DEVELOPMENT AND CALIBRATION OF ENTRY CAPACITY MODEL OF MODERN ROUNDABOUTS

Ph.D. THESIS

by

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JULY, 2016

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A THESIS

Submitted in partial fulfilment of the requirements for the award of the degree of

DOCTOR OF PHILOSOPHY

in

CIVIL ENGINEERING

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ROORKEE

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled **"DEVELOPMENT AND CALIBRATION OF ENTRY CAPACITY MODEL OF MODERN ROUNDABOUTS"** in partial fulfilment of the requirements for the award of the degree of Doctor of Philosophy and submitted in the Department of Civil Engineering of the Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from July, 2012 to July, 2016 under the supervision of Dr. Rajat Rastogi, Associate Professor, Civil Engineering Department, Indian Institute of Technology Roorkee, Roorkee, India.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other Institute.

(**ABDULLAH AHMAD**)

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

(RAJAT RASTOGI)
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The Ph.D. Viva-Voce Examination of **Mr. Abdullah Ahmad**, Research Scholar, has been held on ……………………….

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ABSTRACT

Traffic conditions in India are highly heterogeneous due to variety of vehicles with different static and dynamic characteristics sharing and operating at same road space. Roundabouts are a type of intersections which commonly used as a means of intersection control for moderate traffic flows. These adapt to junctions having variations in the intersection geometry. Efficiently designed roundabouts can handle traffic very smoothly without causing any delay. It facilitates an orderly movement of traffic in a circular motion around a central island whose shape is decided based on intersection geometry. The circulating traffic is considered to be the priority stream and entering traffic shall wait for a suitable gap in the circulating traffic. In this fashion, it reduces the stopped delays as observed on the signalized intersections, and thus improves the operational efficiency. The analysis and design of roundabouts in India is governed by IRC:65-1976. The conversion factors for heterogeneous traffic to homogeneous one are quite old and need validation. The capacity of the roundabout is based on weaving section. This approach has already been replaced by estimation of entry capacity. A look on the literature in this respect indicates that most of the studies are from US and other developed nations where homogeneity of traffic stream and lane discipline of drivers are two important characters of traffic flow. Very few attempts have been made in India to evaluate the entry capacity on roundabouts under mixed traffic and untidy behavior of drivers.

Data were collected at eleven roundabouts spread across cities of Chandigarh, Noida and Panchkula. Geometric parameters were measured manually with measuring tape while traffic data was collected by means of videography. It required four types of analysis, namely estimation of passenger car units, estimation of critical gap for different categories of vehicles, calibration of HCM model based on estimated values of critical gaps under heterogeneous traffic conditions and, estimation of entry capacity model based on the field data (traffic and geometric).

The estimation of passenger car unit (PCU) for different vehicles to convert heterogeneous traffic into homogeneous traffic is a well-accepted procedure. But the parameters used for mid-blocks may not be helpful on roundabouts as traffic flow

characteristics on the two locations are different. Indian Roads Congress code (IRC-65) recommends static PCU values which is based on studies prior to 1976. Since then vehicle technology has changed. This study re-looks on the PCU values based on field studies and suggest the modified ones. The PCU for a vehicle is estimated based on lagging headway and width of the vehicle, to account for vehicle size and untidy flow conditions. It is also not clear whether to use static or dynamic PCU values on account of possible temporal and spatial variations across locations. *It was found that PCU value for motorized two-wheeler reduced by half of its value given in IRC:65-1976. The car category got divided as small and big car. The PCU value of heavy vehicle got increased marginally.* The problem to deal with re-estimation of PCU values at different locations, due to possible traffic flow variations, is dealt with by proposing a Heterogeneity Equivalency Factor (H-Factor). The factor is multiplicative and converts heterogeneous traffic (veh/h) into homogeneous traffic (pcu/h).

Estimation of critical gap for a vehicle type under mixed traffic conditions prevailing in developing countries has been always a challenging task. This is due to the poor lane discipline and very limited priority being followed by the vehicles at priority intersections like roundabouts. A simple procedure, which is based on minimization of the sum of absolute difference between a gap value and accepted / rejected gap, is proposed in this study. The iterative procedure provides a value of gap that is termed as the critical gap under mixed traffic conditions. The method is different from maximum likelihood method (MLM) in two aspects. First, it does not assume any predefined distribution for the critical gap and second, it does not fail even when there are very few rejected gaps. Prominent methods available in literature to estimate critical gap are compared for different categories of vehicles. Based on the results of consistency test, the MLM and the proposed method are found to be the most acceptable estimation methods. *It has been further observed that the proposed method is better than MLM when working with low sample size, as well as, in no-priority conditions, which arise due to heterogeneous traffic flow prevailing in developing countries.*

The entry capacity of a roundabout against variation in circulating flows is being assessed. Queue formation in the approach is taken as an indicator that the approach is operating at the capacity. The normal notion considered is that as circulating flow decreases, the entry flow should increase. This may be due to the higher opportunities being made available to the vehicles desiring to enter the circulation area. *Linear and exponential functions are found to be showing goodness-of-fit between entry capacity and circulating traffic flow.* To find relation between entry capacity and geometric parameters, entry capacities have been plotted against central-island diameter, circulating roadway width and entry width. Power function is found to provide the best fit between entry capacity and central-island diameter, circulating roadway width and entry width. The variation or dispersion of data is found to be quite low for entry capacity and central-island diameter and is a bit high for the other two relationships.

The Highway Capacity Manual of US (HCM 2010) has given gap acceptance model of entry capacity for single-lane and two-lane roundabouts in U.S. This manual is extensively used in different parts of the world for estimating the capacity of a traffic facility. However, the direct transferability of the HCM (2010) entry capacity model to Indian traffic flow conditions was doubtful as the manual do not consider driver behavior under mixed traffic flow. Therefore, the parameters of the equation were calibrated for its adaptation to heterogeneous traffic conditions, using critical gap values obtained from the field data. The modified HCM model for entry capacity was still found differing from the field entry capacity. *Therefore, multiplicative adjustment factors for different size of roundabouts have been developed for modified HCM (2010) equations to satisfy the traffic flow condition prevailing in the developing countries like India.*

A regression model for estimating roundabout entry capacity was developed based on the traffic flow and roundabout geometric data. The analysis indicated that the widths of circulating roadway and central-island diameter have a significant influence on the entry capacity. *The developed regression model is validated on another roundabout and only ±6 percent difference was observed between the field entry capacities and those predicted by the proposed model.* Sensitivity analysis is used to see the effect of physical parameters of the roundabout on entry capacity. It was found that the central-island diameter has the greatest effect on entry capacity while the circulating roadway width has the smallest effect. The circulating traffic flow versus entry capacity charts were made with the purpose of comparing the results of the proposed model with the existing models available in the literature, namely Jordanian, Malaysian and Indian

(Prakash 2010). The comparison indicated that the proposed entry capacity model gave the highest entry capacity, whereas, the Malaysian model gave the lowest entry capacity. The results of the proposed model have shown very good relationship with the field entry flow data, as compared to other models. *The range of entry capacity has been found varying between 3000 to 4000 pcu/h for different size of roundabouts. Lower limit for entry capacity is expected to be ranging between 800 to 1200 pcu/h.*

Keywords: Roundabout, heterogeneous traffic, passenger car unit, gap acceptance, critical gap, entry capacity, HCM, regression analysis, India

ACKNOWLEDGEMENT

I express my sincere gratitude to **Dr. Rajat Rastogi**, Associate Professor, Department of Civil Engineering, I.I.T, Roorkee, for his expert guidance, motivation, and continuous support throughout my research work. He delivered a constant inspiration, positive thoughts and creative ideas in different stages of this research work. His unforgettable suggestions and valuable remarks gave me directional strength to accomplish my research work within required time frame. His immense knowledge has provided me an insight to accomplish research and helped me in different manners during midst of the greatest difficulty. I am greatly benefited from his teaching, scientific approach and ability to put complex ideas into simple expressions. I could not have imagined having a better guidance for my Ph.D thesis. I thankfully acknowledge him for his blessings and giving me an opportunity to work under his exalted guidance.

