied than the ones in the older study area (Jadavpur). Majority of users have provided a rating of 4 out 5 to street lighting and daytime safety, whereas users in the older study area are concerned about air quality while walking, shading/tree cover, signage, and nighttime safety, with all of them receiving a rating of 2 out of 5. The modeling results show that five different factors significantly impact the overall satisfaction of pedestrians, which is significantly different between the two areas in Kolkata. The factors of 'lateral separation' and 'comfort in walking' have the greatest impact in predicting overall satisfaction, whereas 'continuity' although impacts a pedestrian's overall satisfaction, but an improvement in only that factor is improbable in bringing about any improvement to the overall satisfaction of pedestrians.

Rickshaw user satisfaction findings. Rickshaw users' are predominantly utilizing the mode for shopping purposes, and they are doing so mostly only once a week. When it comes to their satisfaction of the existing conditions, it was observed that users in the newer study area (Newtown) are slightly more satisfied than the ones in the older study area (Jadavpur). Majority of users have given a rating of 4 out 5 to the availability of rickshaws, whereas the pick-up/drop-off points have been rated 'poor' by the users in the older study area (Jadavpur). The modeling results show that four different factors significantly impact the overall satisfaction of rickshaw users, which is significantly different between the two areas in Kolkata. The 'physical condition of rickshaw' has the greatest impact in predicting a rickshaw user's overall satisfaction.

Finally, when it comes to improving the overall satisfaction of rickshaw users, several improvements might have to be undertaken simultaneously, as improvement in only a single factor is improbable in bringing about any improvement in overall satisfaction in majority of the scenarios.

7 Conclusion

Based on these findings, the major conclusions that can be drawn from the current research are:

Variability in user satisfaction. The study shows that there exist two different types of variability in users' perception (a) Variability in individual factor and overall satisfaction ratings; (b) Variability in user ratings of different areas within a city. It may not be sufficient to only know how a user perceives about individual factors during his/her NMT trip, as these factors may or may not have a bearing on the users overall trip satisfaction. The study shows that while 'air quality' and 'shading/tree cover' received lower satisfaction ratings from the pedestrians, but the modeling results show that 'ease of walking' is the factor that has the most significant impact on predicting overall pedestrian satisfaction. Similarly, one citywide survey of user perception may not capture the true needs of the users. The model developed in the study shows that a significant difference exists in user perception of residents of a newer versus an older area within a city. However, this could not have been deciphered analyzing only the individual user ratings, as most of the factors received a rating of 3 out of 5 in both the areas. Thus, this study provides a technically sound template that could be used by decision-makers to gauge the perceptions of NMT users in a large metropolitan area.

Targeted/coordinated improvements to enhance satisfaction. The study shows that pedestrians walk primarily for fitness purposes, whereas rickshaws are largely used for shopping trips. Hence, when it comes to providing/improving NMT facilities, decision-makers would be well-advised to target residential areas, parks, etc. for pedestrian facilities, where people may walk for fitness and provide for sufficient quality rickshaw facilities at the commercial/shopping areas. Secondly, the study also shows that improvement in individual factors seldom causes any improvement in the overall satisfaction. For example, an improvement in 'pick-up/drop-off points' of rickshaws in Newtown, Kolkata alone is unlikely in improving the overall user satisfaction of rickshaw users, but when combined with an improvement in 'passenger load' is likely to improve the overall satisfaction. Hence, care has to be taken as to what type of improvement(s) have to be implemented and where in order to achieve an improvement in user perception.

Proactively provide quality NMT environment in upcoming development projects. The study shows that the perception of NMT users in the older neighborhood is significantly different i.e., lower than that of the NMT users in the older area. This could be because of the facilities being under a poor state of maintenance in the older neighborhood when compared to the facilities in the newer area. Improving pedestrian facilities, for example, in an older area may be difficult due to lack of right-of-way and/or expensive due to various factors such as utility relocation, etc. Another factor that is highlighted in the study is that it is a challenging task to alter the perception of residents in an older area when compared to a newer one. For example, improvement under only one scenario i.e., joint improvement in 'comfort in walking' and 'lateral separation', is likely to bring about an enhancement of overall pedestrian satisfaction in the older area, whereas several scenarios of individual and joint improvements are likely to bring an enhancement of overall pedestrian satisfaction in the newer area. As such, decision-makers are well-advised to proactively provide for sufficient quality NMT infrastructure while developing new residential/commercial areas, which could potentially allow for positive user perception to be developed, and also avoid building the NMT infrastructure in a retrograde manner, which could be difficult and expensive.

Acknowledgements The authors appreciate the opportunity provided by Newtown Kolkata Development Authority (NKDA), West Bengal, and Ministry of Education, Government of India, for funding this study. The authors would also like to thank the members of 'Multimodal Urban Sustainable Transportation' (MUST) research group at IIT Kharagpur for their assistance during the survey tasks.

References

1. Kamath N, Saurav S (2016) Handbook of research on strategic supply chain management in the retail industry
2. Boorish A et al, Word list 2. Word J Int Linguist Assoc 1–5
3. Kirby A (2008) Kick the habit—a UN guide to climate neutrality. United Nations Environment Programme, Malta
4. Sridhar KS (2010) Impact of land use regulations: evidence from India's cities. Urban Stud 47(7):1541–1569
5. W. S. Associates (2007) Public attitude survey of bicycle and pedestrian planning Wilbur Smith Associates
6. G. of I. Ministry of Urban Development (2014) National Urban Transport Policy
7. Rahul TM, Verma A (2013) Economic impact of non-motorized transportation in Indian cities. Res Transp Econ 38(1):22–34
8. Tiwari G, Jain D (2013) NMT infrastructure in India: investment, policy, and design
9. Motor Vehicles—Statistical Year Book India (2017) Ministry of Statistics and Programme Implementation, Government of India. Available: http://mospi.nic.in/statistical-year-book-india/2017/189
10. Mahadevia D, Planning E (2014) Walking and cycling in Indian cities a struggle for reclaiming road edges walking and cycling in Indian cities struggle for claiming road edges, no. September 2012
11. Kurosaki T (2012) Urban transportation infrastructure and poverty reduction: Delhi Metro ' s impact on the cycle rickshaw rental market
12. Roy R, Hoque MS (2016) Cycle-rickshaw as a sustainable mode of public transport in Dhaka city, pp 1–9

13. Kang L, Xiong Y, Mannering FL (2013) Statistical analysis of pedestrian perceptions of sidewalk level of service in the presence of bicycles. Transp Res Part A Policy Pract 53:10–21
14. Fujiyama T, Childs C, Boampong D, Tyler N (2005) Investigation of lighting levels for pedestrians. In: International Conference on Walk 21st Century, pp 1–14
15. Kim S, Choi J, Kim Y (2011) Determining the sidewalk pavement width by using pedestrian discomfort levels and movement characteristics, vol 15, pp 883–889
16. Randall AT, Baetz WB (2001) Evaluating pedestrian connectivity for suburban sustainability. J Urban Plan Dev 127(1):1–15
17. Tyrinopoulos Y, Antoniou C (2008) Public transit user satisfaction: variability and policy implications. Transp Policy 15(4):260–272
18. St-louis E, Manaugh K, Van Lierop D, El-geneidy A (2014) The happy commuter: a comparison of commuter satisfaction across modes, pp 160–170
19. Ettema D, Gärling T, Olsson LE, Friman M, Moerdijk S (2013) The road to happiness: measuring Dutch car drivers' satisfaction with travel. Transp Policy 27:171–178
20. Páez A, Whalen K (2010) Enjoyment of commute: a comparison of different transportation modes. Transp Res Part A 44(7):537–549
21. Zheng X et al (2016) Big data for social transportation. IEEE Trans Intel Transp Syst 17(3):620–630
22. Rissel C, Crane M, Wen LM, Greaves S, Standen C (2018) Satisfaction with transport and enjoyment of the commute by commuting mode in inner. Health Promot J Austr 27(1):1–2
23. Olsen JR, Macdonald L, Ellaway A (2017) Changes over time in population level transport satisfaction and mode of travel: a 13 year repeat cross-sectional study, UK. J Transp Heal 6(December 2016):366–378
24. Bian Y, Ma J, Rong J, Wang W, Lu J (2009) Pedestrians' level of service at signalized intersections in China. Transp Res Rec J Transp Res Board 2114(1):83–89
25. Tan D, Wang W, Lu J, Bian Y (2007) Research on methods of assessing pedestrian level of service for sidewalk. J Transp Syst Eng Inf Technol 7(5):74–79
26. Koppelman FS, Bhat C (2006) A self instructing course in mode choice modeling: multinomial and nested logit models by with technical support from table of contents. Elements 28(3):501–512
27. Sanko N (2001) Guidelines for stated preference experiment design
28. Kadali BR, Vedagiri P (2016) Review of pedestrian level of service: perspective in developing countries. Transp Res Rec J Transp Res Board 2581(1):37–47
29. Gopalakrishnan A, Estate I (2018) Quantitative approach to determine pedestrian delay and level of service at signalized intersection. Int J Students' Res Technol Manag 7(1):6–13
30. Papaioannou D, Martínez LM (2016) Measuring satisfaction with transit and car trips with use of one logistic regression model. Transp Res Rec J Transp Res Board 2543(1):101–107
31. Bordagaray M, Ibeas A, dell'Olio L (2012) Modeling user perception of public bicycle services. Procedia Soc Behav Sci 54:1308–1316
32. Laxman KK, Rastogi R, Chandra S (2010) Pedestrian flow characteristics in mixed traffic conditions. J Urban Plan Dev 136(1):23–33
33. Zainol R, Ahmad F, Nordin NA, Aripin AWM (2014) Evaluation of users' satisfaction on pedestrian facilities using pair-wise comparison approach. IOP Conf Ser Earth Environ Sci 18(1)
34. Choi J, Kim S, Min D, Lee D, Kim S (2016) Human-centered designs, characteristics of urban streets, and pedestrian perceptions, no. July 2015, pp 120–137
35. Kim S, Park S, Lee JS (2014) Meso- or micro-scale? Environmental factors influencing pedestrian satisfaction. Transp Res Part D Transp Environ 30:10–20
36. Bivina GR, Parida P (2018) Pedestrian level of service model for evaluating and improving sidewalks from various land uses
37. Rankavat S, Tiwari G (2016) Pedestrians perceptions for utilization of pedestrian facilities—Delhi, India. Transp Res Part F Traffic Psychol Behav 42:495–499
38. Çelik AK, Senger Ö (2016) A comparison of alternative ordered response models for analysing user satisfaction with transport services in Turkey. Transp Telecommun J 17(1):40–54

39. Eboli L, Mazzulla G, Eboli L, Mazzulla G (2013) An ordinal logistic regression model for analysing airport passenger satisfaction
40. Huntsinger LF et al (2013) Trip generation models using cumulative logistic regression. J Urb Plann Dev 139:176–184
41. Dixon LB (1996) Bicycle and pedestrian level-of-service performance measures and standards for congestion management systems. Transp Res Rec 1538:1–9
42. Khisty CJ (1994) Evaluation of pedestrian facilities. Transp Res Rec 1438:45–50
43. Cheng EWL, Li H (2002) Construction partnering process and associated critical success factors: quantitative investigation. J Manag Eng 18:194–202
44. Bernhoft IM, Carstensen G (2008) Preferences and behaviour of pedestrians and cyclists by age and gender. Transp Res Traf Psychol Behav 11:83–95
45. Lipsitz SR, Fitzmaurice GM, Regenbogen SE, Arbor A, Ibrahim JG, Hill C (2014) Model with application to a study of surgical complications. J Roy Stat Soc 62(2):233–250
46. de Dios Ortuzar J, Willumsen LG (2011) Modelling transport. Wiley, Hoboken
47. Chakrabarti S (2017) How can public transit get people out of their cars? An analysis of transit mode choice for commute trips in Los Angeles. Transp Policy 54(October 2016):80–89
48. de Dios Ortuzar J, Willumsen LG (2011) Modelling transport, 4th edn. Wiley, Hoboken
49. Bhaduri E, Bs M, Sen J, Maitra B, Goswami AK (2019) Measuring user satisfaction of pedestrian facilities and its heterogeneity in urban India—a tale of three cities. J East Asia Soc Transp Stud 13:144–171
50. Kleinbaum DG, Klein M, Statistics for biology and health
51. Nag D, Bhaduri E, Kumar GP, Goswami AK (2020) Assessment of relationships between user satisfaction, physical environment, and user behaviour in pedestrian infrastructure. Transp Res Procedia 48:2343–2363
52. Krejcie RV, Morgan DW (1970) Determining sample size for research activities. Educ Psychol Meas 38:607–610

Lateral Placement of Vehicles Under Mixed Traffic in Indian Urban Scenario

Mahindra Deshmukh and Ashish Dhamaniya

1 Introduction

India being a developing country with population around 1.25 billion. The vehicular ownership is also increasing considerably each year. Sub-urban and urban regions are the places where most of the population is attracted. The Central Business District (CBD) zones of the city and the arterials connecting CBD's to the other zones of city, face a large congestion during peak and even at off peak hours. Vehicles not only interact longitudinally, but also laterally to each other with variable speeds and acceleration. No lanes of traffic are observed, neither there is any lane discipline followed by the vehicles. In context to driving, lane discipline is when a driver keeps his/her vehicle within the road marking indicating lane unless changing lanes or direction [6]. Such lane following scenario is quite unseen in Indian traffic conditions. It is really a difficult task to maintain a lane with a maintained speed, as vehicles interaction leads the vehicles to change the longitudinal as well as lateral position with respect to others. This ultimately results into a poor lane behaviour with higher degree of heterogeneity. The speed of vehicle is likely to affect with regard to its lateral position on the roadway geometry. Lennie and Bunker [5] stated that the lateral position of vehicle is the distance between the carriageway centreline and the nearest edge of the vehicle. *Highway {C}apacity {M}anual* (2012) states that the lane widths less than 3.6 m reduce the travel speeds, and width larger than this value are not considered to increase speed above the base level. Whether the vehicle is in the centre lane, either curb side or the inner lane, the observed speed of vehicle is obviously dissimilar. Widespread range of physical dimensions, power to weight ratio, and

M. Deshmukh
Global Traffic Solution, Pune, India

A. Dhamaniya (✉)
Department of Civil Engineering, Sardar Vallabhbhai National Institute of Technology, Surat, India

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_32

other dynamic characteristics of vehicles lead to higher degree of heterogeneity, with mixed traffic. There exists a mixed traffic flow with no lane discipline. Hence the lateral movement of the vehicles are very high due to no lane discipline. Hence [6] studied the lateral movement of vehicles on straight road. Two wheelers and three wheelers are observed with the most reckless driving. Two wheelers accept the minimum available gap to overtake the other vehicles, by utilising the overall width of carriageway uses the same roadway space resulting in heterogeneous traffic. The bi-directional traffic on some roads utilises the same road space regardless of any physical segregation and occupies any lateral position on the road contingent on the availability of space. This forces the other vehicles to slower down their speeds. As observed, different types of vehicles share the same road space available without any lane discipline. Bunker et al. [1] showed that a maximum 7 s time gap between opposing vehicles influences drivers' positioning of their vehicles. It was observed that the lateral positions of cars, utility vehicles, and semi-trailers are statistically different when opposed as compared to unopposed. Non-motorised vehicles share the same right of way along with the other passenger cars and heavy vehicles causing congestions and bottlenecks. Pal and Mallikarjuna [8] stated that drivers vary gaps based on the varying traffic conditions. The extent of vehicular interactions varies with the composition of traffic. As a result, causing delay and ultimately reduction in the capacity. Hence proper study of lateral placement of vehicles has become a challenging task, especially in developing countries like India.

Microsimulation is one of the techniques used to model the traffic and study its characteristics. Simulation is widely used tool in many industries for testing or inspecting the validity and reliability of the proposed methodology. Srikanth et al. [10] used the speed flow curves to calibrate the VISSIM model for multilane highways. Transportation industry is not an exception for the application of simulation technique as a use for problem solving. Microsimulation and macrosimulation tools are popular to study and inspect the traffic trends, behaviour and demand. Parking studies, feasibility studies at toll plaza, public transit stations for pedestrians and users, etc. are the applications of microsimulation modelling extensively used in transportation industry. Using computer-based simulation various traffic characteristics like speed, flow, density etc. can be studied. Though microsimulation is widely used to model homogeneous traffic in developed countries, it is also widely used in the developing countries in Asia to model the heterogeneous traffic. Manjunatha et al. [7] proposed a methodology to calibrate the mixed traffic using a microsimulation model. To model the heterogeneous traffic, non-lane following behaviour is not the only challenge, but the speed and acceleration characteristics of a number of vehicle categories, their power to weight ratio, dimensions, etc. are also to be taken into consideration. Hence to study the lateral behaviour of the vehicles, microsimulation software VISSIM is used in the present study. Kaur [4] modelled the heterogeneous traffic using VISSIM. The calibration of the selected study stretch is done using speed, flow, density curves and further the relation of speed with lateral clearance is compared. The coordinate data of vehicles from the VISSIM software is used to find out the lateral gaps between the different vehicle categories. VISSIM is the state-of-the-art multi-modal simulation software developed by PTV AG, Germany,

Fig. 1 Study corridor site picture taken from the foot over bridge

popularly used to model both homogeneous and heterogeneous traffic. To reflect the field conditions with the simulation model, calibration is done for the selected data set. Drivers are forced to accept the gap as per the prevailing traffic conditions. Figure 1 outlines the site picture taken from the corridor, with a view of the study stretch.