I express my deepest gratitude to **Dr. Satish Chandra**, Professor, Department of Civil Engineering, I.I.T, Roorkee, for sharing his vast knowledge through intellectual negotiations for carrying out this research work. I would be very thankful for his massive support for conducting various field traffic surveys.

I express my gratitude to **Dr. C.S.P. Ojha**, Professor and Head, Department of Civil Engineering, I.I.T Roorkee for his enormous help in providing good library and laboratory facilities, and healthy research environment in the department. I would like to thank Chairman of my Student Research Committee (SRC) **Dr. Praveen Kumar**, Professor, Civil Engineering Department, and members of SRC – **Dr. Vishwas A. Sawant**, Associate Professor, Department of Civil Engineering and **Dr. Tanuja Srivastava**, Professor, Department of Mathematics, for their support, valuable suggestions, inputs and guidance during various stages of my research work. I would express sincere gratitude to other faculty members of Civil Engineering Department **Dr. M. Parida**, **Dr. S.S Jain, Dr. G.D. Ransinchung R.N and Dr. Indrajit Ghosh** for their moral support during my research work.

I would like to express deep appreciation to my seniors and colleagues –**Hari Krishna, Haider Habeeb Aodah, Ashish Dhamaniya, Mithun Mohan, Abhshek Jindal, Udit Jain, Pawar Sachin and Wasim Akhtar** for their assistance, valuable guidance and cooperation. I am also very thankful to **Srinath Mahesh** for his assistance and support during data collection and field study.

I highly appreciate the financial support of **MHRD** (**New Delhi)** and **Indian Institute of Technology, Roorkee** for providing the funding for the research work. I sincerely appreciate the support provided by the office staff members of Civil Engineering Department. I would also like to express my sincere thanks to the Dean Academics, Dean of Students' Welfare; and Assistant Registrar (Academic) for their cooperation. I am also thankful to the staff of Academic and Accounts section, I.I.T Roorkee for their help and support during my research work.

I would like to express my thanks to the whole staff of Transportation Engineering laboratory for their sincere devotion, cooperation and overwhelming support for my research work.

I am very grateful to my parents for giving me strength, motivation and inspiration. I wish to express my indebted love and gratitude to my wife, who supported me through her enlightened heart during my stay in I.I.T Roorkee. I have no words to express my deep love and feelings to my daughters **Afeefa** and **Ifrah**, their charming smile also gave me constant inspiration to make this research work possible.

I would like to express my deepest gratitude to all those who have contributed directly or indirectly in the course of my research work and throughout my life. I am thankful to them for inspiring me through their enlightened hearts, for touching me through their acts of kindness and honesty, and illuminating me through their wisdom and knowledge.

(ABDULAAH AHMAD)

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ABBREVIATIONS

2W	Motorized Two Wheeler
3W	Motorized Three Wheeler
Abs	Absolute
AWSC	All Way Stop Controlled
BC	Big Car
CRRI	Central Road Research Institute
CAGR	Compound Annual Growth Rate
DCU	Dynamic Car Unit
DTU	Dynamic Two-wheeler Unit
FHWA	Federal Highway Administration
FreSim	Freeway Simulation
H-factor	Heterogeneity Equivalency Factor
HCM	Highway Capacity Manual
HeteroSim	Heterogeneous Simulation
HV	Heavy Vehicle
IHCM	Indonesian Highway Capacity Manual
IRC	Indian Roads Congress
LCV	Light Commercial Vehicle
LOS	Level of Service
MATLAB	MATrix LABoratory
MEU	Motorcycle Equivalent Unit
MHRD	Ministry of Human Resources and Development
MLM	Maximum Likelihood Method

MORTAB	MOdel for depicting Road TrAffic Behavior
MS EXCEL	MicroSoft Excel
NCHRP	National Cooperative Highway Research Program
NN	Neural Network
NMT	Non-Motorized Traffic
PCE	Passenger Car Equivalent
PCU	Passenger Car Unit
PCU/h	Passenger Car Units per hour
QDF	Queue Discharge Flow
R^2	Coefficient of determination
SC	Small Car
SIDRA	Signalized Intersection Design and Research Aid
TEF	Taxi Equivalence Factor
TRANSYT	Traffic Network and Isolated Intersection Study Tool
TRW	Transport Research Wing
TWSC	Two Way Stop Controlled
U.K.	United Kingdom
U.S.	United State
Veh/h	Vehicle per hour
VISSIM	VISual traffic SIMulation
VMS	Vehicle Moving Space
VTI	Vehicle Track Interaction

CHAPTER 1:
INTRODUCTION

1.0 INTERSECTION: AN ENTITY

An intersection is defined as an area where two or more highways join or cross each other. This includes the roadway and roadside facilities needed for traffic movements within the area. This area is designated for movement of the vehicles in different directions, depending upon the number of approach legs. Four-legged intersection is the most common intersection at which two highways cross each other. Main function of the intersection is to guide vehicles to their respective directions. The primary objective is to provide for the convenient, easy, comfortable, and safe movement of vehicles and other users through the intersection while reducing potential conflicts between motorized vehicles, bicycles, and pedestrians. High or low volume intersections, if not controlled properly, are considered the most risky locations in terms of conflicts. Intersection design should be fitted closely to the natural transitional paths and operating characteristics of its users.

The intersections may be categorized as un-channelized and channelized intersection. Un-channelized intersection places no restriction on the vehicles in using any portion of the intersection area. The assumption is that vehicles will follow the governing rules of movement through the intersection. Channelization is achieved by constructing islands into the intersection area, with an aim to reduce the conflict points and area. Roundabout is a type of channelized intersection where traffic moves around a central-island, clockwise for left-side driving and anti-clockwise for right-side driving. The shape of the island at the center is modulated to synchronize with the traffic flowing around it, as well as, with the orientation of intersecting legs. In case the roundabout is laid with four mutually perpendicular approaches, the best shape is circular. The size of the roundabout is kept such that the traffic has to slow down while entering it and traversing along the central island. At the same time, the exit is designed in a manner that exiting vehicle can attain and move at higher speeds.

Roundabouts are used as intersection control measures for a number of traffic and site conditions as they do not require any active control in terms of the presence of a traffic police personal. For moderate traffic, roundabouts increase the capacity of an intersection and improve its performance by reducing delays and crashes. Reduction in the number and severity of crashes is due to the decrease in the number of conflict points, as a result of channelization. The traffic enters a roundabout after seeking a suitable gap in the circulating stream of vehicles thereby, the crossing conflicts which are the most severe are completely eliminated and converted to merging and diverging. Quantitatively, the number of conflict points reduce from 32 in a Two-Way Stop Controlled (TWSC) intersection to 8 in the case of a roundabout as shown in Figure 1.1. Besides, during low flows, there is less likelihood of crashes due to over-speeding of vehicles as there are inherent geometric features in the roundabout which discourage high vehicle speeds.

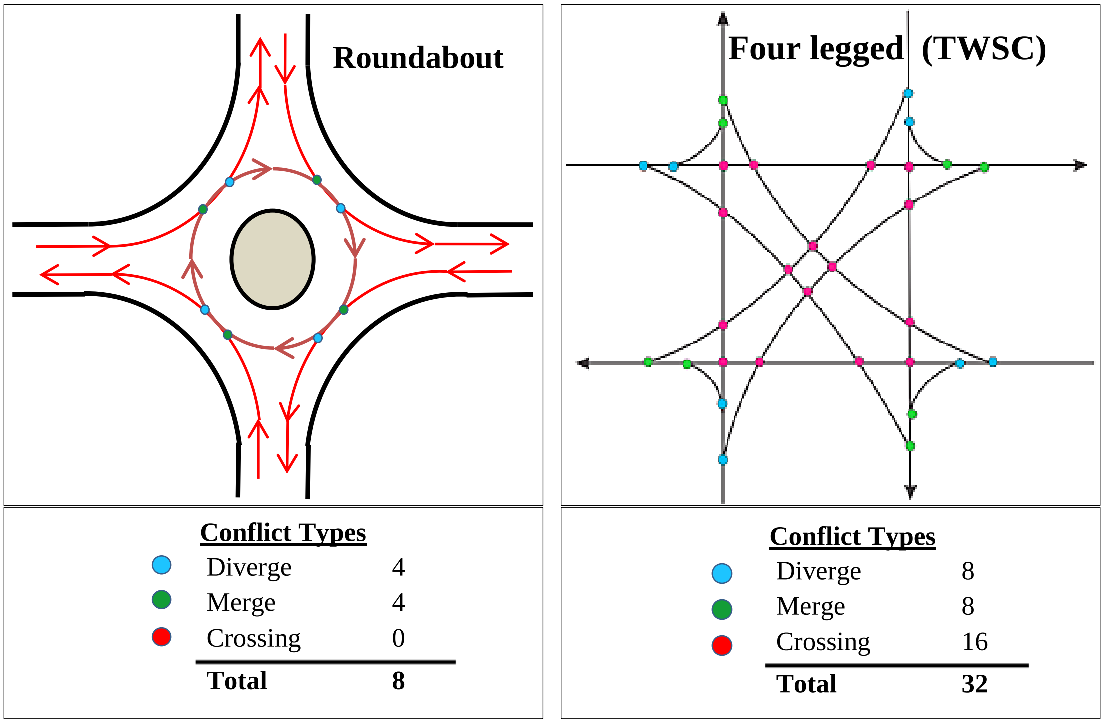
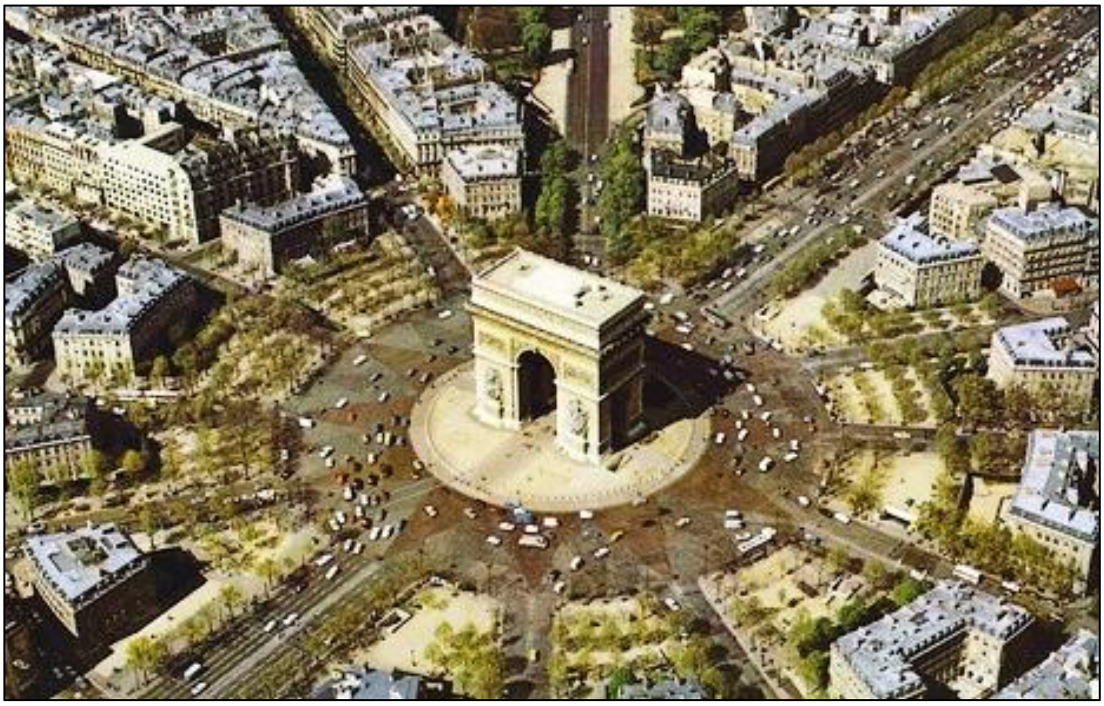


Figure 1.1 Vehicle-vehicle conflict points at an intersection

1.1 EVOLUTION OF ROUNDABOUTS

The first circular intersection, the Columbus Circle, was introduced by William Phelps Eno in New York in 1905. The first roundabout in Paris was constructed in 1907 at De l’etoile. In the UK, amid 1925-1926, roundabouts were constructed in London, at Aldwych, Parliament Square, Hyde Park Corner, Marble

Arch, and Trafalgar Square. Some of these roundabouts are shown in Figure 1.2. These are taken up from archives to despite their real orientation and vehicular flows on them.



(a) Roundabout at De l'etoile

(b) Roundabout at Trafalgar Square

Figure 1.2 Roundabouts during 1905-1950

Around 1950s, because of the issue of locking of roundabouts and an increasing number of accidents resulting from drivers defying the traffic rule, there was a loss of confidence in roundabouts as an effective form of intersection control. Enhancements in traffic signals and the development of coordinated traffic signal networks also made roundabouts less preferable and many of them were subsequently replaced. In Germany, roundabout failure was due to a lack of suitable capacity estimation, a high accident rate and congestion due to the misinterpretation of the priority rules (Brown 1995). In 1966, the survival of roundabouts in the UK was enhanced with the new allocated off-side priority rule (an entering vehicle gives way to circulating vehicles) and the yield-at-entry operation. With this new priority rule, entry was now controlled by the ability of entering drivers to detect gaps in the circulating flow. An entering vehicle simply merged into any acceptable gap in the circulating flow and diverged as it reached the desired exit. This prevented vehicle from entering when no gap in the circulating stream was available, thus avoiding the locking problem of the roundabout. Moreover, the capacity of roundabouts was no more dependent on the weaving operation, but on the availability of gaps. This increased both the capacity and safety of roundabouts (Taekratok 1998).