Number of vehicle classes comprising different dimensions, speed and acceleration characteristics, and occupancy characterise the Indian traffic. Not only motorised but also non-motorised vehicles share the same right of way with other vehicles. These slow moving non-motorised vehicles followed by the reckless driving behaviour of three-wheelers and motorised two-wheelers force the passenger cars to slow down their speeds. Two-wheeler drivers accept the minimum available gap to surpass the vehicles, which also leads to diamond shaped queuing at intersections. Curb side bus stops are cause for the bus to wait until the passenger's board/alight, which forms a bottleneck on the urban roadways. Roadside parking's, frequent stopping of three-wheeler drivers at curb side, to board or alight more passengers for more fare, ultimately results into a mixed traffic condition with poor lane following behaviour. The degree of heterogeneity is hence observed more in case of developing countries than that of the developed countries, which are having proper lane following behaviour [9]. Hence to study such a complex and mixed traffic behaviour microsimulation technique might present a faithful outcome for obtaining the desired results. Hence, prior data is required to be collected for erecting successful simulation model with minimal errors.

2 Data Collection and Analysis

The study stretch considered was free from U-turning or crossing movements of vehicles, there was not any direct access for vehicles to enter, merge or diverge near the study stretch, the stretch was fairly straight and of uniform width throughout. The care of side friction, which is often observed in urban intercity roads was also taken in consideration that, there were no curb side parking's at the stretch. These objectives were defined for selection of the study section; as pure interactions of the operating vehicles are to be studied. Also, the selected study stretch was free from gradient (firm) and curvature. The urban roads also often face problems like undesignated crossings of pedestrians. To encounter the issues related to pedestrians, as discussed in the previous section, the study stretch was selected near a foot-over bridge (FOB) facility for pedestrians.

By virtue of the FOB and divided roadway, the pedestrians were not observed to cross on the road section. Video graphic method of data collection was used to capture the wide-ranging variation of the vehicular interactions. Outer ring road from New Delhi was selected for the data collection purpose. Over the selected urban intercity arterial road, a 30 m section was marked for studying the interactions of vehicles. A wide lens high pixel resolution camera was used. The camera was placed on the edge of FOB in order to cover all three lanes (one direction) of traffic clearly. Figure 1 shows the Google image of the location of the study stretch. It is located at the Pamposh Enclave, greater Kailash, New Delhi. The collected data from the field was then analysed by Avidemux Software with high precision, having ten further parts (microseconds) of one second, for more accurate speed data of individual vehicles. The speed of 30-m study stretch was calculated for each class of vehicle, and this speed was considered as the average speed of particular vehicle class over the mid-block section of the arterial. The data was collected for 2 h duration. Out of various categories of vehicles were observed from the video, which were converted into five main categories for simplicity in the analysis. The categories of vehicles defined in the present study were namely two-wheeler (2W), three-wheeler (3W), small car (SC), big car (BC) and heavy vehicles (HV). The proportion and the average dimensions considered in the present study are as shown in Table 1. The dimensions for the set of vehicles which were observed while data collection was extracted from the

Table 1 Average dimensions and proportions of vehicles

Vehicle type	Proportion (%)	Average dimensions		
		Width (m)	Length (m)	Area (m^2)
2W	30.96	0.85	2.00	1.70
3W	9.56	1.08	2.36	2.55
SC	47.83	1.69	3.89	6.57
BC	7.73	1.86	4.61	8.56
HV	3.92	2.83	10.88	30.79

official websites of respective motor vehicles. These dimensions were used to model the vehicles in the simulation model as well, in order to integrate more accuracy to vehicular dimensions from the simulation model and field.

The two-wheeler was composed of all motorised two-wheeler vehicles (excluding bi-cycle). Three-wheelers consists of motorised three-wheeler vehicles often observed in the urban regions. The car traffic was divided into two main classes. SC consists of sedan, prime-sedan, hatchback cars moderately having the engine capacity up to 1000–1200 horsepower. BCs consists of sport utility vehicles (SUVs) having more occupancy and engine capacity as compared to SCs. The length of BC vehicles is also comparatively more than those of SC. Lastly, buses, trucks, tempos, multi-axle vehicles and containers were converted into a single HV category of vehicles, as their speed and acceleration characteristics are nearly similar.

3 Capacity of Roadway Section

The capacity of a system can be defined as the number of vehicles, passengers or the like, can accommodate a system under the prevailing roadway and traffic conditions. Capacity is independent of demand. It is constant, irrespective of the total number of vehicles demanding the service. Dhamaniya and Chandra [3] stated that the lane capacity of was soundly related to the operating speed of the vehicles in the stream. In the present study, the capacity of roadway is calculated using the Greenshields model. Figure 2 refers the capacity of the study stretch and it was found to be 5957 veh./h and 6034 PCU/h.

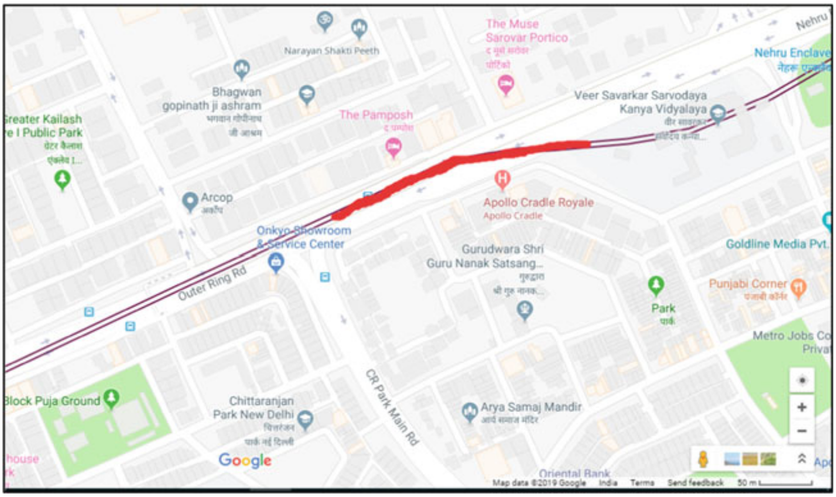


Fig. 2 Study stretch

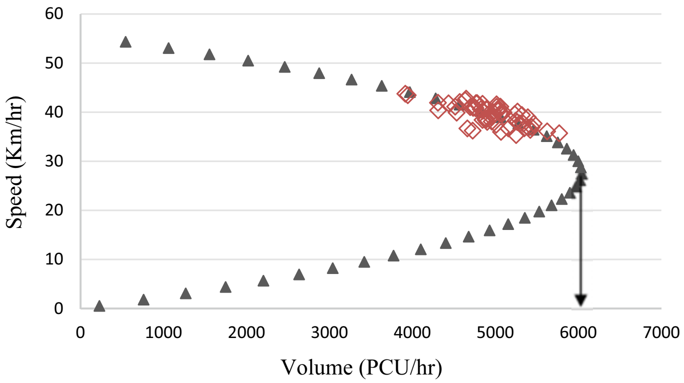


Fig. 3 Capacity of the roadway

Capacity of a roadway is a probabilistic approach and may vary with time and position. It is difficult to derive capacity analytically, hence in most of the practices, capacity is calculated from the field observed conditions.

4 Methodology for Calibration

The collected data was used as input for the simulation model development. The Calibration is progression to refine the model to replicate the observed data to an adequate level of accuracy (with negligible errors). The main aim of calibration is to satisfy the model objective to replicate the field conditions within the developed model. In the present study calibration was done by adjusting the driver's behaviour parameters which are most important to modify the output of the model. VISSIM is by default a homogeneous lane-based traffic tool, developed by PTV-AG, Germany. It is based on the traffic conditions that observed in developed countries, especially from European and American continent. However, a heterogeneous non-lane-based traffic can also be modelled by a rigorous modification in drivers' behaviour parameters. This flexibility of VISSIM has mainly achieved worldwide popularity for simulating the traffic. Many researchers used VISSIM to model the heterogeneous traffic, mainly from the developing countries including India. The flow chart for calibration is as shown in Fig. 3.

5 Calibration of Speed Data

Speed being the qualitative measure of traffic, it was considered as the primary parameter to calibrate for the simulation model being developed. Various percentile

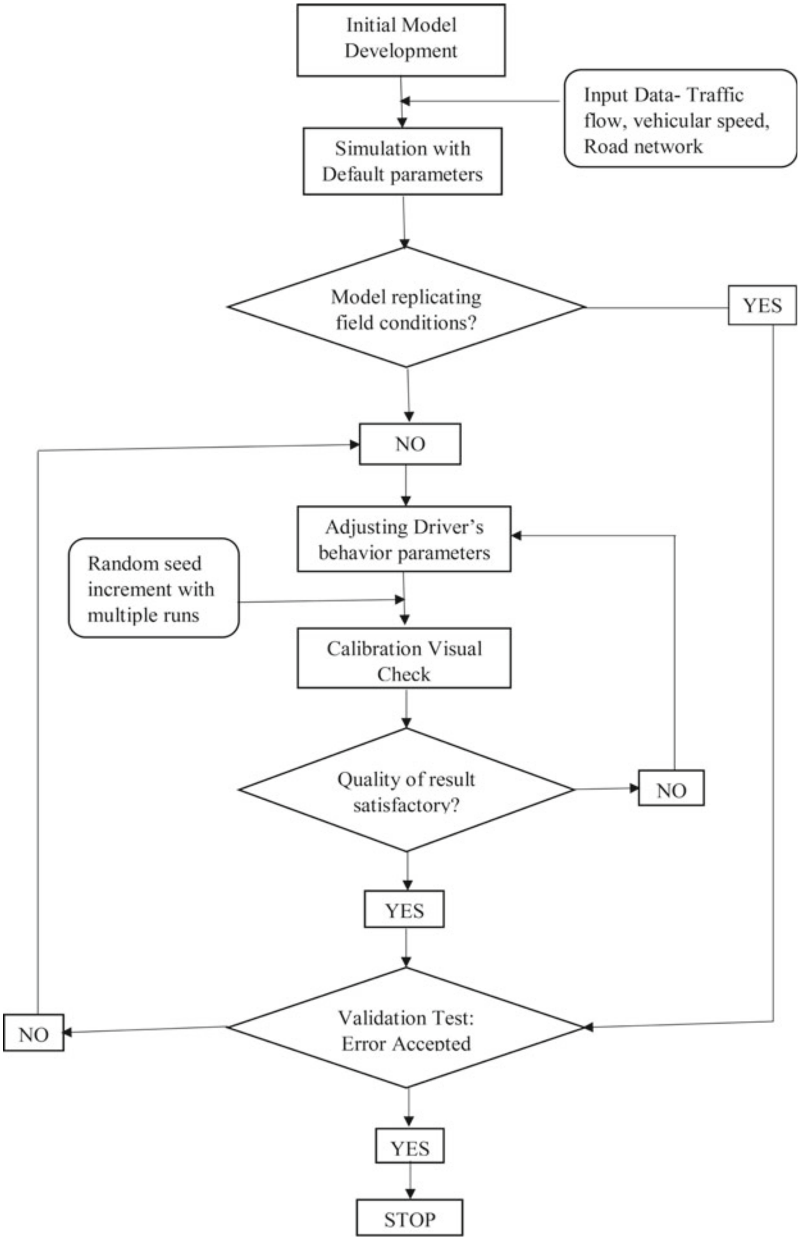
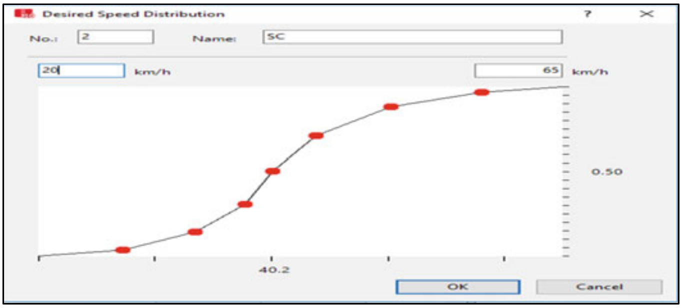


Fig. 4 Methodology flowchart of calibration

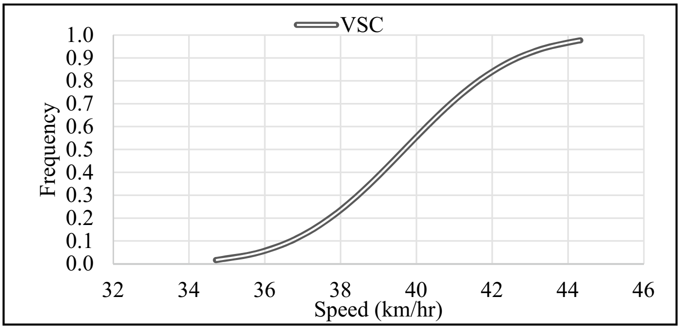
values of speed were calculated, and used as input for the desired speed distribution curve in the VISSIM interface, for each category of vehicle. The speed distribution of vehicles in the simulation model will replicate more precisely with the field data, if a greater number of percentile speed values are given to the speed distribution curve. Figure 5 shows the speed distribution for SC. The respective fractional values of cumulative frequency of speed were used to obtain the percentile value to be input in the desired speed distribution curve of vehicles in VISSIM, hence a separate statistical chi-squared test is not required.

The most basic and fundamental parameters of traffic are considered to be the speed, density and the volume. Hence, in the present study the comparison between the field observed and simulation results was done by associating the degree of similarity between the speed, density and volume of traffic of both. In the subsequent part of the study, the comparison of speed-flow and flow-density is also presented.

From Fig. 6a, b it can be observed that the results obtained from the simulation model are nearly matching with those of the field observed data values. This comparison was checked graphically, but to check for more accuracy and reliability of the simulation model results, (Analysis of Variance) ANOVA test for the three fundamental parameters speed, density and volume.

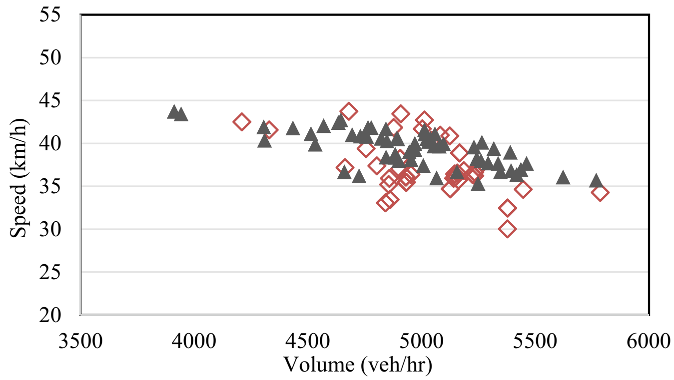


(a) Desired Speed Distribution used in VISSIM

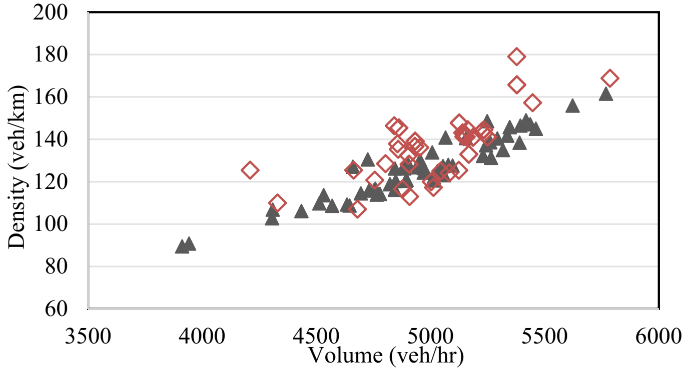


(b) CDF for V_{SC}

Fig. 5 Speed data calibration



(a) Comparison of field-observed and simulated speed-flow relationship



(b) Comparison of field-observed and simulated Volume-density relationship

Fig. 6 Validation of simulated data with field data

The ANOVA test was performed between the field observed data set, and the same data set obtained from the simulation model, for one-hour duration each. If there is any significant difference within the dataset is checked using the test. The Null Hypothesis was considered as "There is no significant difference between the two datasets", and the Alternate Hypothesis was "There is significant difference between the two datasets". Table 2 shows the results of ANOVA test.

Table 2 ANOVA test results for field observed and simulated data

	P-value	P-crit.	$P > P$-crit.	F-value	F-crit.	$F < F$ crit.
Flow (Q)	0.3568	0.05	Yes	0.856	3.921	Yes
Density (K)	0.8607	0.05	Yes	0.031	3.991	Yes
Speed (V)	0.0884	0.05	Yes	2.969	3.949	Yes

From the ANOVA test results table, it can be seen that the P-value obtained for flow, density and speed is 0.3568, 0.8607 and 0.0884. All of these three P-values are greater than the *P*-critical value, which is 0.05 at 5 degrees of freedom. From this, it can be stated that there is no significant difference is observed between the field observed values and simulation model values of speed, flow and density. Hence, the model is said to represent the field observed vehicular traffic conditions (with negligible errors). As, the basic fundamental quantities of traffic, on which the overall performance of the roadway system can be judged with volume to capacity ratio, delay, etc. can be derived from these three basic quantities. Hence, these three quantities were considered for the model calibration purpose.

The calibrated simulation model was used to simulate the various volume levels for the study stretch. The vehicular volume ranging from LOS-C (moderately low traffic) to LOS-E (Dense Traffic). The volume at various LOS levels were calculated based on the V/C ratio criteria, as capacity value was already obtained for the study stretch.

To represent the volume or capacity of a system in context to number of vehicles per unit time cannot be an appropriate method of representation. As, the combined effect of 2W in numbers is completely different as compared to numbers of HV. Hence, in the present study the capacity of the study section is converted into equivalent number of PCU/h. The method proposed by [2] was used to find the PCU values of vehicles.

$$\mathrm{PCU} = \frac{V_c/V_i}{A_c/A_i} \tag{1}$$

where, V_c, V_i is the speed of passenger car (SC in the present study) and subject vehicle respectively, and A_c, A_i is the projected rectangular area of the passenger car and subject vehicle, respectively. As SC is considered as the standard passenger vehicle, all other class of vehicles were converted into the equivalent number of SC. PCU of a vehicle type is very dynamic in nature and it has impact of all variables that affect the driving behaviour of the vehicles. Hence, the PCU for each class of vehicle was calculated and the overall throughput of traffic stream was measured in PCU/h. Based on the various volume levels, the Passenger car unit (PCU) values was calculated. The variation of PCU value at various LOS level was studied. Table 3 shows the variation of PCU with respect to volume and V/C ratio.