Roundabouts have been utilized for controlling traffic for quite a few years though the operating principle has changed over the long run. The initial circular intersections were designed for the circulating traffic to yield to the approaching traffic from a leg. The modern roundabout changed the operating principle to a yield for all entry vehicles, which merged only when there was an acceptable gap in the circulating traffic. Also the size of the modern roundabout was considerably reduced and designed to calm traffic before merging into the circulation lane. This rule prevented traffic from locking and permitted free-flow movement on the circulatory roadway and more importantly it changed the task of the driver from merging and weaving at high speed to the task of accepting a gap in the circulating traffic while moving at low speed. The slower speed also made it unnecessary to design roundabouts with bigger radii, making the modern roundabout a possible alternative in urbanized zones where accessible area is constrained (Mensah et al. 2010).

Roundabouts have distinct features which separate them from the broader class of rotary junctions referred to as traffic circles. Including deflection and yield on entry, roundabouts also differ from traffic circles in that they prohibit parking on the approaches or within the circulating roadway. They also do not have stop signs or yield signs within the circulating roadway (Flannery and Datta 1996a). In the United States (US), the circular intersections were classified into three categories such as rotaries (traffic circles), neighborhood traffic circles and roundabouts. Their description, as used in the US (FHWA 2000) is given below:

Rotaries (Traffic circles): Traffic circles have been part of the transportation system in the United States, since 1905 when one of the first circles, known as the Columbus Circle in New York City, was constructed. This is shown in Figure 1.3. The prevailing designs enabled high-speed merging and weaving of vehicles. Priority was given to entering vehicles. These are characterized by a large diameter, often in excess of 100 m. This large diameter typically resulted in travel speeds within the circulatory roadway that exceeded 50 km/h.

Figure 1.3 Columbus Circle, New York city (Andrew-Prokos-Photography 2015)

Neighborhood traffic circles: Neighborhood traffic circles are typically built at the intersections of local streets for reasons of traffic calming and/or aesthetics as shown in Figure 1.4. The intersection approaches may be uncontrolled or stop-controlled. They typically do not include raised channelization to guide the approaching driver onto the circulatory roadway.

Figure 1.4 Neighborhood Traffic Circle (Blog.fusedgrid.ca 2015)

Roundabouts: Roundabouts are circular intersections with specific design and traffic control features. These features include yield control for all entering traffic, channelized approaches, and appropriate geometric curvature to ensure that travel speeds on the circulatory roadway are typically less than 50 km/h. These features allow only low speeds and create safe driving condition. Vehicles in the roundabout will have a priority over the entering vehicles. Aerial view of the Roundabout in Chandigarh City, India is shown in Figure 1.5.

Figure 1.5 Roundabout in Chandigarh city, India (March, 2013)

Roundabouts are a type of circular intersections, as are traffic circles and rotaries. The common characteristics distinguishing a modern roundabout from a traffic circle or a rotary type intersection are given in Table 1.1. The comparison between Roundabout and Rotary is also shown in Figure 1.6.

Table 1.1 Distinguishing characteristics of a roundabout and a traffic circle

S.N.	Feature	Traffic Circle or Rotary	Roundabout
1	**Design Philosophies**	Rotary geometry encourages high-speed merging and weaving, made possible by larger diameters and large high-speed entry radii	Roundabouts control and maintain low speeds for entering and circulating traffic. This is achieved by relatively small diameters and low-speed entry geometry
2	**Control of Vehicle Speed**	Geometric design elements allow high speed entries therefore creating highly risky driving conditions	Geometric design elements allow only slow speeds and create condition for safe driving

3	**Approach Geometry**	The traffic enters the intersection tangentially	The traffic enters the intersection in a direction normal to the diameter of the central-island
4	**Traffic Control**	Some traffic circles use stop control, or no control, on one or more entries	Yield control is used on all entries, but circulatory roadway has no control
5	**Priority to Circulating Vehicles**	Some traffic circles require circulating traffic to yield to entering traffic	Vehicles in the circulating area will have a priority over the entering vehicles
6	**Deflection**	Entry angle likely to be reduced to allow higher speed at entry	Large entry angle helps to control speed through the roundabout
7	**Circulating Speed**	Higher speeds allowed (> 50 km/h)	Maintain relatively low speeds (< 50 km/h)
8	**Circle Diameter**	Larger diameters	Larger to Smaller diameters
9	**Splitter Island**	Optional	Required
10	**Parking**	On large traffic circles, occasional parking is permitted within circulating roadway	No parking is allowed within the circulatory roadway or at the entries
11	**Pedestrian Access**	Some traffic circles allow pedestrian access to the central-island	Pedestrian access is allowed only across the legs of the roundabout, behind the yield line

Source: (WDOT 2011)

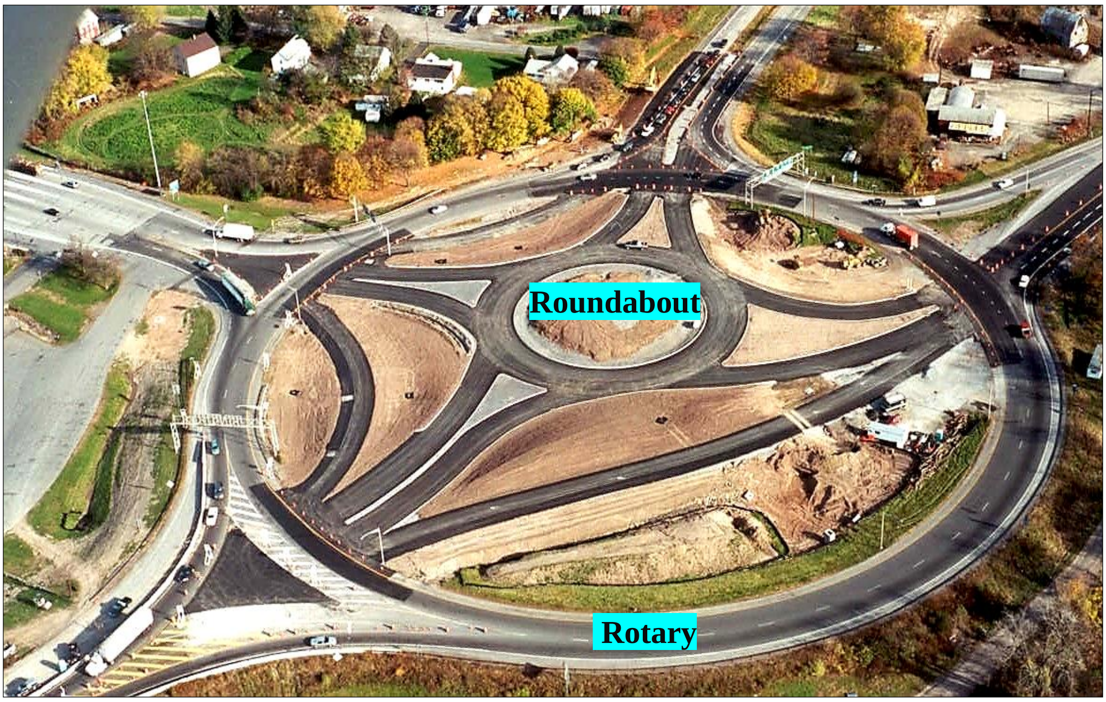


Figure 1.6 Distinguishing roundabout from a rotary or a traffic circle (NCHRP Report-672, 2010)

Roundabouts have some advantages and disadvantages in several traffic conditions when compared with other intersections. These are given in Table 1.2.

Table 1.2 Operational merits and concerns on roundabouts (Taekratok 1998)

S.N.	Category	Merits	Concerns
1	**Safety**	Reduced numbers of conflict points as compared to an uncontrolled intersection. Chances of accidents are less, and even if it occurs, the severity is low.	Crashes may provisionally increase at roundabout due to improper driver education. Signalized intersections can preempt control for emergency vehicles.
2	**Vehicle Flow Capacity**	Nonstop and continuous traffic flows give higher flow capacities.	The flow at capacity may cause lock-up condition. Once roundabout has locked-up, the movement of vehicles will be completely stopped.

3	**Traffic Movement**	None of the vehicles need to be stopped, unlike in a signalized intersection. All turns can be made with ease.	Right turn traffic have to travel extra distance.
4	**Delay**	During the off-peak period, signalized intersections without traffic actuation produce unnecessary delays to stopped traffic even though large gaps in the other flows are available.	Drivers may not like the geometric delays which force them to divert their vehicles from straight paths. When queuing develops, entering drivers tend to force into the circulating streams with shorter gaps. This may increase the delays on other legs.
5	**Cost**	For moderate traffic, no traffic control is required and hence operation is economical, whereas, maintenance costs of signalized intersections include electricity, maintenance of loops, signal heads, controller and timing plans.	Construction costs may be higher. In some locations, roundabouts may require more illumination, thus increasing costs.
6	**Environment**	Traffic proceeds at a fairly uniform speed. Frequent stopping and starting are avoided consequently air pollution reduces.	Extended vehicle presence due to geometry and locking of roundabout may increase pollution concentration.

1.1.1 Usability of the Roundabouts

Intersections, with the following flow related characteristics, can potentially benefit from the implementation of a roundabout (PENNDOT 2001):

a) Heavy delay on a minor road
b) Large delays due to traffic signal
c) Heavy traffic flows requiring right turning
d) More than four legs or unusual geometry
e) History of crashes involving crossing traffic
f) Traffic growth expected to be high
g) Future traffic patterns uncertain or changeable

h) Need for U-turns
i) History of right angle crashes

1.2 TYPES AND DESIGN

1.2.1 Types of Roundabouts

There are three basic categories based on environment, number of lanes, and size (NCHRP Report-672, 2010):

a) Mini-roundabouts
b) Single-lane roundabouts
c) Multi-lane roundabouts

The following section provides a qualitative discussion on each category.

Mini-Roundabouts: Mini-roundabouts are small roundabouts with a fully traversable central-island. Figure 1.7 shows the features of a typical mini-roundabout. They may be useful in environments where conventional roundabout design is impossible by right-of-way constraints. Mini-roundabouts are relatively inexpensive because they typically require minimal additional pavement at the intersecting roads and minor widening at the corner curbs. They are mostly recommended when there is insufficient right-of-way to accommodate the design vehicle with a traditional single-lane roundabout. Because they are small, mini-roundabouts are perceived as pedestrian-friendly with short crossing distances and very low vehicle speeds on approaches and exits. A fully traversable central-island is provided to accommodate large vehicles and serves as one of the distinguishing features of a mini-roundabout. The mini-roundabout is designed to accommodate passenger cars without requiring them to traverse over the central-island.

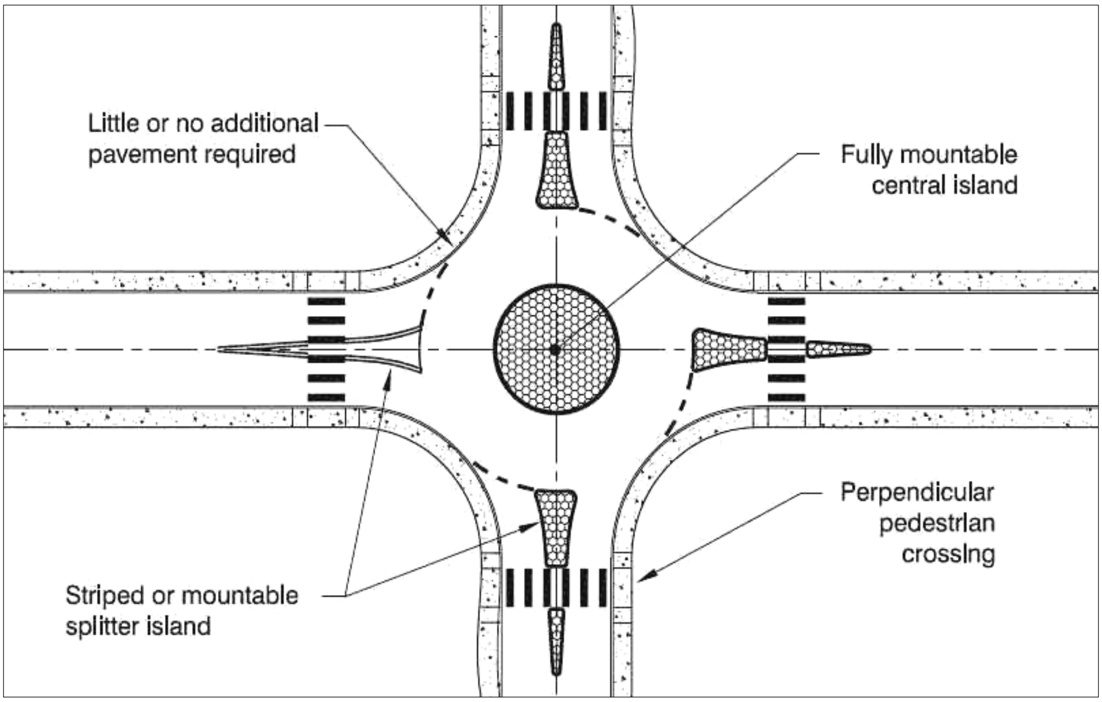


Figure 1.7 Features of a typical mini-roundabout

Single-Lane Roundabouts: This type of roundabout is characterized as having a single-lane entry at all legs and one circulatory lane. Figure 1.8 shows the features of a typical single-lane roundabout. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-traversable central-islands. Their design allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit. The geometric design typically includes raised splitter islands, a non-traversable central-island, crosswalks, and a truck apron. The size of the roundabout is largely influenced by the choice of a design vehicle and available right-of-way.