Table 3 Variation of PCU values

Volume	V/C ratio	Passenger car unit (PCU)				
		2W	3W	SC	BC	HV
5350	0.90	0.24	0.42	1.00	1.30	6.08
4750	0.80	0.25	0.43	1.00	1.25	5.87
4200	0.71	0.26	0.43	1.00	1.23	5.76

To check the reliability of the calculated PCU values for the developed simulation model, the observed composition of heterogeneous traffic was made run for one hour into the developed simulation model with the previous volumes, obtained from the various V/C ratios considered for the study. Further, this traffic was converted into the PCU/h traffic using the PCU values obtained for specified vehicle class at particular volume. The standard dimensions of field observed set of vehicles, those obtained from the official websites of the respective vehicles were used to model the dimensions of vehicles in VISSIM. Accompanying to this, for same volumes considered, the total vehicle input given to the simulation model was of only SC, i.e. the standard passenger vehicle considered for the study. The results obtained from the equivalent PCU/h volume and the SC only volume from the simulation were compared. The t-test was applied to the obtained results. The results obtained for one-hour simulation, i.e. for sixty data points were considered for the t-test. The t_0 value obtained was 3.08 and the critical value was 5.73 which is greater than the t_0, hence it proves that there is no significant difference between the two data sets. Hence the simulation model and the PCU values both are said to be valid and reliable, at specified volume levels. Figure 7 shows the comparison of the SC only data and PCU/h data of the volume, obtained at various volume levels.

Figure 7 describes about the similarity of the calibrated data of SC only traffic, with the equivalent mixed traffic in the simulation model. Which implies that the simulation model is capable of replicating the field conditions effectively with minimal errors.

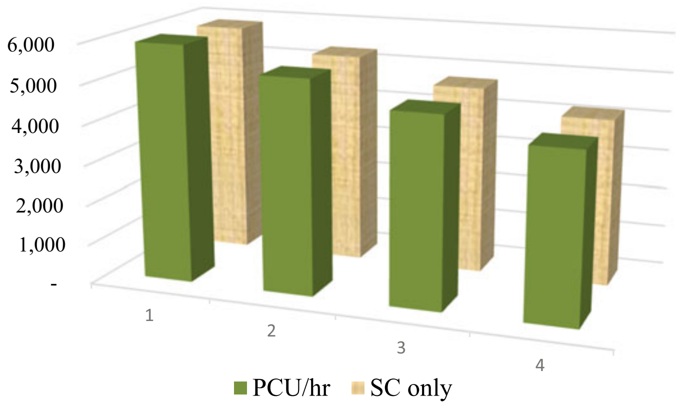


Fig. 7 Comparison of heterogeneous (PCU/h) and SC-only traffic

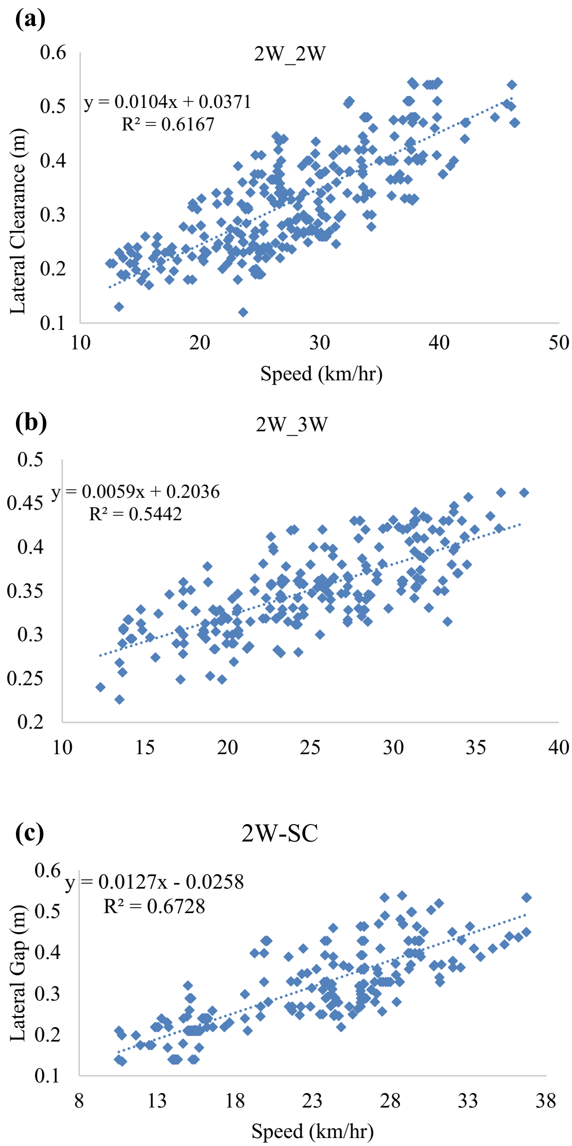
6 Modelling Lateral Clearance of Vehicles

From the number of simulations runs with different random increment of seed for each simulation run, the average data was obtained, purposefully for more accurate results of analysis. The variation of vehicle specific speed with the lateral clearance maintained with adjacent vehicles was observed. All possible combinations of five class of vehicles with each other were extracted from the trajectory data. The coordinates of vehicles were obtained for every time step in simulation run. One second was divided into five parts (five-time steps per second) for more accurate results. Firstly, the centre to centre (c/c) distance of vehicles were extracted and then half of the widths of two adjacent vehicles was subtracted from the c/c lateral clearance between the vehicles. This lateral spacing (clearance) of vehicles was studied further. It was observed that, the lateral clearance of vehicles varied with respect to speed of vehicle, type of vehicle (size/dimensions/manoeuvrability), and the lane in which the vehicle is moving. Large variation of lateral clearance was observed among the vehicle pairs. It was observed from field that a vehicle moving adjacent to another vehicle, within a one-metre boundary has significant effect over its lateral movement, hence the range of lateral clearance values was selected from zero to one metre from the data set. The trajectory data obtained from the simulation run was completely in a scattered form, hence the outlying data (outliers) was removed from the analysis, as those data points would not make substantial difference to the results. As a vehicle within the lane is moving adjacent to another vehicle within one-metre distance laterally, then it is believed to have had an effect on the parallelly moving vehicle. This variation of speed of vehicle with the lateral clearance was captured with all pairs considering one vehicle pair at a time. For 2W as the subject vehicle, the results of speed versus (vs.) lateral clearance share as maintained with all categories of vehicles is plotted. Though the data points are scattered, the outliers were removed to obtain the results based on the data points which actually matters for the lateral clearance behaviour.

From Fig. 8a–e, it can be observed that the lateral clearance for the 2W varies significantly as the subject vehicle (vehicle adjacent to it) changes. For heavy vehicles, there is not much difference in the clearance maintained at various increasing speed levels, but in case of 2W and 3W these fluctuations are different. The cumulative frequency of the speed and lateral clearance together is represented in Fig. 9, considering 2W as standard vehicle.

Here, in Fig. 9, the term V-2w refers to the combined speed data of the 2W, while considered to calculate the lateral clearance with all vehicle classes, including 2W. Hence, V-2w implies the combined speed profile of 2W with all vehicles moving adjacent, and the other five curves of all vehicle classes represent the variation of lateral clearance of the specific vehicle with 2W as the subject (adjacent) vehicle. Figure 9 explains the variation of lateral clearance maintaining behaviour of all vehicles together, with their cumulative frequency distribution compared with the combined speed data of subject vehicle (2W).

Fig. 8 Speed versus lateral clearance



7 Percentile Variation of Speed Versus Lateral Clearance

The field practices are however limited to minimum (15th), average (50th) and maximum (85th) speed limits of the vehicles. The overall data obtained from the simulation results of all three LOS levels (LOS-C, D and E) was combined together and a combined percentile variation was studied for the traffic stream. The design of

Fig. 8 (continued)

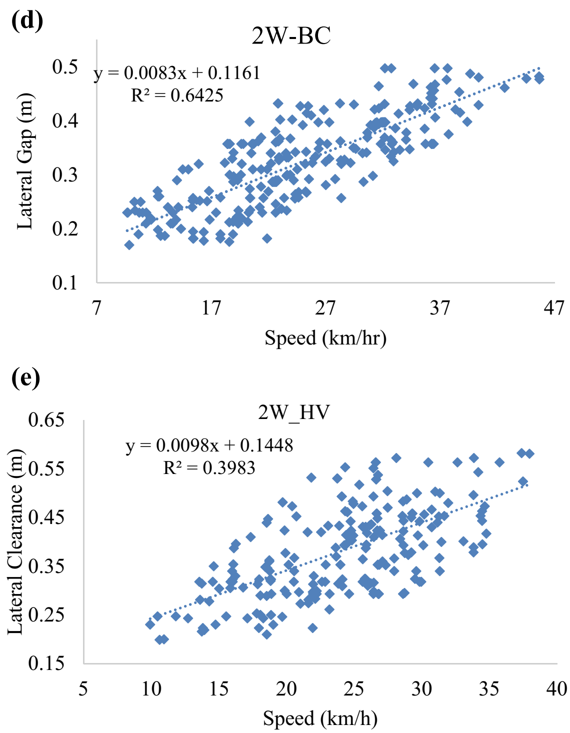
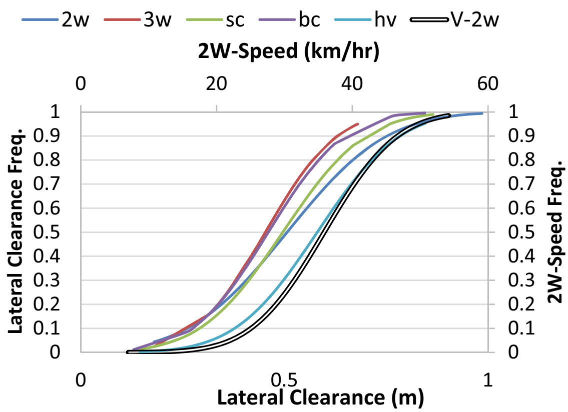


Fig. 9 CDF plot for 2W-speed and lateral clearance



the roadway is mainly based on these numbers, hence a typical percentile variation of percentile speed versus percentile lateral clearance for combined data is shown in Fig. 10.

In Fig. 10, the three percentile values mentioning the 15th, 50th and 85th percentile value, respectively for the three points in the figure from left to right. It has been

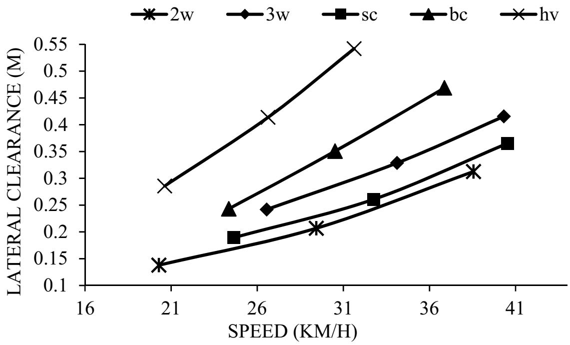


Fig. 10 Percentile variation of speed and lateral clearance (combined data)

observed that, as percentile speed increases, vehicles lean towards maintaining more distance with the vehicles moving adjacent to them. It was observed from field that, 2W drivers accept the minimum available gap (available lateral clearance between the adjacent vehicles) to overtake the vehicles, even at congested conditions. This heterogeneous behaviour of 2W was clearly captured. The clearance maintained by 2W with adjoining 2W was least, as compared to any other vehicle class moving adjoining with 2W. This lead 2W pair to have the least lateral clearance values at 50th and even at 85th percentile speeds which may be due to the smaller dimensions and better manoeuvrability characteristics of 2W. In contrast to this, the clearance maintained by HV was maximum at 50th and 85th percentile speed, as compared to all other vehicles. This is observed due to the large dimensions of HV. When a HV is operating at considerably higher speed, it was observed that the vehicles present on the roadway were maintaining larger gaps with HV and provides the right of way for HV to move. Psychologically, it can be stated that the large dimensions/size of heavy vehicles might be the reason behind this behaviour of other vehicles, and 2W is not an exception to this. Even, SC and BC have similar manoeuvrability and acceleration deceleration capabilities, but a BC is slightly superior to SC in case of engine capacity and size. The percentile speed values and the clearance maintained by BC with 2W were also observed to be more than SC by 22% at 15th percentile, and 25% at 50th percentile. This may be due to slight superiority characteristics of BC over SC in case of engine capacity and manoeuvrability. An increasing trend was observed between the speed and the lateral clearance of vehicles, with increase in speed of vehicles. Furthermore, it was observed that the clearance value surged with vehicle size ranging from 2W to HV.

8 Conclusions

The calibrated VISSIM model was used to simulate the various levels of traffic volume level. The capacity of the midblock section is obtained using the Greenshields's model. The behaviour of individual vehicles is likely to change with the variation in the traffic stream, and it is very difficult to study this from field. The lateral behaviour being a very intricate and complex parameter, hence it was studied using the microsimulation technique. The reliability of the simulation model was checked by calibrating the speed, volume and density of the traffic stream. The speed data collected for each vehicle category was calibrated separately, for the simulation model to represent the actual field conditions more accurately. Also, the default drivers' parameters in the VISSIM software were modified using trial and error method to replicate the volume and density that observed from field. The capacity of the urban arterial road selected for study was obtained as 5957 veh./h. (6034 PCU/h). The six-lane divided urban arterial road was observed to have V/C ratio as 0.91 and hence the Level of Service of the system was obtained as E. The PCU of vehicles was calculated at each volume level and their variation was studied with the V/C ratio. Further, the variation of speed of vehicles with lateral clearance with the vehicles in the traffic stream was studied. Five categories of vehicles, with one type as subject vehicle was considered. The percentile variation of speed with lateral clearance was also studied for 15th, 50th and 85th percentile values.

References

1. Bunker J, Parajuli A (2006) Examining lateral positions of cars and heavy vehicles on a two lane, two way motorway. Transp Eng Australia 10(2):129–139
2. Chandra S, Kumar U (2003) Effect of lane width on capacity under mixed traffic conditions in India. J Transp Eng 129(2):155–160
3. Dhamaniya A, Chandra S (2017) Influence of operating speed on capacity of urban arterial midblock sections. Int J Civ Eng 15(7):1053–1062
4. Kaur S (2015) Use of Vissim in heterogeneous traffic modelling—a review:453–454
5. Lennie S, Bunker J (2004) Using lateral position information as a measure of driver behaviour around MCVs:1–19
6. Mahapatra G, Maurya AK (2013) Study of vehicles lateral movement in non-lane discipline traffic stream on a straight road. Proc Soc Behav Sci 104:352–359
7. Manjunatha P, Vortisch P, Mathew T (2013) Methodology for the calibration of VISSIM in mixed traffic. In: Transportation research board 92nd annual meeting, p 11
8. Pal D, Mallikarjuna C (2016) Analysis of the effect of variable lateral gap maintaining behavior of vehicles on traffic flow modeling. Proc Eng 142:197–203
9. Siddharth SMP, Ramadurai G (2013) Calibration of VISSIM for Indian heterogeneous traffic conditions. Proc Soc Behav Sci 104:380–389
10. Srikanth S, Mehar A, Parihar A (2017) Calibration of Vissim model for multilane highways using speed flow curves. Stavební obzor Civ Eng J 26(3):303–314

Trajectory Data and Driver Behavior Analysis at Urban Mid-Block Section

T. Sowjanya and S. Moses Santhakumar

1 Introduction

Indian traffic conditions are highly heterogeneous with virtually no lane discipline [1]. The vehicles not only interact with the vehicles present in front of it but also interacts laterally with the vehicles present in its neighborhood [2]. Lane changing has a significant impact on traffic flow. Driver behavior on road sections is one of the most complicated phenomena to study, particularly under mixed traffic conditions on urban roads particularly, on the mid-block sections [3].

An urban road is a road located within the boundaries of a built-up area. Lane changing behavior is much more complex in mixed traffic conditions, where the behaviors of vehicle and driver show a massive change compared to the behavior under homogeneous traffic conditions. During a lane changing maneuver, small-sized vehicles can be easily maneuvered and tend to change lanes using smaller gaps. The lane change duration and the space gaps required for lane changing may also depend on vehicle-type dependent characteristics. The absence of lane behavior implies that a driver not only interacts longitudinally with the vehicles ahead but also laterally with vehicles on either side.

Macroscopic impacts on traffic flow result from aggregate results of microscopic driver behavior. Microscopic driver behavior includes acceleration, deceleration, car-following, lane changing behavior, and gap acceptance. Non-lane-based movement is an indication of weak lane discipline of vehicles, which is predominant in mixed traffic conditions and it is a very important aspect in lane maintenance behavior of mixed traffic. The development of suitable models, to represent the typical behavior of drivers on mid-block sections will be of much practical importance. The car

T. Sowjanya (✉) · S. M. Santhakumar
Department of Civil Engineering, National Institute of Technology, Tiruchirappalli, India

S. M. Santhakumar
e-mail: moses@nitt.edu

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_33

following theory is one of the important criteria about the lane discipline. Safety on roads is a major concern in the developing world. Due to the rising population, the traffic risk has increased especially in developing nations like India as the system is unable to cope up with and control the increasing traffic.

1.1 Objectives

The main objectives of this study include

- Developing vehicular trajectory data at the mid-block study locations under heterogeneous traffic conditions.
- Analyzing the lane changing behavior of driver on the mid-block sections in heterogeneous traffic conditions.