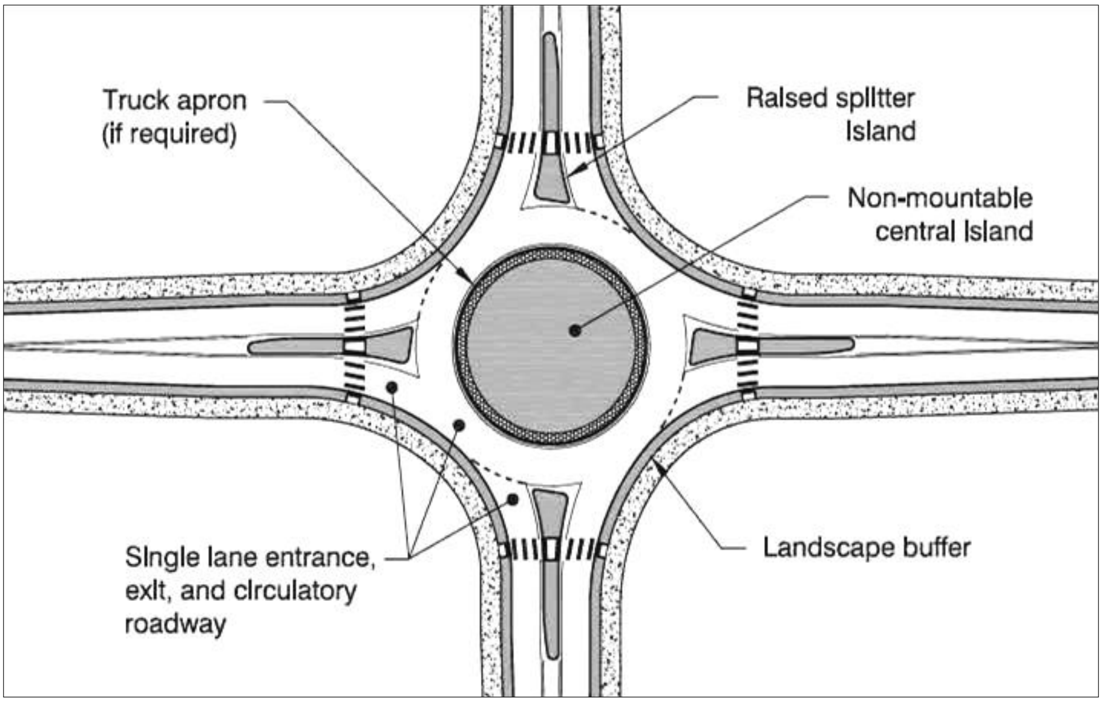


Figure 1.8 Features of a typical single lane roundabout

Multilane Roundabouts: Multilane roundabouts have at least one entry with two or more lanes. In some cases, the roundabout may have different number of lanes on one or more approaches (e.g., two-lane entries on the major street and one-lane entries on the minor street). They also include roundabouts with entries on one or more approaches that flare from one to two or more lanes. These require wider circulatory roadways to accommodate more than one vehicle travelling side by side. Figure 1.9 shows the features of a typical multi-lane roundabout. The speeds at the entry, on the circulatory roadway, and at the exit are similar or may be slightly higher than those for the single lane roundabouts. The geometric design will include raised splitter islands, truck apron, a non-traversable central-island, and appropriate entry path deflection.

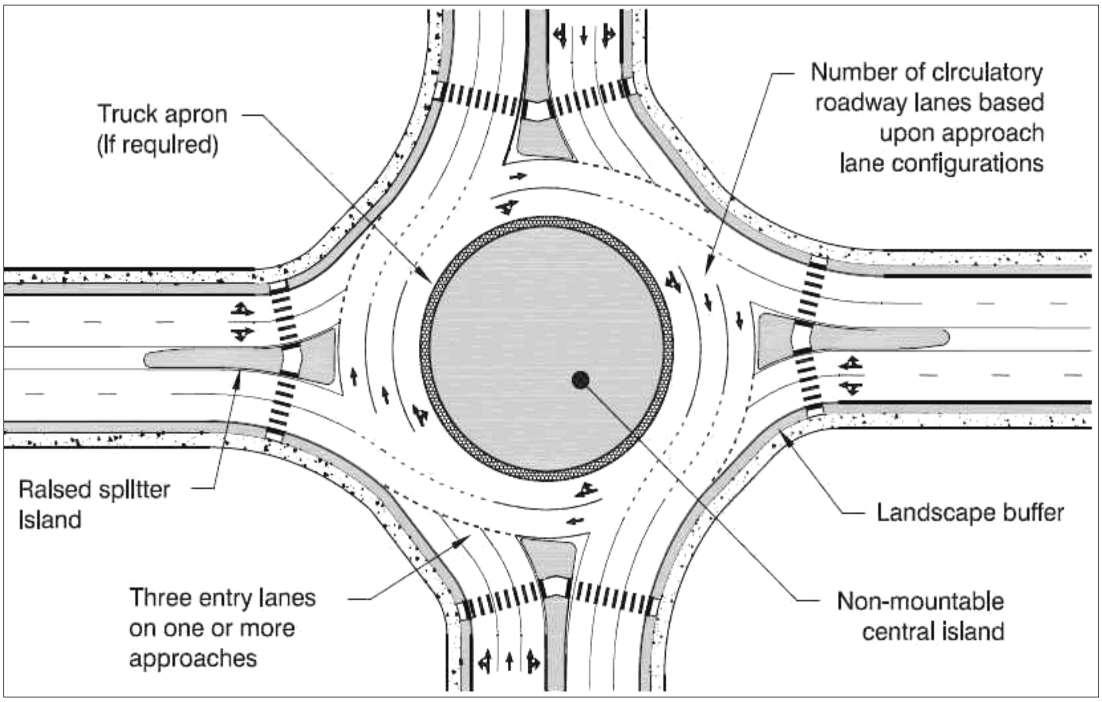


Figure 1.9 Features of a typical multi lane roundabout

1.2.2 Classification as per IRC:65-1976

IRC:65-1976 is an Indian code used for the design of rotaries / roundabouts (implemented as synonym) in India. This is the guideline issued by Indian Roads Congress (IRC), New Delhi, India. The design of the rotary is based on connecting the one-way entrance and exit roads to form a closed figure with at least the minimum weaving lengths interposed between two intersecting legs and then adjusting for the minimum radius of the rotary corresponding to the design speed. In doing so, it may be necessary to try out a number of alternatives, before selecting the best. While finalizing the shape of the rotary island, traffic streams within the rotary should be given dominance over the streams of traffic entering from different roads. Asymmetric shapes, either wholly curved or with a combination of straight and curves may often provide the only satisfactory solution. The possibility of realigning one or more of the intersecting legs could also be considered to achieve the minimum weaving lengths and the desired intersection angles. Some of the more common shapes and disposition of the rotary islands as suggested in this code of guidelines are discussed below.

Circular shaped rotary: A circular shape is suited where roads of equal importance intersect at nearly equal angles and carry nearly equal volume of traffic. Under these conditions, with a circular shape, a constant and regular flow is achieved. The circular shaped rotary is shown in Figure 1.10.

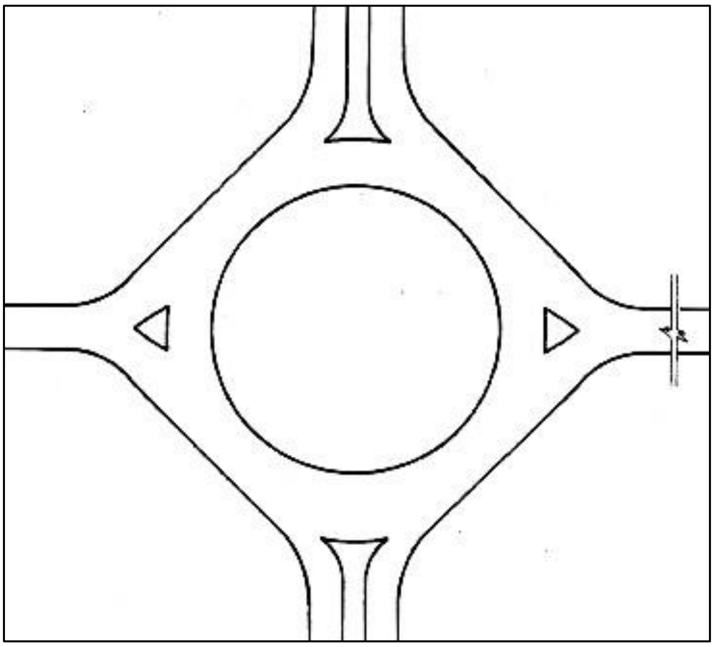


Figure 1.10 Circular shaped rotary

Squarish with round edges: This is a modification of the circular shape and is composed of four straights or four large radii curves roughly forming four sides of a square, and four small radii curves at the corners. The advantages of this layout is that it is suitable for predominantly straight ahead flows. The squarish rotary with round edges is shown in Figure 1.11.