2 Literature Review

It is essential to know the vehicle trajectory data for investigating vehicle behavior in a mixed traffic conditions. Several methods have been used to know the vehicle trajectory data in the past works. Raju et al. [4] have studied vehicle behavior in mixed traffic environment. They analyzed the macroscopic and microscopic flow characteristics, including the relative velocity, relative spacing between the vehicles both in the longitudinal and lateral direction. They have concluded that the lateral behavior of a vehicle plays a vital role in the driving behavior and its patterns which is not predominant under homogenous traffic conditions on high-speed multilane highways. Kumar et al. [5] extracted the trajectory data of the urban mid-block section and smoothened the extracted trajectory data to minimize human errors by moving average method, local weight regression method, and Savitzky golay filtering method and performed SWOT analysis for smoothening techniques. They concluded that for smoothening vehicle trajectories, ‘Moving Average Method’ is the best technique. Kanagaraj and Asaithambi [6] evaluated vehicle following behavior and concluded that vehicles in the mixed stream-in particular motorcycles move unsubstantially in the lateral direction.

Bangarraju and Ravi Shankar [7] worked on the lateral distance keeping driver behavior in mixed traffic conditions with little lane discipline. They have concluded that Lane changing frequency is influenced by the density, flow, and mean speed of the traffic stream. Lane by lane vehicular arrivals is not very independent. Sanik et al. [8] worked on driver lane changing behavior at urban intersection by using gap acceptance approach and they concluded that the main factor which influences lane changing event is the flow rate of vehicle movements. Sanik et al. [8] have researched on driver lane changing behavior at urban intersections by using the gap acceptance approach. They have concluded that the main factor which influences the lane changing event is the flow rate of vehicle movements. Mahapatra et al. [9] studied the vehicles lateral movements on non-lane discipline traffic stream, on a straight road and concluded that lateral acceleration and heading angles are high, in the case of three-wheelers than in case of Cars. The variation of lateral acceleration and heading angles are high at lower speeds. Daniel (Jian) Sun studied the driver behavior characteristics considering focus-group studies and in-vehicle driving tests and developed a gap acceptance algorithm to model lane changing on urban arterials. CORSIM was used to develop the model. Durga Rani et al. [10] created a simulation model for a 4-kilometer urban road corridor in Chennai using real-time traffic data. They found that this model is useful for applications of Intelligent Transportation Systems. Chai [11] conducted an intersection survey and developed a Cellular Autometa (CA) model to simulate vehicle interactions, concluding that this model may be utilised to determine the severity of vehicle conflicts for various geometric layouts and traffic control strategies. Munigetty et al. [12] created a semi-automated technique to extract vehicle trajectories from traffic data with an accuracy of 0.1 s. To get real-world coordinates from two-dimensional coordinates, a videographic survey was utilised, followed by a camera calibration technique. Mallikarjuna et al. [13] have done research on lateral gaps maintained by different types of vehicles under different traffic conditions. The statistical analysis was done and the gaps maintained by the vehicles with more are less same speeds were normalized.

2.1 Gaps in the Literature

The studies were carried out to analyze the safety and lane changing behavior of the driver and to extract the vehicle trajectories were done on signalized intersections [14], national highways, multi-lane roads [15], and on bridges i.e., on control points. The focus group and in-vehicle data may not give accurate results as the group of selected people already knows about the test. Only a few studies were found in urban areas to describe the acceleration and deceleration, longitudinal and lateral behavior of vehicles. Most of the research works were carried out without considering the lateral speeds and lateral accelerations of the vehicles on selected road sections [16]. There is a need to study the behavior of drivers on urban mid blocks, considering the above-mentioned characteristics.

3 Study Area and Methodology

The Road approaching near Government General Hospital—Kurnool is located as a study area.

Kurnool, one of the most populous districts of the state of Andhra Pradesh, has seen a considerable increase in traffic over the last few years.

Government General Hospital is located in the central part of Kurnool city having heavy traffic. Road dividers have been provided on the road. Traffic flow videos are taken by providing CCTV camera on the foot over bridge located at a height of 8.0 m from the surface of the road. Traffic is recorded continuously for a period of 5 h from 8:00 AM to 1:00 PM to covering up to 120 m from the Foot Over Bridge located to toward the Kurnool—Kadapa canal, popularly known as K-C canal to assess the volume, speeds, trajectories, and lane change behavior of drivers of different types of vehicles (Figs. 1 and 2).

For analyzing the traffic characteristics, the peak 15 min data was considered. The total length of road stretch is divided into four parts, each part 30 m in length. This was done to know the accurate values of vehicle trajectories and lane changing behavior of the driver. The vehicle trajectories, flow, speed were extracted using Traffic Data Extractor (TDE), for every 1.0-s resolution. To minimize the error of parallax, the video is cropped and enlarged for every 30 m and the lateral movements corresponding to the longitudinal distance of the vehicle were reported. The Methodology involved in the study is shown in Fig. 3

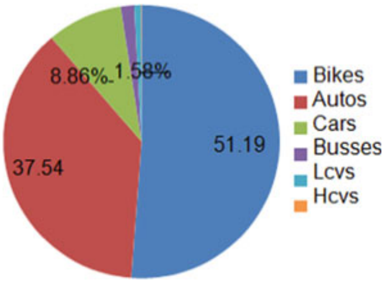


Fig. 1 Percentage of vehicle types

Fig. 2 Screenshot of study area

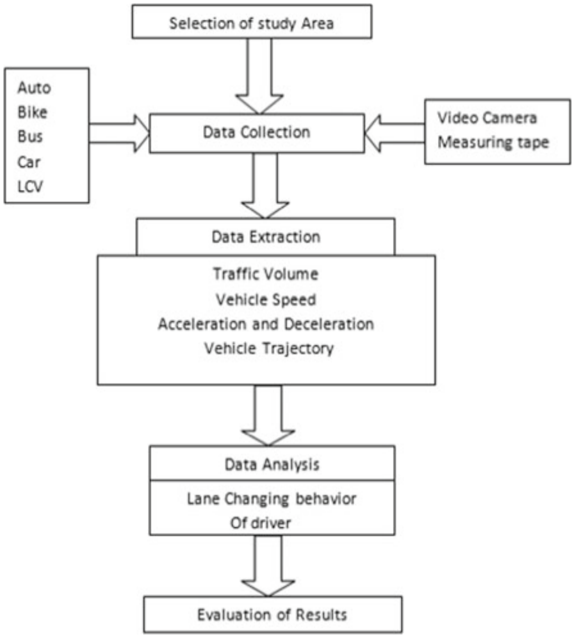


Fig. 3 Methodology of study

3.1 Longitudinal Flow Characteristics

The traffic video was taken for a period of 5 h, from morning 8:00 am to mid-afternoon of 1:00 pm to cover the peak and off-peak traffic data. For analysis of traffic characteristics, the peak 15 min data between 9:45 AM to 10:00 PM was considered. Traffic flow, speed, and densities were reported for 1-min accuracy. Table 1 presents a summary of the statistics of the traffic flow characteristics in the longitudinal direction. The reported speed and acceleration statistics are for instantaneous values. The total maximum traffic flow observed in the study section is 6,342 vehicles per hour (vph). The maximum density (veh./km/h) was observed as 83.142 for Bikes and 71.971 for Autos, followed by 11.874 for Cars. The instantaneous speeds of all the vehicles vary from 0 to 23.81 m/s. Average speeds of all vehicles observed as 1.82 m/s. The mean speed of the Buses is highest 11.74 m/s followed by Cars 11.14 m/s. Autos travel at very less speed 9.923 m/s. Analysis of variance (ANOVA) tests was conducted on individual and pair-wise vehicles. These tests showed that the differences among the vehicle types are statically significant. Pair-wise comparisons show that Auto-Bikes and Cars-LCVs are statistically significant with p-value < 0.01. Bike-Car with p-value 0.228 and Auto-LCVs with p-value 0.654 are not statically significant.

The mean values of Acceleration and Deceleration of all types of vehicles are found to be 1.667 m/s^2 and $-$ 0.039 m/s^2, respectively. Acceleration of Bikes is higher at 2.107 m/s^2 followed by Cars at 1.99 m/s^2. Acceleration of LCVs is very

Table 1 Longitudinal flow characteristics

Flow (Vph)				
Vehicle type	Minimum	Maximum	Mean	Std. dev.
Auto	1373	2519	1803	522
Bike	1378	3201	2454	816
Bus	44	102	76	24
Car	383	463	425	33
LCV	18	57	37	16
All types	3196	6342	4765	1411
Speed (m/s)				
Auto	2.077	21.428	10	2.022
Bike	1.308	25	11	2.466
Bus	2.3	22.727	11.74	2.963
Car	1.91	23.781	11.14	2.64
LCV	1.506	20.27	9.923	3.008
All types	1.8202	22.641	10.760	2.619
Acceleration (m/s^2)				
Auto	0.016	1.784	0.449	0.432
Bike	0.016	2.107	0.415	0.469
Bus	0.153	1.763	0.662	0.548
Car	0.038	1.99	0.374	0.356
LCV	0.060	0.694	0.327	0.268
All types	0.056	1.667	0.445	0.414
Deceleration (m/s^2)				
Auto	− 2.88	− 0.024	− 0.519	0.548
Bike	− 3.60	− 0.015	− 0.437	0.499
Bus	− 1.58	− 0.078	− 0.553	0.632
Car	− 2.14	− 0.049	− 0.626	0.553
LCV	− 0.57	− 0.031	− 0.269	0.175
All types	− 2.15	− 0.039	− 0.48	0.481

low at 0.694 m/s^2. This may be due to the poor dynamic operations of LCVs. The average acceleration of all vehicles is 1.667 m/s^2. ANOVA test was conducted on pair-wise vehicles shows that there are not statically significant. Auto-bike, bike-bus, and bike-car are reported with *p*-values 0.275, 0.272, and 0.737, respectively.

Cars and bikes have higher lateral speeds of 1.982 m/s and 1.307 m/s, respectively, compared with other vehicles which can be attributed to their higher maneuverability. Buses and LCVs have minimum lateral speeds due to their size and less maneuverable ability. Pair-wise analysis of ANOVA test results show that the lateral accelerations of auto and bike are statically significant with p-value < 0.01, but, bike-cars and

Table 2 Details of lateral flow characteristics

Vehicle type	Minimum	Maximum	Mean	Std. dev.
Speed (m/s)				
Auto	0	1.164	0.125	0.124
Bike	0	1.307	0.185	0.169
Bus	0	0.959	0.167	0.187
Car	0	1.982	0.135	0.169
LCV	0	0.074	0.073	0.265
Acceleration (m/s^2)				
Auto	0	0.823	0.131	0.112
Bike	0	1.377	0.198	0.169
Bus	0	0.726	0.163	0.135
Car	0	0.624	0.150	0.122
LCV	0	0.344	0.126	0.092
Deceleration (m/s^2)				
Auto	– 0.871	0	– 0.150	0.137
Bike	– 0.96	0	– 0.193	0.172
Bus	– 0.636	0	– 0.156	0.124
Car	– 0.963	0	– 0.141	0.131
LCV	– 0.416	0	– 0.122	0.096

bus-LCV are not statistically significant with p-values 0.006 and 0.238, respectively, for 99% significant levels. The details of lateral flow characteristics are shown in Table 2.

Toledo et al. [17] used locally weighted regression analysis to estimate the vehicle trajectories and concluded that this method is well suited for mapping highly non-linear functions. The sensitivity analysis revealed that the approach has to be rebooted in terms of measurement errors and missing values. The Time–Space diagram is useful in studying the relationship between the location of vehicles in a traffic stream, and the time and vehicle move along the roadway. Figure 4 shows the movement of vehicles in longitudinal direction concerning time.

The upper part of Fig. 4 shows the Time–Space diagram when all the vehicles are considered for obtaining the observations. In contrast, the lower part shows the Time–Space diagram when only a few numbers of vehicles are considered for obtaining the observation.

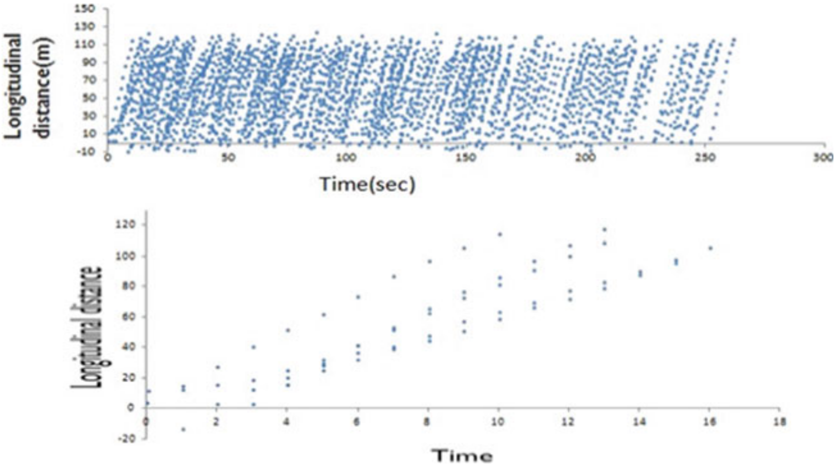


Fig. 4 Time–space graph

3.2 Lateral Flow Characteristics

The trajectory data was extracted, both in longitudinal and lateral directions using the TDE. In the traffic data extractor user interface, the surveyed video files were loaded. The trap length and width of road sections were given as length and width of the marked rectangle portion, which acts as calibration of the road section on the software screen. Further, the vehicle class of a given vehicle was entered in the software interface by observation. Then, that particular vehicle will be tracked employing clicking on the vehicle with the help of the mouse pointer. On a similar basis, every vehicle present in the traffic stream was tracked for the selected time duration. The tracked data were exported to MS Excel files in the form of image coordinates, and it was further converted to real-world coordinates along with vehicle trajectories. Vehicle trajectories were extracted with a time resolution of 1.0 s. The tracked vehicles will be shown with a green color mark.

The lateral behavior was studied using extracted data by TDE. For this, the median side was taken as the origin with lane-1 followed by lane-2 and lane-3. The data after 3 lanes were not considered though the road is extended up to 1.5 m, due to only 1.022% of the vehicles are observed while extracting the traffic volume data. Pedestrian volume and crossings were not considered in the study due to the presence of foot over bridge. It is observed that a minimum gap from the median for bikes is 0.112 m, whereas the LCVs are moving at a minimum distance of 0.574 from the median. Distributions of lateral displacements of various types of vehicles are shown in Fig. 5

These charts reveal that most of the autos are moving in lane-2, buses, cars, and LCVs are moving close to the median, and bikes are moving almost all the width of the lane.

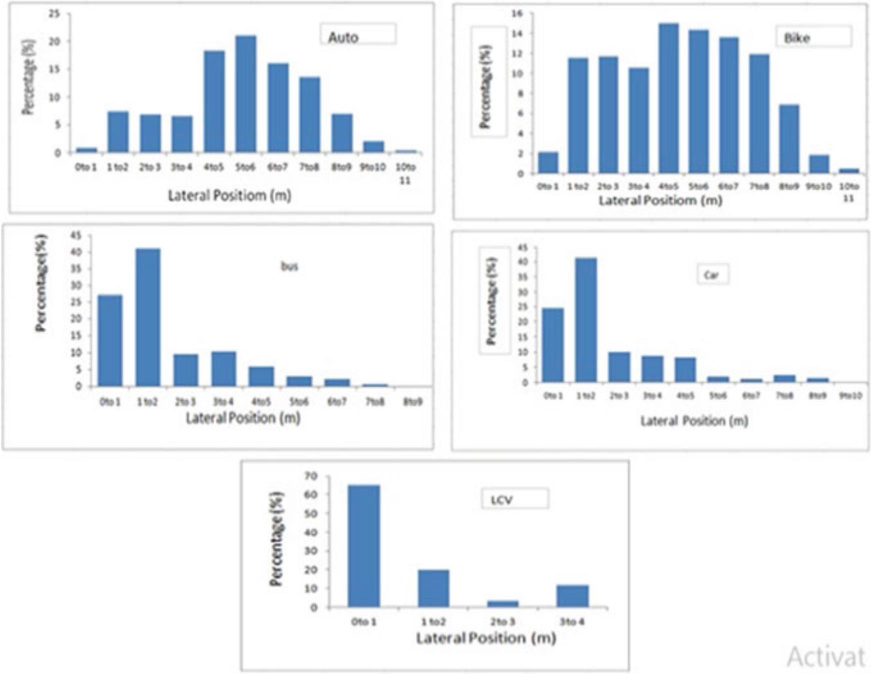


Fig. 5 Lateral displacements of vehicles from median side

3.3 Lateral Amplitude

To study the lateral behavior of vehicles, lateral amplitude was computed for each vehicle category over the study stretch. Lateral amplitude is the lateral weaving of vehicles, which is the difference in maximum and minimum lateral positions over the study section (Narayana raju), as shown in Fig. 6. From the analysis, it may be inferred the smaller vehicles (Auto and Bike) tend to have a more weaving nature, when compared with other vehicles. From this, it can be inferred that mainly small vehicles are responsible for this non-lane-based movement of vehicles (Table 3).

Fig. 6 Lateral amplitude of vehicles at mid-block section

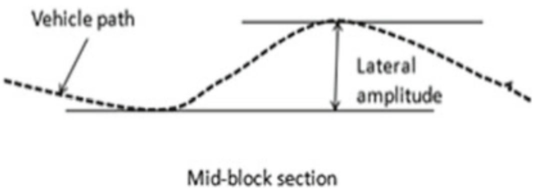


Table 3 Lateral movement of vehicles

Vehicle type	Minimum	Maximum	Mean	Std. dev.	Range (m)
Auto	0.194	6.193	1.423	0.905	4.464
Bike	0.112	4.989	1.265	0.827	4.877
Bus	0.332	2.303	1.025	0.615	1.971
Car	0.123	4.558	0.916	0.859	3.426
LCV	0.291	1.293	0.615	0.348	1.001

4 Summary and Conclusions

In this paper, an attempt has been made to study the traffic characteristics of mixed traffic. A detailed set of vehicle trajectory data was collected in an urban mid-block road section in the Kurnool City of Andhra Pradesh, to estimate, longitudinal and lateral speed and acceleration values. Statistical analysis was also done to know the significance level for 99%. The resulting data was studied for combined traffic flow characteristics and variables related to the lateral movement of the vehicles. Based on the study and analysis of data observed, the following broad conclusions have been arrived at. A significant characteristic of the mixed traffic is weak lane discipline. The study revealed that vehicles in the mixed stream, in general, and in particular, Bikes and Autos particularly move substantially in the lateral direction. This was also reported by a study conducted by Singh et al. [18] on lateral position of the vehicles. Acceleration and deceleration values obtained for all types of vehicles indicated that the Bikes and Autos have higher values, differing substantially from those of Cars and LCVs having lower values.

- The longitudinal flow characteristics show that the speeds and accelerations are high in the case of bike, cars, and buses comparative to the LCV’s. This may be due to poor dynamic operations of LCV’s, whereas the lateral speeds and accelerations are high for cars and bikes due to their higher maneuverability.
- All most all the buses and LCV’s are moving very close to the median and choosing the first lane. As the majority of autos and bikes are choosing second and third lanes.
- The lateral amplitude of vehicles shows that the lateral weaving of bikes and autos were more as these vehicles tend to have more weaving nature due to their smaller size compared to other vehicles.

References

1. Singh S, Santhakumar SM (2021) Platoon-based impact assessment of heavy-duty vehicles on traffic stream characteristics of highway lanes under mixed traffic environment. Int J ITS Res. https://doi.org/10.1007/s13177-021-00268-z

2. Singh S, Santhakumar SM (2021) Empirical analysis of impact of multi-class commercial vehicles on multi-lane highway traffic characteristics under mixed traffic conditions. Int J Tranpo Sci Tech. https://doi.org/10.1016/j.ijtst.2021.07.005
3. Singh S, Santhakumar SM (2021) Evaluation of lane-based traffic characteristics of highways under mixed traffic conditions by different methods. J Inst Eng India Ser A. 102:719–735. https://doi.org/10.1007/s40030-021-00549-6
4. Raju N, Kumar P, Jain A (2018) Application of trajectory data for investigating the vehicle behaviour in mixed traffic environment. Transp Res Rec J Transp Res Board 2018
5. Raju N, Kumar P, Jain A (2017) Examining smoothening techniques for developing vehicular trajectory data under heterogeneous conditions. J Eastern Asia Soc Transp Stud 12
6. Kanagaraj V, Asaithambi G, Toledo T, Lee T-C (2015) Trajectory data and flow characteristic of mixed traffic. Transp Res Rec J Transp Res Board 2491:1–11
7. Bangarraju VSH, Ravi Shankar KVR, Mathew TV (2016) Analysis of lateral distance keeping behavior in mixed traffic conditions with little lane discipline. Int J Traff Transp Eng 6(4):431–443
8. Sanik ME, Hamid NB, Nor AHM, Prasetijo J, Ani MSC, Mustakim F. Drivers lane changing behaviour at urban intersection by using gap acceptance approach. ARPN J Eng Appl Sci
9. Mahapatra GM, Akhileshkumar M (2013) Study of vehicles lateral movement on non lane discipline traffic stream on a straight road. Selection and peer-review under responsibility of International Scientific Committee. Elsevier Ltd.
10. Durga Rani K, Venkata Subbaiah K, Satyanarayana Reddy CNV, Gopala Raju SSSV (2010) A simulation study on overtakings and lane changing behaviour on an urban road corridor. Int J Eng Res Technol 3(3):691—698. ISSN 0974-3154
11. Chai YDW. Micro-simulation of vehicle conflicts involving right-turn vehicles atsignalized intersections based on cellular automata. Centre for Infrastructure Systems, Nanyang Technological University, 40 Nanyang Avenue, 639798, Singapore
12. Munigety CR, Vivek V, Mathew TV (2014) Semi-automated tool for extraction of microlevel traffic data from video graphic survey. Transp Res Rec, J Transp Res Board 2443:88–95
13. Mallikarjuna C, Phanindra A, Rao KR (2009) Traffic data collection under mixed traffic conditions using video image processing. J Transp Eng 135(4):174–182
14. Singh S, Barhmaiah B, Kodavanji A, Santhakumar SM (2020) Analysis of two-wheeler characteristics at signalised intersection under mixed traffic conditions: a case study of Tiruchirappalli City. Resilience and sustainable transportation systems. In: Proceedings of the 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 35–43. https://doi.org/10.1061/9780784482902.005
15. Singh S, Panda RK, Saw DK, Santhakumar SM (2020) Effect of heavy transport vehicles on speed-flow characteristics of mixed traffic on multi lane divided intercity highways, resilience and sustainable transportation systems. In: Proceedings of the 13th Asia Pacific transportation development conference. American Society of Civil Engineers (ASCE), Shanghai, China, pp 176–185. https://doi.org/10.1061/9780784482902.021
16. Singh S, Vidya R, Shukla BK, Santhakumar SM (2021) Analysis of traffic flow characteristics based on area-occupancy concept on urban arterial roads under heterogeneous traffic scenario—a case study of Tiruchirappalli City. In: Mehta YA et al (eds) Trends and recent advances in civil engineering (TRACE) 2020, Advances in water resources and transportation engineering, Lecture Notes in Civil Engineering, vol 149. Springer Nature, Singapore (2021). https://doi.org/10.1007/978-981-16-1303-6_6
17. Toledo T, Koutsopoulos HN, Ahmed KI (2007) Estimation of vehicle trajectories with locally weighted regression. Transp Res Rec J Transp Res Board 1999:161–169
18. Singh S, Vidya R, Panda RK, Santhakumar SM (2021) A study on characteristics of vehicular lateral position on rural highways. IOP Conf Ser: Earth Environ Sci 796(1):012046. https://doi.org/10.1088/1755-1315/796/1/012046

Dynamics of Rental Prices and Land Value Relative to Changes in Transportation Infrastructure

Karan Barpete, Arnab Jana, and Karl Meakin

1 Introduction

In the early years of the twenty-first century, the Indian real estate market boomed. The urbanization of India, contributed to by factors such as urban job opportunities, the explosion of the IT industry, and pro-urban state and central government policies, has led to a massive increase in housing demand [1, 2]. The rapidly expanding economy and liberalized investment policies led to a large increase in foreign direct investment, venture capital, and corporate investment, particularly in India's urban areas. This influx of funding contributed to urban development policies, including the construction of new cities like Gurgaon [3].

Mumbai, the capital of Maharashtra, the most urban state in India, is the second densest city in the world at 31,700 people per square kilometer [4, 5]. Mumbai is expected to grow by about 5 million people, to a total population of almost 25 million, by 2030. Currently the 7th largest city in the world, it is projected to be the 6th largest city by 2030 [6]. Mumbai is a major center of urban real estate investment. In addition to being home to some of the richest people on earth, Mumbai experienced population growth of 17% in the first decade of the twenty-first century [5, 7]. More than 40% of the city's population lives in slums; Mumbai's slum population exceeds the total population of Los Angeles. As a result of job opportunities, demand for

K. Barpete (✉)
School of Planning and Architecture, New Delhi, India
e-mail: karanbarpete@spa.ac.in

A. Jana
Centre for Urban Science and Engineering, IIT Bombay, Mumbai, India
e-mail: arnab.jana@iitb.ac.in

K. Meakin
Northeastern University, Boston, USA
e-mail: meakin.k@husky.neu.edu

A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_34

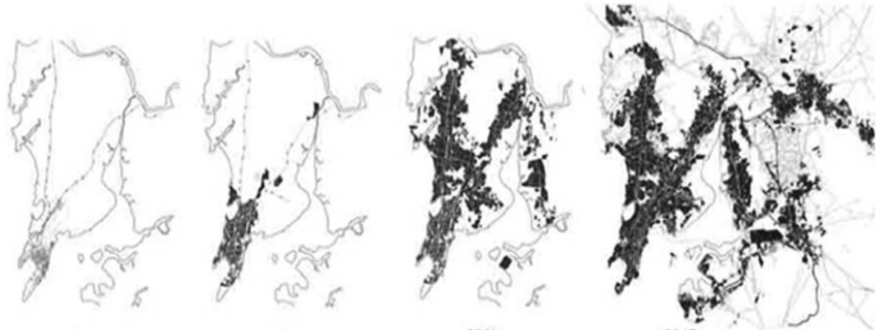


Fig. 1 Urban Sprawl and the growth trend of the Mumbai city

housing is extremely high, driving the price of real estate in Mumbai upwards. In the past decade, housing prices have increased by more than 70%—growth unmatched by most other major cities [8]. Moreover, the price of housing in Mumbai is highly dependent on location. Proximity to the urban core is given a significant premium in housing valuation. Access to transit is thus a valued factor in real estate, as this can significantly reduce travel time to business centers in Mumbai.

Mumbai has two narratives of growth. One narrative is a tale of informal growth, starting with small fishing settlements that has turned into today's informal settlements. The second narrative is that of systematic planning, influenced by the financial market forces of the corporates. This has now spread across the city. Figure 1 shows the growth of urban area in the last 100 years of Mumbai.

1.1 Background Information on Transportation in Mumbai

Mumbai has a number of transportation modes. Most commuting occurs on the local surface rail network, which carries approximately 7.5 million passengers per day [9]. In addition, personal vehicles, taxis, autorickshaws, and buses are common modes of transportation. In 2004, the Mumbai Metro Master Plan was approved by the Mumbai Metropolitan Region Development Authority (MMRDA, n.d.). The first line of the Mumbai Metro, connecting the neighborhoods of Ghatkopar and Versova, was completed in June of 2014. Metro Line 1 has connections with two stations on the Mumbai suburban rail network: Ghatkopar on the Central Line and Andheri on the Western Line. According to MMRDA, Line 1 has an average daily ridership of approximately 300,000 people. Over the coming years, nine more metro lines will be constructed. These metro lines will shape the transportation behavior of Mumbai residents (Fig. 2).

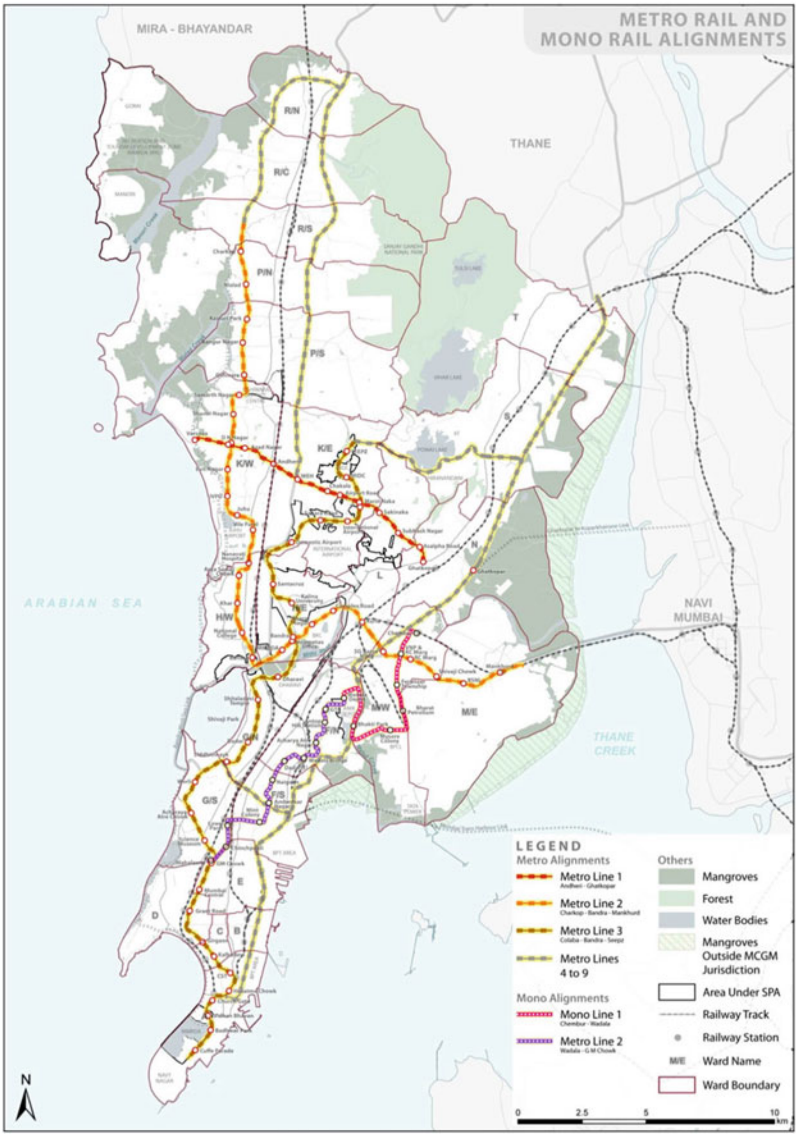


Fig. 2 Mumbai metro rail network (operational and proposed network)

2 Models

Hedonic Price Models are a regression-based model used to assess the effect of various factors on housing prices. HPM analysis assumes several market characteristics, including a homogenous land market, perfect competition, free entry and exit, information symmetry, and market equilibrium. There are several different functional forms that can be applied in an HPM, and the choice of form is important for accurate results. Forms include linear, log-linear, linear-log, and double log, depending on the treatment of dependent and explanatory variables in the model [10].

There have been several studies using HPM to quantify the effects of metro rail on housing prices. Most studies using this method were outside of India—a notable exception being a 2018 study on the Bangalore Metro system [9]. This topic has been studied with some depth in other regions of the world. In some cases, the context or the setting of the study area influenced was the key element of the research. Gadziński and Radzimski [11] used a survey of households along with real estate transaction records, while [12] relied on survey data alone. Heavy/metro rail studies had generally high model R^2 values, ranging from 0.696 in [13] to 0.954 in [14]. Some studies considered variables relating to the accessibility of various amenities, such as shops [15] or green areas [13]. HPM models seen in research included ordinary least squares [13], geographically weighted regression and generalized least squares [14].

Linear and Linear-log hedonic price models are used in this paper to assess the impact of various proximity variables on the rental price of units.

2.1 Data Collection

The land price data for this study was downloaded from a publicly available real estate value aggregator called Ready Reckoner. It is a government source of property transactions and the stamp duty paid on every transaction. It is a proxy for the real market value of the land. Government uses these values to decide property taxes and these values are revised annually. Data on residential, commercial, and industrial value per square meter for the years 2006–2016 were collected.

The land rates are aggregated for regions that are usually the size of a neighborhood. However, in case of larger plots of industrial land the size of this land parcel becomes huge. For the purpose of this research, a hexagonal grid with 200 m side is used and the land prices are overlaid on to the grid. Figure 3 shows the tessellation of hexagonal grid with 4688 hexagons used to uniformly distribute the land prices. Due caution is taken to use tessellation in a way that grids do not comprise of ocean area or protected forest area. However, certain grids completely lie on water bodies like mainland lakes or protected non-saleable open spaces. These grids are identified later and removed from the HPM analysis (Fig. 4).

In addition, the Google Places API was used to gather neighborhood variables (points of interest) and Google Distance Matrix API was used to calculate network distances and travel time.

A GIS based network analysis is also done to calculate Euclidean distances and network distances that could not be calculated by Google Distance Matrix API. For creating selection variables of metro accessibility, similar network analysis was used and Fig. 5 shows one such selection variable.

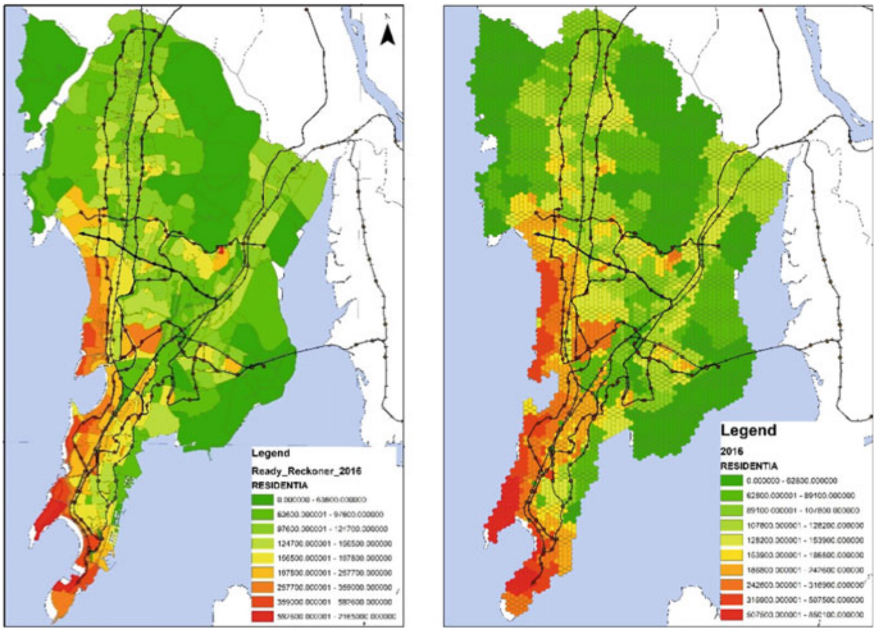


Fig. 3 Ready Reckoner prices reflected on the map of Mumbai (left—government records, right—tessellation on hexagonal grids)

Model function that explains the impact of accessibility on land value change is

$$
\begin{aligned}
\mathrm{LV}_n = {} & \beta_0 + \beta_1\,\mathrm{TT}_{\mathrm{metro}} + \beta_2\,\mathrm{DIST}_{\mathrm{bus}} + \beta_3\,\mathrm{DIST}_{\mathrm{Rail}} \\
& + \beta_4\,\mathrm{DIST}_{\mathrm{Auto}} + \beta_5\,\mathrm{DIST}_{\mathrm{Hospital}} + \beta_6\,\mathrm{DIST}_{\mathrm{Openspaces}} \\
& + \beta_7\,\mathrm{DIST}_{\mathrm{School}} + \beta_8\,\mathrm{DIST}_{\mathrm{Market}} + \beta_n\,\mathrm{DIST}_n + \cdots + \varepsilon \qquad (1)
\end{aligned}
$$

where, LV_n is Land Value in Rupees per square meter (n = Residential, Commercial Office, Commercial-Shop, and Industrial land-use); TT is travel time (Metro has a considerably less travel time in Mumbai than other modes); and DIST is a distance variable from all transportation modes and urban amenities.