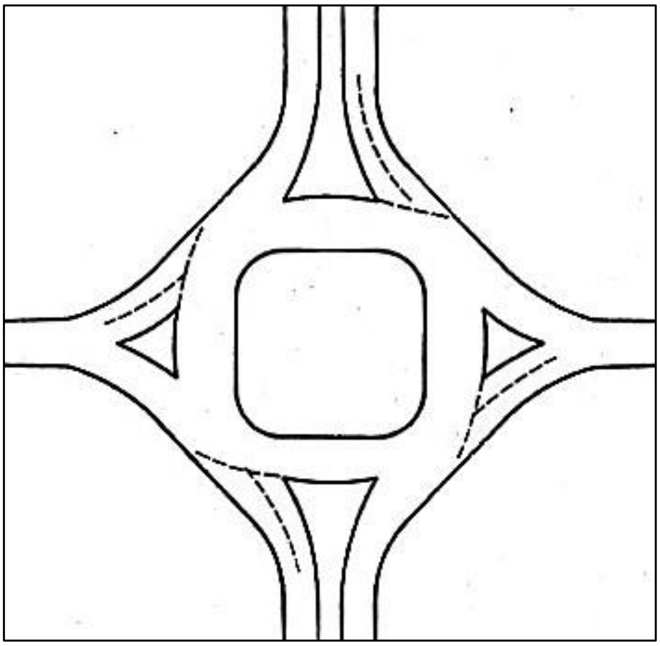
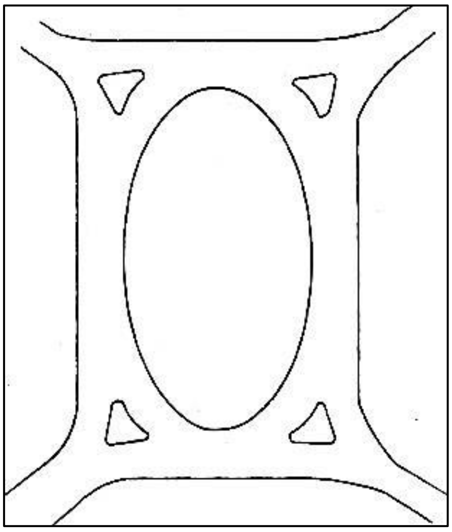
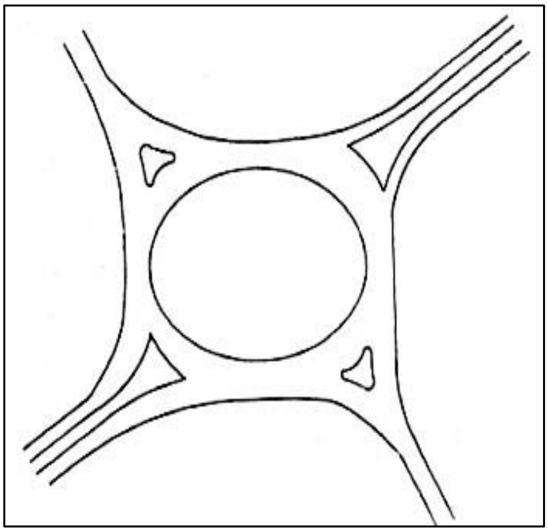


Figure 1.11 Squarish rotary with round edges

Elliptical, Oval or Rectangular Shapes: These types of shape are provided to favour through traffic, to suit the geometry of the intersecting legs, or to provide a longer weaving length for increasing the weaving capacity. These are shown in Figure 1.12.



(a) Elliptical rotary



(b) Rectangular rotary

Figure 1.12 Elliptical and rectangular shaped rotary

Complex intersection with many approaches: A layout of a complex intersection whose shape is dictated by the existence of a large number of approaches is shown in Figure 1.13.

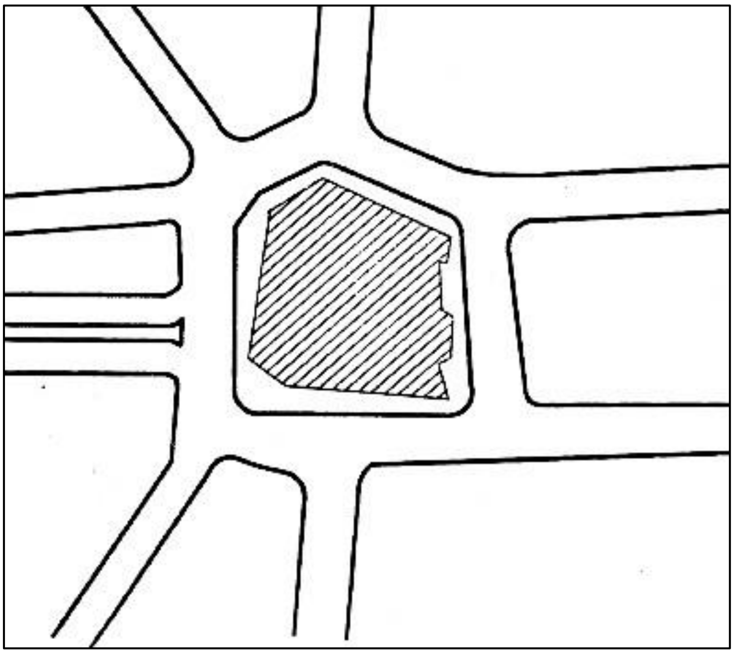


Figure 1.13 Layout of complex rotary intersection

1.2.3 Geometric Design Elements

The geometric design elements of a roundabout are given below. These are shown in Figure 1.14 also.

a) **Approach width:** The approach width is the width of a roadway used by vehicle in approaching traffic upstream.

b) **Departure width:** The departure width is the width of the roadway used by vehicle in departing traffic downstream.

c) **Central-island:** This is mostly circular in shape around which traffic circulates. This island may be raised or flushed (for mini-roundabouts), or it may have a raised central-island with a mountable or drivable truck apron surrounding it.

d) **Circulatory roadway:** It is the roadway around the central-island on which circulating vehicles travel in a clockwise direction for left-hand drive rule and anti-clockwise direction for right-hand drive rule.

e) **Entry width:** The entry width defines the width of the entry where it meets the inscribed circle.

f) **Exit width:** The exit width defines the width of the exit where it meets the inscribed circle.

g) **Entry radius:** The entry radius is the minimum radius of curvature of the outside curb at the entry.

h) **Exit radius:** The exit radius is the minimum radius of curvature of the left-side curb at exit.

i) **Inscribed circle diameter:** It is measured between the outer edges of the circulatory roadway.

j) **Circulating roadway width:** The circulatory roadway width defines the roadway width which is used for vehicle circulation around the central-island. It is measured as the width between the outer edge of this roadway and the central-island.

k) **Splitter Island:** A splitter island is a raised or painted area on an approach used to separate entering traffic from exiting traffic, as well as, it deflects and slows down entering traffic.

l) **Apron:** An apron is the increased portion of the central-island adjacent to the circulatory roadway. It may be required on smaller roundabouts to accommodate the wheel off-tracking of large size vehicles.

m) **Yield line:** A yield line is a pavement marking used to mark the point of entry from an approach into the circulatory roadway and is generally marked along the inscribed circle. Entering vehicles must yield to any circulating traffic coming from the right (for the left-hand driving rule) before crossing this line into the circulatory roadway.