Consequently, a survey of 1000 households was conducted within 2 km of eight selected metro stations, five of which are operational and three of which are under construction. See Fig. 6 for a map of the survey area. In total, 700 samples were collected from apartments surrounding operational stations, and 300 from apartments surrounding proposed (under-construction) metro stations. The survey was designed for another study on the changing housing choices in Mumbai. However, some variables are used in this research to validate the hypothesis of change in land value, by using “change in rent” as a proxy for change in land value.

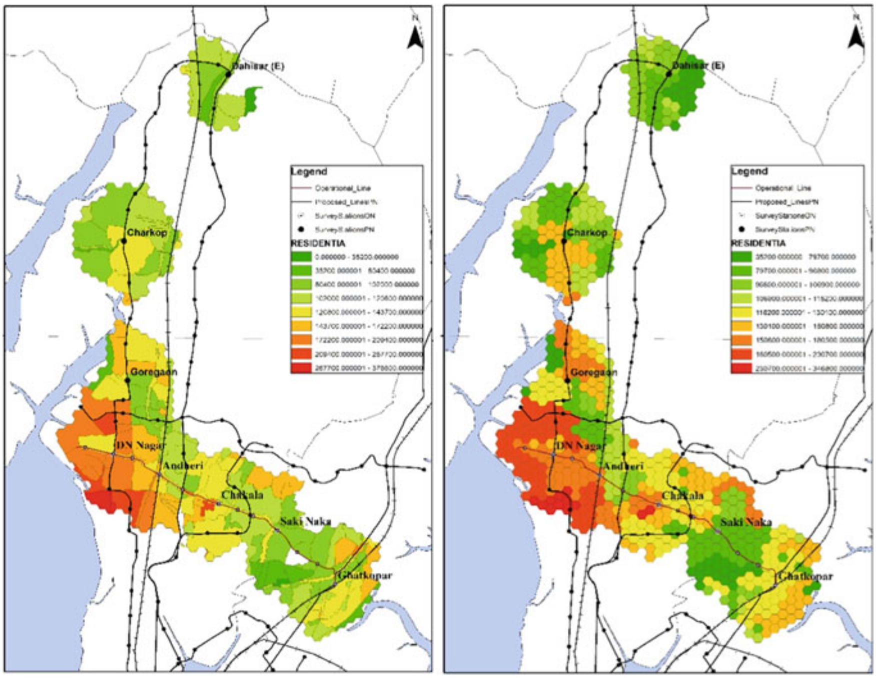


Fig. 4 Residential Land Prices within 2 km network radius from 5 operation and 3 proposed metro stations (left—Ready Reckoner land parcels, right—Hexagonal Grid remastered considering the local land variables)

Primary survey and its sampling method

From the findings of 2 pilot surveys, the criteria for primary survey is decided. The income group for primary survey is identified as medium income group, because it was found to be the majority living near metro stations and also is a larger share of Mumbai population. MIG housing can be identified in Mumbai city by their location, housing structure, and the carpet area.

For a stratified purposeful sampling, the criteria were set to survey households that have lived in at least 2 different homes in Mumbai city (including sub-urban area). This was done so their relocation decision can be analyzed in relation to their change in travel behavior.

The survey was conducted in two categories. First category collected survey responses for the areas near the operational metro line, and the second in the areas near the metro line under construction. This is done to create two control groups. First control group is of renters who do not have access to metro services, Second group (comparison group) is of renters who may have chosen the rental property as the metro services will soon be available.

Criteria for selection of household for survey:

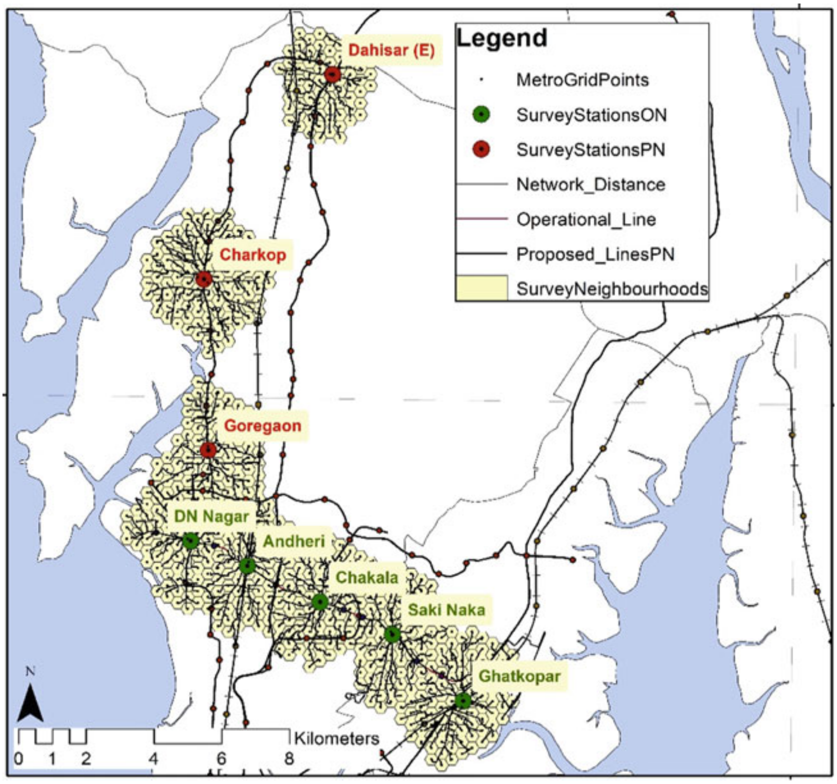


Fig. 5 Network distance of metro stations (5 operational and 3 proposed) with hexagonal grid centroids within 2 km of network radius

- The household should be within 2 km from the selected metro stations.
- The household should be part of MIG housing (which is almost exclusively mid-rise apartments in the case of Mumbai city).
- The resident should be living in the house on a monthly/annual rent basis
- The members of household should have lived in one other place in Mumbai on rent (Table 1).

2.2 Survey Descriptive and Results

Survey variables were selected based upon the literature review. The final survey collected 216 unique answers, including identification variables. Variables useful for constructing a respondent profile, such as age, gender, employment, income, and highest level of education attained, were collected. Additionally, the address and exact location of the apartments were collected, in order to facilitate geospatial analysis. To assess travel behavior, respondents were asked how often they use various

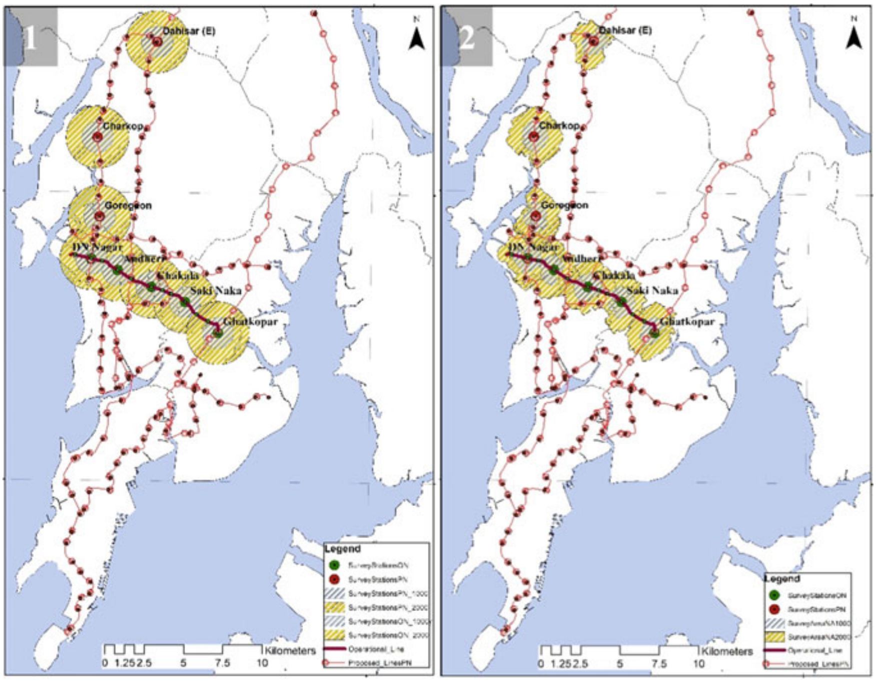


Fig. 6 Identified metro stations for survey and distance from operational network (ON) and proposed network (PN) in meters. 1 Euclidean distance (left), 2 network distance (right)

Table 1 Spatial Distribution of household survey samples

S. No.	Metro station	Metro line	Wards in catchment area	Type of station	Number of samples
1	D.N. Nagar*	1	K/W	Operational	150
2	Andheri+	1	K/E, K/W	Operational	150
3	Chakala	1	K/E	Operational	100
4	Saki Naka	1	K/E, L	Operational	100
5	Ghatkopar*,+	1	N	Operational	200
6	Dahisar	2A	R/N	Under construction	100
7	Charkop	2A	R/S, P/N	Under construction	100
8	Goregaon	2A	K/W, P/S	Under construction	100

* Terminal Station for Metro Line 1
+ Major Station with multi-modal interchange junction and high residential density

modes of transportation (e.g., 2-wheeler, autorickshaw, car, train, and metro). These variables were collected only for the respondent, and were collected on an ordinal scale corresponding to how often they use the mode. Respondents were also asked about the travel behavior of their family members; family member mode choice was collected as a binary variable. Respondents were also asked about the average distance each family member travels daily. These variables can help create a travel behavior profile of the household as a whole and family members individually. To determine respondent's reasoning for use or non-use of the Metro, each respondent was asked to rate the Metro on a variety of aspects using a Likert scale.

In order to generate neighborhood and housing profiles, questions related to the (perceived) safety and type of neighborhood were collected. Additionally, the type (1 BHK, 2 BHK, etc.), area, and rental price of the apartment were collected. Variables on the distance to various transportation hubs (train or metro stop) and to social infrastructure (schools, parks, or grocery stores) were collected for the present and previous residence, allowing for a comparison. Finally, to identify the degree to which residences were chosen for sociocultural reasons, respondents were asked about the importance of living in a community of their religion, caste, or language (Fig. 7).

However, for the purpose of this paper, the change in rental value is used as a dependent variable, and the impact of change in commute distance, change in housing characteristics are used as dependent variable, using a linear hedonic price

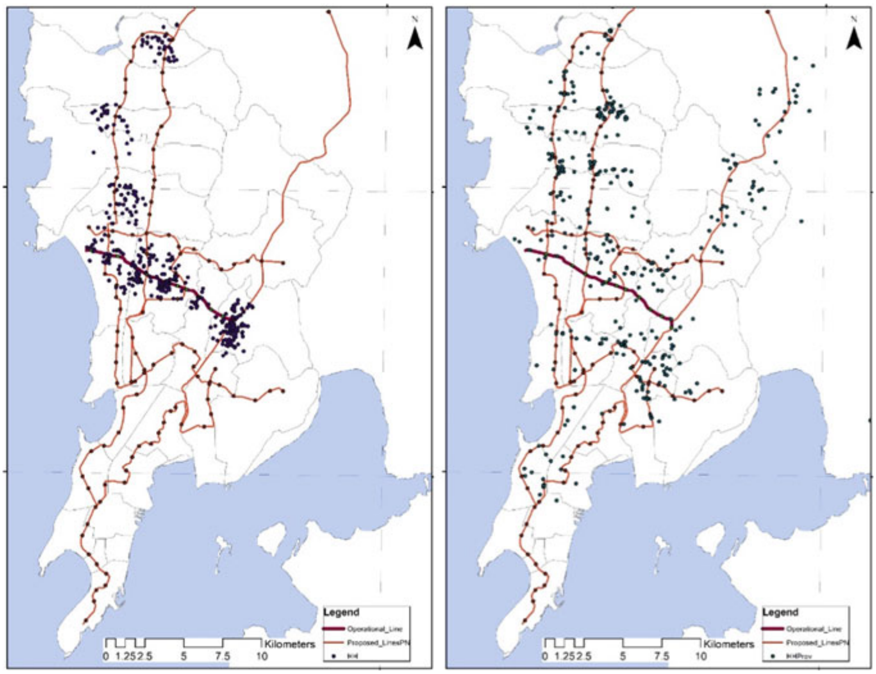


Fig. 7 Survey sample locations (present residence—left, previous residential location—right)

model. The functional form of the model is

$$\Delta R = \beta_0 + \beta_1\,\Delta \mathrm{D} + \beta_2\,\Delta A + \beta_3\,\mathrm{HH} + \beta_4\,K + \beta_5\,I + \varepsilon \tag{2}$$

where ΔR is the change in rent between previous residential location and present location; β_0 is the model constant; β_n (where n is 1, 2, 3, 4, 5) are the coefficients of variables (n = number of variables), ΔD is the cumulative change in commute distance for all the members of household; ΔA is the change in floor area (in m^2) between previous and present house; HH is the number of household members; K is the number of kids (age < 18) in the household; and I is the cumulative income of the household. ε is the residual error term in the model. Comparative results of this model is shown in Table 2, which highlights that while these variables are significant for houses in the vicinity of operational metro, they are not significant in areas where metro is not accessible.

Table 2 Hedonic model for change in rent values near operational metro station versus near proposed metro stations

Dependent variable = delta rent (change in rent between previous and present house)	Group 1 (access to metro = 1) $n = 700$ $R^2 = 0.520$, Adj R^2 = 0.517, $\varepsilon = 3553.470$		Group 2 (access to metro = 0) $n = 300$ $R^2 = 0.406$, Adj R^2 = 0.396, $\varepsilon = 3484.735$	
Variables ↓	Coefficient	*t*	Coefficient	*t*
(Constant)	7289.241	7.919*	4841.941	3.421*
Delta distance (change in network distance to work place, cumulative for all members of household)	− 382.218	− 9.985*	− 402.565	− 5.721*
Delta area (change in floor carpet area between previous and present house)	13.204	6.810*	0.887	0.265
Household size (no. of members)	− 970.967	− 4.110*	− 363.222	− 0.972
Number of kids (age < 18)	835.006	3.615*	165.592	0.467
Household income (income slabs 1. Less than Rs. 25,000/- 2. Rs. 25,000–Rs. 50,000/- 3. Rs. 50,000–Rs. 75,000/- 4. Rs. 75,000–Rs. 100,000/- 5. Rs. 100,000–Rs. 1,50,000/- 6. More than Rs. 150,000/-)	− 1245.437	− 6.652*	− 961.352	− 3.370*

* Significant at 0.01 level

3 Discussion

The results of this research raise a number of questions. What actions can be taken to optimize the outcome of mass transportation projects in their early years, so that there is a systemic integration and the entire city benefits from it? How should mass transportation works be implemented in a way to achieve the overall objectives of Transit Oriented Development? In cities like Mumbai, where the quality of habitat is poor for a wide majority, can high speed transit options like Metro rail help in providing better housing choices in the sub-urban areas?

As this study and the real estate market growth of Mumbai suggests, there is an increase in demand of residential properties near metro stations. For this paper, not just residential land, but commercial (office and retail) land and industrial land parcels were also analyzed over a period of ten years (2006–16). Apart from residential properties, office space prices also observe that distance to metro stations is a significant variable in the hedonic price model. However, office space prices are impacted by metro stations only within a distance of 2 km significantly, whereas in residential rental prices, the impact continues and decreases gradually.

The research hypotheses are largely a product of empirical findings. The hedonic price modeling is not without its shortcomings, more so because the model discussed in this paper uses cross-sectional application, despite there being a temporal dynamic nature of price bidding in land market. However, since there is a decent timeline (2006–16) of real estate price data, the use of multiple points to test the same hypothesis responds to lack of temporal dimension of survey data.

The findings of this research are consistent with the past work on the impact of railway networks in cities in developed nations. Use of hedonic price model is usually done for tax assessments and academic research because of the simple fact that they have shown reliability and statistical robustness in comparison to other primitive approaches of assessing land price.

This paper also validates the notion that the value premium gained from a new transit mode is not just from the infrastructure, but also from the reduced travel time and improved quality of daily commute.

References

1. Joshi H (2006) Identifying asset price bubbles in the housing market in India—preliminary evidence, vol 27, no 1. Occasional papers. Reserve Bank of India, p 16
2. Rouanet H, Halbert L (2016) Leveraging finance capital: urban change and self-empowerment of real estate developers in India. Urban Stud 53(7):1401–1423. https://doi.org/10.1177/0042098015585917
3. Shatkin G (2016) The real estate turn in policy and planning: land monetization and the political economy of peri-urbanization in Asia. Cities 53:141–149. https://doi.org/10.1016/j.cities.2015.11.015
4. Brodie C (2017, May 22). These are the world's most crowded cities. World Economic Forum. https://www.weforum.org/agenda/2017/05/these-are-the-world-s-most-crowded-cities/

5. Gandhi S, Tandelb V, Tabarrokc A, Ravid S (2019) Too slow for the urban march: litigations and real estate market in Mumbai, India
6. UN Department of Econ. and Society Affairs, Population Division (2018) The World's Cities in 2018—Data Booklet. United Nations. https://www.un.org/en/events/citiesday/assets/pdf/the_worlds_cities_in_2018_data_booklet.pdf
7. Pow CP (2016) Courting the 'rich and restless': globalisation of real estate and the new spatial fixities of the super-rich in Singapore. Int J Housing Policy 17(1). https://doi.org/10.1080/14616718.2016.1215964
8. Global Cities House-Price Index (2019, Mar 11) The economist. https://www.economist.com/graphic-detail/2019/03/11/global-cities-house-price-index?date=2009-03&index=real_price&places=MUM&places=LON
9. Sahu PK, Sharma G, Guharoy A (2018) Commuter travel cost estimation at different levels of crowding in a suburban rail system: a case study of Mumbai. Publ Transp 10(3):379–398. https://doi.org/10.1007/s12469-018-0190-6
10. Chau KW, Chin TL (2003) A critical review of literature on the hedonic price model. Int J Hous Sci Appl 27(2):20
11. Gadziński J, Radzimski A (2016) The first rapid tram line in Poland: how has it affected travel behaviours, housing choices and satisfaction, and apartment prices? J Transp Geogr 54:451–463
12. Cao X, Handy SL, Mokhtarian PL (2006) The influences of the built environment and residential self-selection on pedestrian behavior: evidence from Austin, TX. Transportation 33(1):1–20
13. Medda FR, Modelewska M (2011) Land Value Capture as a funding source for urban investment: the warsaw metro system. Ernst & Young Polska
14. Bae C-HC, Jun M-J, Park H (2003) The impact of Seoul's subway line 5 on residential property values. Transp Policy 10(2):85–94. https://doi.org/10.1016/S0967-070X(02)00048-3
15. Yankaya U, Celik HM (2004) Modelling the impacts of rail transit investment on the values of residential property: a hedonic price approach in the case of Izmir Subway, Turkey. Report Izmir Institute of Technology, Department of Civil Engineering. Report Izmir Institute of Technology, Department of Civil Engineering

Control Delay Estimation at Signalized Intersections Using Vehicular Trajectory Data

Ritvik Chauhan, Ashish Dhamaniya, and Shriniwas Arkatkar

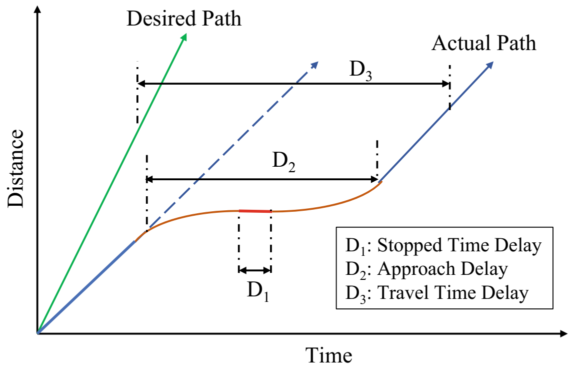
1 Introduction

Characterized by high degree of heterogeneity in driver's population and vehicle classes combined with the non-lane-based driving behavior, the operation and management of traffic in developing countries like India are a challenging task. With significant vehicular population growth (around 8%) [1], the traffic infrastructure in India is often affected by congestion, long queues, and subsequently vehicular delay. The best and most widely used traffic control parameter in urban areas is providing an intersection at nodes to smoothly maneuver the traffic on its desired path. Based on the requirement (demand) and the warrants, different types of intersection design may be deployed. The most common of them being the signalized intersection [2]. Signalized intersection control the traffic by apportioning the right of way to a specific or group of non-conflicting movements cyclically. The key importance of providing the intersection is to dissipate the conflicting traffic while providing higher safety standards, efficiency, and best possible level of service (LoS). As a measure of effectiveness for signalized intersection, LoS is computed in terms of delay. Delay in traffic scenario can be defined as the excess time consumed while traversing through a facility or infrastructure due to some control measure or external condition, causing a reduction in speed of the vehicle from its desired travel speed. Based on the definition, delay for a traffic infrastructure might consists of various subsections like travel time delay, approach delay, control delay, time in queue delay, and stopped delay (Fig. 1). Moreover, computation of delay is also done on various levels like uniform delay, random delay, and overall delay.

R. Chauhan · A. Dhamaniya (✉) · S. Arkatkar
Department of Civil Engineering, Sardar Vallabhbahi National Institute of Technology (SVNIT) Surat, Surat, Gujarat 395007, India

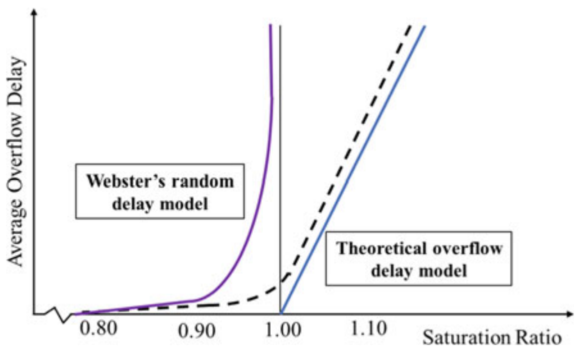
A. K. Maurya et al. (eds.), *Proceedings of the Fifth International Conference of Transportation Research Group of India*, Lecture Notes in Civil Engineering 219,
https://doi.org/10.1007/978-981-16-8259-9_35

Fig. 1 Types of delay at signalized intersection



Over the years, several methods and models have been proposed for delay estimation. Presently, two of the most widely used methods of delay estimation at signalized intersection are Webster's delay model and US-HCM delay model. US Highway Capacity manual 2010 [3] has given the LOS measure for signalized intersection in terms of density and average delay caused. Following the same study and study by Othayoth [4], Indian Highway Capacity manual [5] also uses delay as a measure to quantify the LoS at intersections. The US-HCM delay model was derived based on the growth, modification, and corrections to Webster's delay-based model [6]. Webster model comprises of three parts that respectively consider a uniform arrival rate, random arrival rate, and overflow delay. The model though proposed decades ago, still holds significant value as apart from uniform delay component, Webster also considered the random arrival nature and also even after saturation flow rate being less than 1, sometimes the signal phase may fail to cater the demand due to external factors, and in these aspects, an overflow delay model was given. But if we consider the random delay model, Webster had found that the model many times overestimates delay and hence introduced a factor of 0.9 for computing true delay. But, the Webster's delay model prominently shows inconsistencies for saturation ratio between 0.85 and 1.15 (Fig. 2).

Fig. 2 Inconsistency in Webster's delay model [8]



Following this, models were developed to represent all the saturation flow ratios, among which Akcelik's model [7] has been most commonly used. Being used by the Australian research board for signalized intersection design apart from their highly famous and efficient Sydney Coordinated Adaptive Traffic System (SCATS) traffic management system. The model acts as an extension to Webster's uniform delay (UD) model. Akcelik's model focuses on an overflow delay (OD), and to compute total delay, OD is added to UD.

Two studies [9, 10] have proposed the modification to Webster's model, but the reliability of the method by Hoque and Imran [9] has been questioned because of the source and quality of data, and since, the results by Raval and Gundaliya [10] were not checked for statistical strength, and the data were from a single city; its application is not verified [11]. In their study to measure delay at signalized intersections under mixed traffic conditions, Saha et al. [11] compared various delay estimation methods and also proposed their own model which apart from uniform delay model from Webster's also considered and additional parameter related to platoon ratio replacing the random delay (d_2) component of HCM model. Additionally, apart from the traditional methods, delay estimation based on queue length using Simpsons 1/3rd rule has also been discussed.

Global positioning system-based (GPS) data have proven to be of significant importance in various studies. Initial works by Quiroga et al. [12], proposed a methodology of computing delay based on the trajectories obtained from GPS-enabled devices. The results though measured in the field, their accuracy to represent the field conditions is based on the quantity of GPS-based data. Other studies conducted based on the theme of GPS-based analysis are Liu et al. [13] and Ko et al. [14]. Though statistically proven to be in the interval of delay values, their proximity to real value may be significantly low. Also, the data from GPS are based on how well the device is able to connect with the designated satellite so as to record the details. With the arrival of GPS-enabled smartphone devices a few years ago and have gone serious upgradations till present time, we experimented and compared the results of trajectory details from performance V-box and GPS smartphone. The travel time results were strikingly varying with each other. Though the performance V-box resulted in good results as it also has been tested and proven to do so, for a GPS-enabled smartphone, the variation in results can be attributed to the model and version of GPS hardware, ability of phone to connect and transpond with the satellite and also the quality of application being used for recording the trajectory data can affect the results. Also, the interval in which the data is recorded also varies. V-box recorded the data at every tenth of a second while GPS-based smartphone recorded at every seconds. So, the smartphone is ten times less accurate compare to V-box. Still ultimately, the fact remains that the accuracy for GPS data is dependent on the number of devices or runs deployed during the data collection interval.

Traffic flow theory-related research is more than often affected by the inadequacy of data qualitatively and quantitatively. The ones available are either conventional aggregate data or small samples of trajectories, that hinder the possibility to support and corroborate microscopic theories. This results in the research to be driven toward model-based theoretical analysis [15]. Hence, a major challenge in conducting research related to traffic flow theory is to enhance the study of traffic models with the study of their characteristics and properties in stochastic simulation environments. To meet this aim, a detailed and wide-ranging traffic data are vital. Accessibility to vehicle trajectories during the objective focused study section and time (i.e., a predetermined space–time domain) of all the vehicles permits to validate the theories and models at the microscopic as well as macroscopic levels [16]. Unlike traditional GPS data, trajectory-based data comprehends the traversing of all vehicles belonging to different vehicle classes, and not just busses or taxis from where the GPS data are mostly obtained. Hence, it can provide accurate traffic statistics [17]. These information's are very valuable in transportation modeling. Contrarily to the GPS data, recorded only when device can properly transpond with the satellite. The above differences make the trajectory data unique. From another viewpoint trajectory, such data may allow establishing the overdue link between individual driving behaviors and traffic phenomena. Even in these study, all the parameters required for analysis have been easily computed after development of trajectories. Previously, it might happen that different set of data had to be extracted using manual or semiautomated methods. But with trajectory data though the time required to extract the data is more, the depth which it provides in analyzing and studying the traffic is immeasurable.

Based on the facts and findings from previous studies, six methods, namely GPS-based probe vehicle (performance V-box), queue length-based delay estimation (Simpsons 1/3 rule), Webster's delay estimation model, Akcelik's model, modified westers model, and Indian Highway capacity manual delay model have been used for control delay estimation and compared with the actual field control delay measured using trajectory data.

2 Data Analysis

Data driven study and analysis are required to compute a parameter, its effect and to conclude on a statement. In traffic-related studies, with advent of new technologies, data collection methods have also improved, starting from manual counting at the field, to time series (time lapse) photography, videographic methods to GPS, and sensor-based equipment's. Availability of fast and efficiently processing computers has eased the monstrously task of computation data with ease. Videographic method is most commonly used today as it enables replaying of the field in laboratories so as to further analyze the condition in details [18]. Further, using videographic data and use of computers trajectories can be extracted of all the vehicles within required time frame and section so as to carry on an in-depth study at microscopic level. Being true representative of field conditions, trajectory data can be said to be the most rich and potent representative of the field for computing safety and operational parameters and human behavior in traffic. GPS-based equipment provides a real-time-based data, but it is restricted by the limitation of magnitude of data in terms of quantity. Also, deployment of GPS for all the vehicles in the traffic stream during study period is a mammoth task. Getting data from navigation service provider without consumers consent results in privacy breach. Till and until this hurdle is crossed over, analysis has to rely on the videographic data and self-probed GPS data (dependent on the number of GPS vehicle or number of runs deployed). Sensors are being mounted for the purpose of traffic operations and management, but the accuracy and usage are still not completely dependable. Sensor-based classification of traffic for heterogeneous traffic condition is still limited. With these facts in mind, the data collection was carried out using videographic data and GPS-enabled probe vehicles.

2.1 Data Collection

Videographic data were collected at old RTO signalized intersection, a three-legged T-intersection near Mahavir Hospital in Surat Metropolitan city in the state of Gujarat, India. The data were collected during the peak hours on a weekday with normal and bright weather conditions. The camera was mounted at the terrace of a high-rise building with good vantage point providing a complete view of the intersection with at least 300 m distance visible at upstream and 200 m downstream of the stop line. A good quality video recording camera able to record the video at high definition resolution was mounted at the tripod for stable video. Simultaneously, during the video recording, probe vehicle equipped with performance V-box was deployed on multiple runs.

2.2 Data Extraction

For trajectory extraction, recorded videographic data were replayed in Traffic Data Extractor software (TDE) [19] on a large high-resolution screen supporting the resolution of the video recorded in the laboratory. The data were played at an accuracy of 0.4 s and calibrated to recorded field dimensions measured during data collection. Trajectories for a total of 1865 vehicles corresponding to five vehicle categories were developed summing up for over four complete traffic signal cycles, covering a distance of over 300 m upstream of the intersection stop line and 70 m downstream of the intersection stop line. Figure 3 shows a screen shot from trajectory data extractor software during data extraction, followed by the space vs time plot in Fig. 3. The traffic composition is found to be 11.48% for motorized three-wheelers (Auto-Rickshaws), motorized two-wheelers (60%), bus (1.80%), cars (25.75%), and LCV (0.97%) traffic volume share, respectively.

Fig. 3 Screenshot during vehicular trajectory data extraction using TDE

2.3 *Preliminary Analysis*

Raw trajectory data extracted may contain some minor ambiguities caused due to human error, resulting in reverse tracking causing sudden jumps (while moving) and negative speed (when in stopped conditions) [20]. To avoid these, the trajectory data were post-processed using five-point moving average method to tone down the peaks and smoothen the movements of the vehicles.

From smoothened data, a scatter plot of speed vs distance to identify the influence area of the intersection. It was found at around 180 m upstream of the stop line, vehicles started decelerating, based on the position of the maximum observed queue length. This signifying the area to be calculating decelerating delay (approach delay) a part of control delay.

Subsequently, the vehicle moving during end of the green phase of the cycle was isolated to study the free flow characteristics (intersection being undersaturated). Travel time for these vehicles was also calculated to set a base travel time without the effect of the traffic control measures. Based on 195 free flowing vehicles, free flow speed profile was devised (Table 1).

Considering the 85th percentile speed of 13.81 m/s over the segment influence zone length of 160 m before stop line and 60 m after stop line, base travel time was calculated to be 14 s. Figure 4 gives a discharge pattern for four cycles analyzed.

Table 1 Free flow speed distribution

Percentile speed	V_{15}	V_{50}	V_{85}
Speed (m/s)	9.79	12.37	13.81

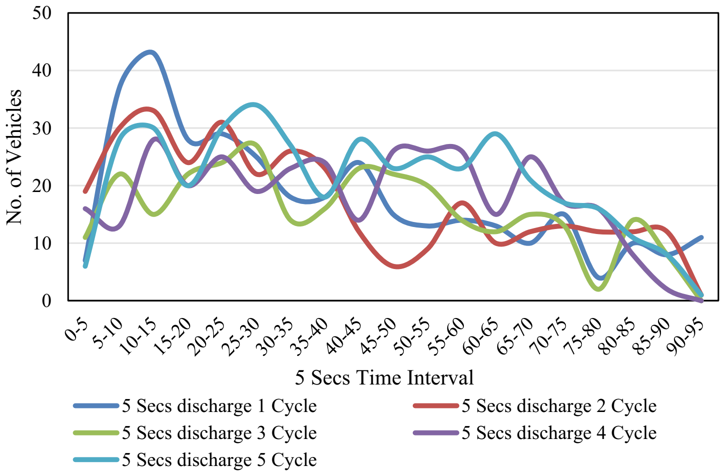


Fig. 4 Discharge profile per cycle from trajectory data

Moreover, all the parameters required for delay estimation using the specified delay estimation methods were calculated using the trajectory data (Table 2).

Table 2 Intersection-related parameters for delay estimation via various models

	Parameter	Value	Standardized
1	No. of vehicles	1483 vehicles	8355 Veh./h
2	Analysis time period	639 s	0.18 h
3	Vehicle arrival rate	2.27 Veh./s	1.13 PCU/s*
4	Cycle length	150 s	
5	Effective green time	86 s	
6	Effective green ratio	0.57	
7	Saturation flow rate	12,869 Veh./h	6283 PCU/h*
8	Volume	8355 Veh./h	4079 PCU/h*
9	Capacity	10,121 Veh./h	4941 PCU/h*
10	Degree of saturation	0.83	
11	Average number of vehicles crossing stop line during green	344 vehicles	
12	Average number of vehicles per second of green	4 vehicles	
13	% of vehicles arriving during green	0.60	

* Indian Highway Capacity manual specified PCU values have been used for conversion of mix traffic to homogeneous conditions, though some of the methods

3 Delay Estimation

3.1 GPS Probe Vehicle-Based Delay Estimation

Based on the identified influence area, from the performance box runs, data were trimmed to only analyze the influence area for delay estimation (Fig. 5; Table 3).

A base free flow travel time over the influence area was calculated form the 195 isolated free flow vehicles. Average free flow travel time was then subtracted from the average travel time of trimmed down V-box runs to find the delay.

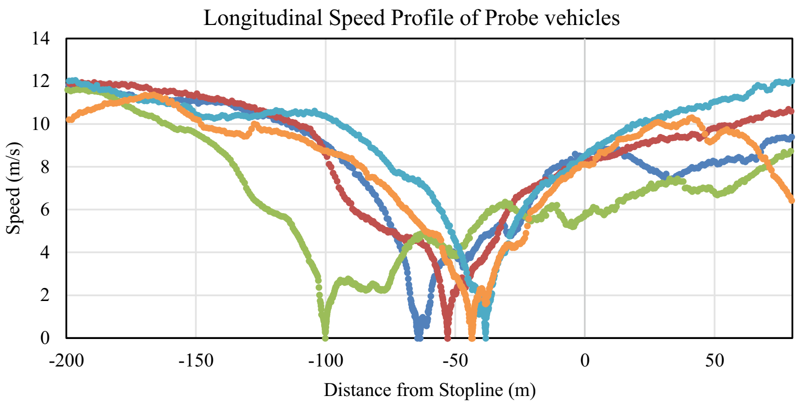


Fig. 5 Performance V-box data in influence zone

Table 3 Performance V-box data for travel time calculation

V-box						
Run	Start point		End point		Distance traveled (m)	Travel time (s)
	Distance (m)	Time (s)	Distance (m)	Time (s)		
1	897.85	127.50	1250.10	163.30	352.26	35.80
2	494.14	62.60	958.12	134.40	463.98	71.80
3	294.93	28.30	647.67	82.70	352.74	54.40
4	283.52	28.30	633.65	77.30	350.14	49.00
5	267.00	28.20	616.54	75.10	349.55	46.90
6	170.20	32.40	436.62	73.80	266.43	41.40

3.2 Field Traffic Trajectory-Based Delay Estimation.

After smoothening of the data and identification of the influence area, the trajectory data were also trimmed down so as to measure only the delay caused by signalized traffic control (Fig. 6).

Following the common definition of measuring speed, as speed increases, time requires to cover the same distance decreases proportionally. Hence, considering the fact that a vehicle moving at or more than the desired free flow speed will not face any delay, and proportionally, the delay is caused with the speed proportionally less than the desired free flow speed.

Since the data have been extracted at an accuracy of 0.4 s, between any two intervals, if the speed is more than 50 Kmph, zero delay is introduced. If the speed is zero, the delay is the complete interval of 0.4 s. Accordingly, delay time addition parameter for each interval has been computed based on the linear interpolation of speed respective to vehicle and trajectory interval. Figure 7 explains the methodology for computing the delay using trajectory data.

All the time addition parameters respective interval throughout the study time period were added to compute total delay. When divided by the number of vehicles analyzed, average delay per vehicle was obtained. Similar procedure is followed for measuring delay for each vehicle class.

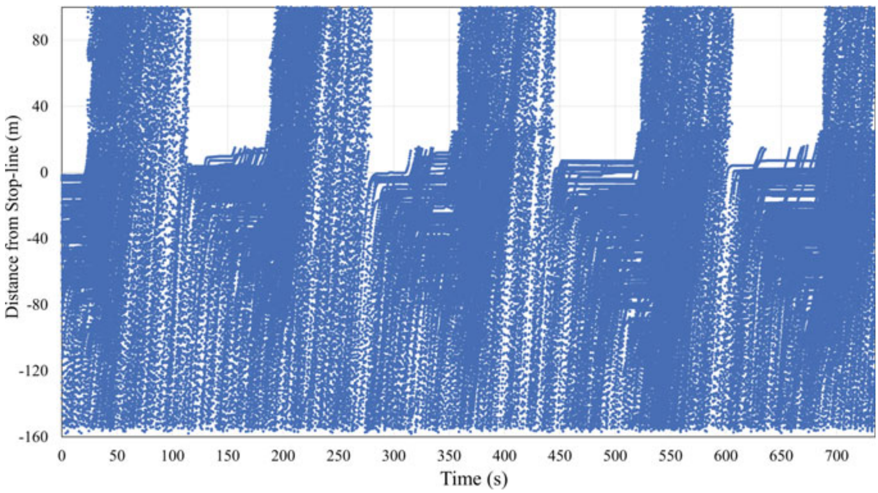


Fig. 6 Trajectory data respective to influence zone

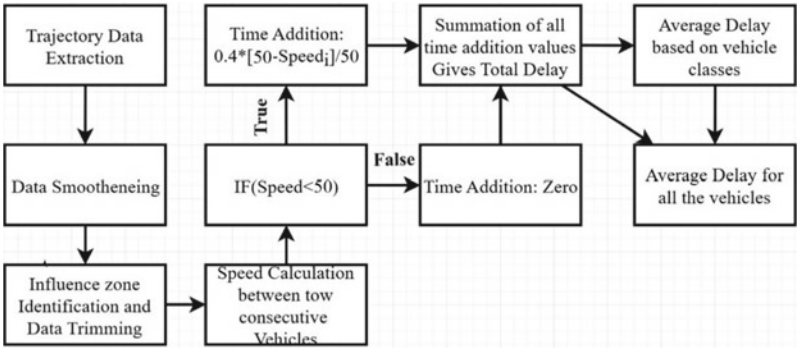


Fig. 7 Methodology for delay estimation form trajectory data

3.3 Delay Estimation Models

As discussed in the earlier sections, four delay estimation models have been studied. Though each model contains inherent advantages and limitations, they have been used previously in several studies to estimate the control delay at signalized intersection in several Indian studies. To be comprehensive, all the parameters used in the models are explained beforehand as:

v vehicle arrival rate,
g/C effective green ratio for the approach,
T analysis period (h),
g effective green time, s,
d average overall delay per vehicle (s/vehicles),
d_1 uniform delay (s/vehicles),
d_2 incremental, or random, delay (s/vehicles),
d_3 residual demand delay to account for over-saturation queues that may have existed before the analysis period (seconds/vehicles),
PF adjustment factor for the effect of the quality of progression in coordinated systems,
C traffic signal cycle time (s),
g effective green time for approach (s),
X Saturation Ratio (volume to capacity ratio of approach),
s saturation flow rate, (vehicles per second green),
c capacity of approach (vehicles/h),
k incremental delay factor dependent on signal controller setting (0.50 for pre-timed signals; vary between 0.04 and 0.50 for actuated controllers),
I upstream filtering/metering adjustment factor (1.0 for an isolated intersection),
T evaluation time (h),
P proportion of vehicles arriving during the green interval,
fp progression adjustment factor.

Webster's Model

The model gives three equations that respectively measure the uniform delay, random delay, and overflow Delay. Uniform delay assumes a uniform arrival rate and stable flow, but contrarily, the arrival pattern of vehicles in is random, in addition to uniform delay, random delay should be added. Since none of the cycle observed didn't fail ($x < 1$), overflow delay has been ignored. Equations 1, 2, and 3 are used to compute delay as per Webster's model.

$$\text{Uniform Delay} = \frac{C\left(1 - \frac{g}{C}\right)^2}{2\left(1 - \frac{g}{C}X\right)} \tag{1}$$

$$\text{Random Delay} = \frac{X^2}{2v(1 - X)} \tag{2}$$

$$\text{Total Delay} = 0.9 * (\text{Uniform Delay} + \text{Random Delay}) \tag{3}$$

Webster's model provides results very well till the degree of saturation value is limited less than 0.85. It has been found that the uniform and random delay model of Webster's model hold strong till this limit. But, when X increases more than 1, overflow delay models perform well only when $X \geq 1.15$. Hence, for $0.85 \leq X \leq 1.15$, the results have been reported to be inconsistent.

Akcelik's Model

To overcome the limitation of Webster's model for the X value between 0.85 and 1.15, Akcelik's model has proven to hold very well for all the values X. Akcelik's model can be said as an extension for Webster's model for delay estimation. This model only gives an equation to compute overflow delay, and the total delay is computed as the addition of overflow delay model and uniform delay model from Webster's. Moreover, for base degree of saturation (X_0) computed using equation, if found to be more than the field degree of saturation, overflow delay is considered to be zero; in such cases, the total delay is expressed as uniform delay from Webster model.

$$\text{Overflow Delay} = \frac{C * T}{4}\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{12(X - X_0)}{c * T}}\right] \tag{3}$$

Provided that $X \geq X_0$, but if $X \leq X_0$, then the overflow delay is considered to be zero.

$$X_0 = 0.67 + \frac{s * g}{600} \tag{4}$$

To compute total delay, uniform delay calculated from Webster's model is used

$$\text{Total Delay} = \text{Uniform Delay} + \text{Overflow Delay}$$

Modified Webster's Model

As per the name, the model by Hoque and Imran [9] introduced an additional terms to the Webster's delay mode. This model also introduces the effect of non-motorized vehicular traffic on the average delay.

$$\text{Delay} = \frac{C\left(1-\frac{g}{C}\right)^2}{2\left(1-\frac{g}{C}X\right)} + \frac{X^2}{2v(1-X)} + 46.93 \\ - 46.04 * v - 37.32 * x - 0.3608 * \text{pnmv} \tag{5}$$

Simpsons 1/3rd Rule

Using the trajectory data, queue lengths were estimated [21, 22] in terms of number of vehicles stacked-up at the intersections. Due to non-lane-based behavior, deterministic queue are hard to observe. Hence, all the vehicles arriving and standing at the intersection during the red phase have been counted and segregated in the intervals of 5 s. Total delay was computed for all the vehicles in a specific cycle and then divided by number of vehicles in the cycle to get the average delay faced by the vehicle in a particular cycle. Equation 7 describes the total delay estimation for each cycle.

$$\int_0^C f(Q_L)dq = \frac{h}{3}\big[(q_0 + q_n) + 4(q_1 + q_3 + q_5 + \cdots + q_{n-1}) \\ +2(q_2 + q_4 + q_6 + \cdots + q_{n-2})\big] \tag{6}$$

where,

$$h = \frac{(C-0)}{n} \tag{7}$$

And

$C-0$ difference between start time and end time of queue.
Q_i queue length in interval i.
C cycle length,
N number of intervals

$$\text{Avg. Delay/Cycle} = \frac{\text{Total Delay}}{\text{no. of vehicle crossing stop line during green phase of cycle.}} \tag{8}$$

Following this, the average delay for all the cycles was aggregated to determine the average control delay.

3.4 Guidelines-Based Delay Estimation

Based on US-HCM delay model, Indo-HCM delay model has been modified in several aspects to meet the Indian scenario.

$$d = 0.9 * d_1 + d_2 \tag{9}$$

$$d_1 = 0.5\, C \frac{\left(1 - \frac{g}{C}\right)^2}{\left(1 - \frac{g}{C}\min(X, 1)\right)} \tag{10}$$

$$d_2 = 900T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{4X}{cT}}\right] \tag{11}$$

It should be noted that the model has been calibrated and developed using under-saturated intersections only. Hence, the terminology of d_3 (delay due to initial queue at start of analysis) of US-HCM delay model is neglected. Also, for computing the random delay parameter (d_2), unlike in US-HCM model "*k*" adjustment factor for the signal control is considered as 0.5 since almost all the signals in India are fixed pre-timed controller and "*I*" adjustment factor for upstream filtering/metering is kept as 1, i.e., all the intersections are considered to be isolated. So, with changes in any of these above conditions if observed in the field, the model shall have to be modified and calibrated accordingly.

4 Results

Table 4 describes the results obtained from each method of control delay estimation. Trajectory data development is a time consuming process, but that is the only limitation to the method. In return, the results are precise and true representative of the field conditions. GPS-based approach is found to be in best agreement to the true delay obtained from trajectory data. But, it should be noted that coincidently the probe runs made for data collection have yielded this results. If for the limited number of runs, the probe vehicle had arrived during green time or onset of red, and then, the results would have varied greatly, and hence, this method should be used only as a crude estimation of delay, even then a sufficient number of runs are needed for computing delay. Coming to queue length-based delay estimation using simpsons 1/3rd rule, the total delay at undersaturated intersections is computed based on the queue length estimated according to the procedure given in HCM 2010 [3, 23]. Finding area under the queue length curve yield total delay from which, dividing by number of vehicles, yields average delay. But, this method assumes constant arrival and departure rate. Although departure rate may be assumed to be constant as following, the shockwave theory (if considering PCU/h) till the queue at stationary bottleneck (stop line) is

Table 4 Delay values respective to various estimation approaches

Method	Individual parameter		Control delay (s/veh.)	MAPE (%)
	Category	Avg. delay (s/veh.)		
Trajectory	Auto	25.29	24.87089	
	Bike	21.88		
	Bus	31.16		
	Car	25.97		
	LCV	20.06		
V-box		24.83	24.833	0.15
Simpsons 1/3 rule	Cycle 1	22.89	24.97751	0.43
	Cycle 2	26.31		
	Cycle 3	24.12		
	Cycle 4	26.59		
Webster's	Uniform delay	25.92	26.78	3.07
	Random delay	0.86		
Akcelik's model	Overflow delay	1.24	25.92032	4.3
	Uniform delay	25.92		
Modified Webster's		22.55	22.55236	9
Indo-HCM	d_1	27.59	27.59278	12

dissipated, but arrival rate is not constant. Also, during the onset of green, it may happen that the queue length during last interval of red time and first interval of green both may show almost similar peaks. In this instance, consideration of peak though total area remains same (assuming triangular or trapezoidal shape), the delay results vary significantly. But, since Simpsons 1/3rd rule uses, the integral function and area by parts to measure the area under curve, the results though obtained from complex methods provide very good results. Postprocessing of trajectory data enables us in determining the queue length with ease.

As for the delay estimation models, all the methods used are in one way or other, a derivative of Webster's delay model. But as described in literature review, these models have their limitation and shortcomings. In this study, the location selected was undersaturated during the observation period, and hence, no overflow delay or higher degree of saturation ratios was observed. Also, Indo-HCM model though based on Webster's model, at present is only calibrated for under saturated intersections (hence, effect of d_3 is ignored). Moreover while computing random delay parameter (d_2), the upstream filtration factor is considered to be unity. Indo-HCM considers all the intersection to be isolated.

5 Conclusion and Summary

Intersections are the critical nodes in the road network where the vehicles have to compromise with the travel quality and face delay in order to efficiently traverse through the section without any conflicts. Measure of effectiveness of the intersection is measured in terms of delay. According to literature review, several methods exist for computing the delay. Depending on the field situation, objective and equipment available one or the other method can be implemented for delay estimation. Fortunately, the results for this study has proven all the methods to be well versed in computing control delay at signalized intersection. But, this is only limited to undersaturated intersections which the study location is characterized with. As discussed in Sect. 4, each approach has some restrictions and limitations, but with precision-wise assured results and ease in computation, trajectory-based approach stands as the best method to estimate the control delay, in addition to the data which can further be used for all types of studies applicable to the study section.

With weak lane-based traffic, compromised compliance levels (rarely), blockage by right turning vehicles, spillover at way after stop line, the delay estimated by the models might turn out to be significantly underestimated at higher levels of saturation ratios. Hence, the authors emphasize that with the added advantages and immeasurable potential the trajectory data holds, the method should be explored and implemented. Maybe, by visualizing the field conditions in terms of numbers comprising of all the conditions, the analysis using trajectory data may lead toward more robust and dependable models for delay measurement.

References

1. MoRTH (2017) Road Accidents in India. Ministry of Road Transport and Highway, Government of India
2. Arkatkar S, Mitra S, Mathew T (2019) International practice on road traffic signal control chapter 12: India. In: WCTRS book series
3. Transportation Research Board, Highway Capacity Manual (2010) Interrupted flow, vol 3
4. Othayoth D, Krishna Rao KV (2018) Estimation of stopped delay to control delay conversion factor and development of delay model for non-lane based heterogeneous traffic. Euro Transp 69:1–21
5. CSIR-Central Road Research Institute, New Delhi (2017) Indian highway capacity manual. CSIR: Central Road Research Institute, New Delhi
6. Webster FV (1958) Traffic signal settings. H.M. Stationery Office
7. Akcelik R (1980) Time-dependent expressions for delay, stop rate and queue length at traffic signals
8. Mathew TV (2014) Signalized intersection delay models. Transp Syst Eng:1–18
9. Hoque S, Imran A (2007) Modification of Webster's delay formula under non-lane based heterogeneous road traffic condition. J Civ Eng 35(2):81–92
10. Gundaliya PJ, Raval NG (2012) Modification of Webster's delay formula using modified saturation flow model for non-lane based heterogeneous traffic condition. Highw Res J 5:41–48
11. Saha A, Chandra S, Ghosh I (2017) Delay at signalized intersections under mixed traffic conditions. J Transp Eng Part A Syst 143(8):17p

12. Quiroga CA, Bullock D (1999) Measuring control delay at signalized intersections. J Transp Eng 125(4):1–16
13. Liu K, Yamamoto T, Morikawa T (2006) Estimating delay time at signalized intersections by probe vehicles. In: Proceedings of traffic and transportation studies, pp 615–624
14. Ko J, Hunter M, Guensler R (2008) Second-by-second GPS speed data. J Transp Eng 134:338–346
15. Montanino M, Punzo V (2015) Trajectory data reconstruction and simulation-based validation against macroscopic traffic patterns. Transp Res Part B Methodol 80:82–106
16. Wei Y, Li K, Tang K (2019) Trajectory-based identification of critical instantaneous decision events at mixed-flow signalized intersections. Accid Anal Prev 123:324–335
17. Wang Z et al (2014) Visual exploration of sparse traffic trajectory data. IEEE Trans Vis Comput Graph 20(12):1813–1822
18. Olszewski P et al (2016) Pedestrian safety assessment with video analysis. Transp Res Proc 14:2044–2053
19. Munigety CR, Vicraman V, Mathew TV (2014) Semiautomated tool for extraction of microlevel traffic data from videographic survey. Transp Res Rec J Transp Res Board 2443(1):88–95
20. Punzo V, Borzacchiello MT, Ciuffo B (2011) On the assessment of vehicle trajectory data accuracy and application to the next generation SIMulation (NGSIM) program data. Transp Res Part C Emerg Technol 19(6):1243–1262
21. Cheng Y, Qin X, Jin J, Ran B, Anderson J. Cycle-by-cycle queue length estimation for signalized intersections using sampled trajectory data, pp 87–94
22. Li F, Tang K, Yao J, Li K (2017) Real-time queue length estimation for signalized intersections using vehicle trajectory data. Transp Res. Rec J Transp Res Board 2623:49–59
23. Transportaion Research Board (2010) Highway capacity manual 2010—volume 1: concepts