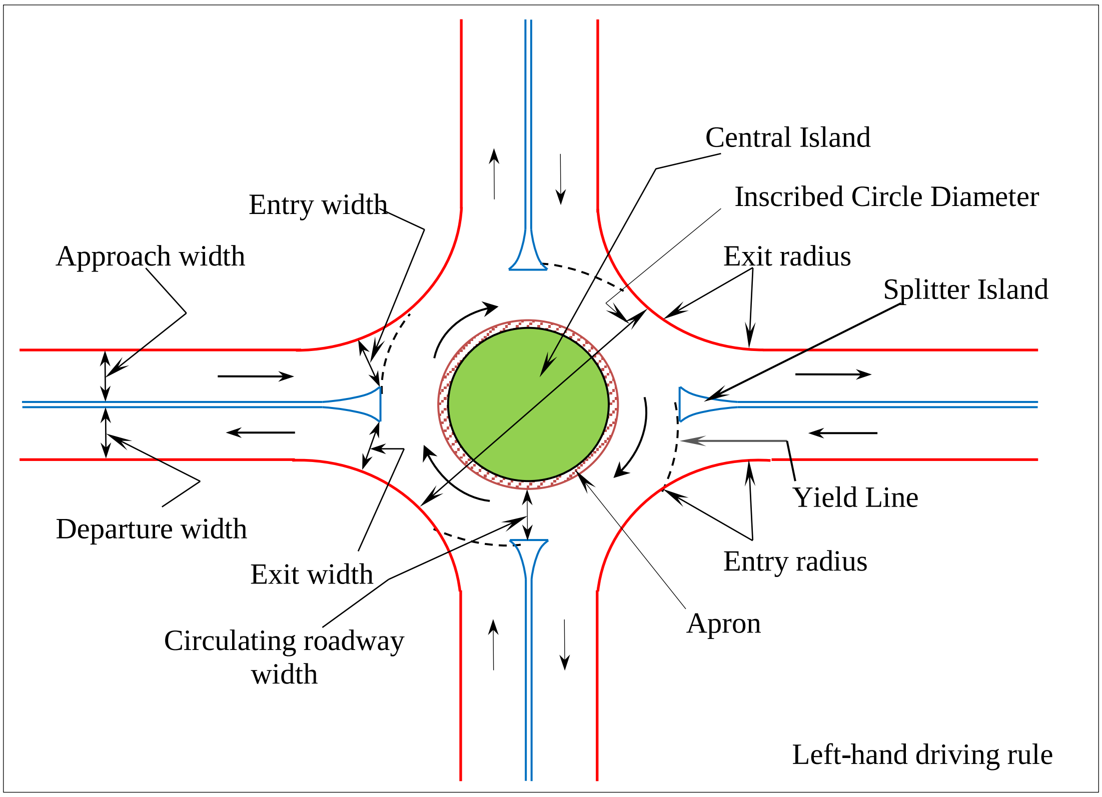


Figure 1.14 Typical geometric design elements of a roundabout

1.3 PROBLEM IDENTIFICATION

India has experienced rapid urbanization as the growth of urban population was 31.2% as compared to 17.9% growth of rural population in the last decade. With the present trend of urbanization, the number of cities with population of more than one million is expected to increase from 35 (2001) to 70 (2025). Accordingly, the share of urban population is expected to increase from 28 to 54% (Basu and Maitra 2010). India has also encountered a huge increment in the total number of registered motor vehicles as demonstrated in Figure 1.15. The total number of registered motor vehicles increased from about 0.3 million as on 31st March, 1951 to 159.5 million as on 31st March, 2012. The total registered vehicles in the country grew at a Compound Annual Growth Rate (CAGR) of 10.5% between 2002 and 2012. Amongst the different classifications of vehicles, the highest CAGR amid the period 2002 to 2012 was recorded by cars, jeeps and taxis (11%), followed by two-wheelers (10.7%) and goods vehicles (9.9%) (TRW 2013). The trend between 2001-2012 indicates an exponential increase rather than a linear trend. This certainly has implications on urban transport infrastructure facilities. The users are increasing and the performance of transport facilities is going down. In the case of roundabouts, it translates into higher cumulative delays due to quite low speeds or at times locking condition of the roundabout, as well as, on the capacity of the roundabout.

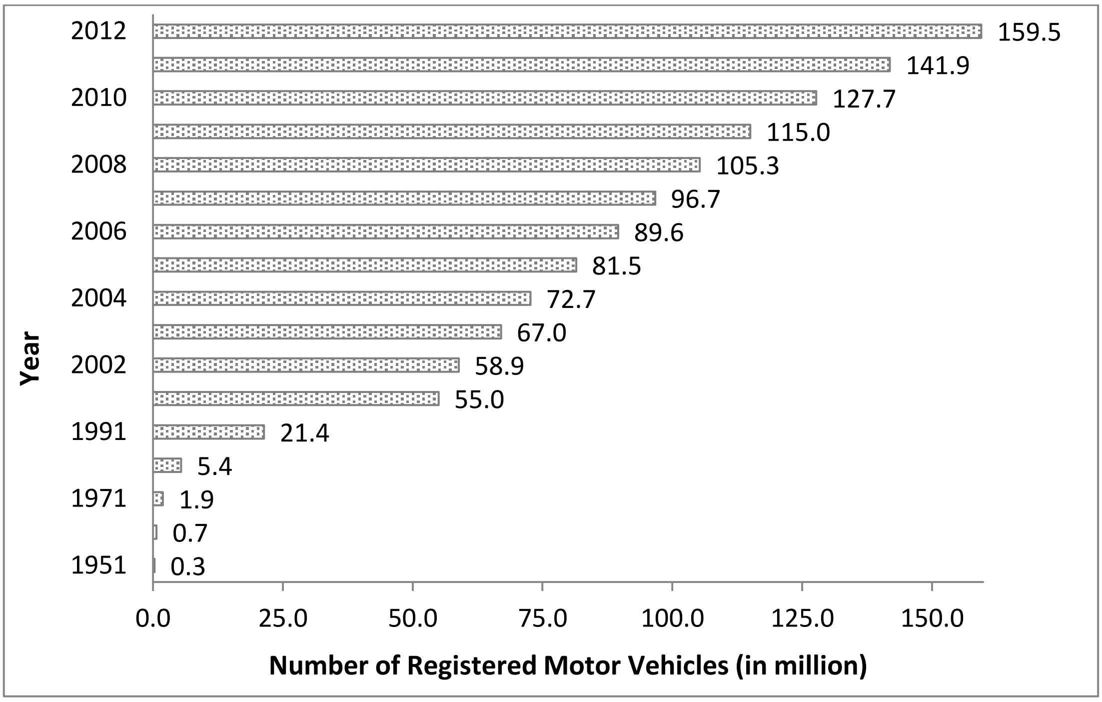


Figure 1.15 Total number of registered motor vehicles (in million): 1951-2012 (TRW 2013)

Figure 1.16 demonstrates the share of different categories of vehicles in the total registered motor vehicle population, as on 31st March 2012. Two-wheelers represented the largest share of 72.4%, followed by cars, jeeps and taxis (14%), other vehicles (8%), goods vehicles (5%) and buses (1%). The term 'other vehicles' comprise tractors, trailers, three wheelers, Light Motor Vehicles (LMVs) and other miscellaneous vehicles (TRW 2013).

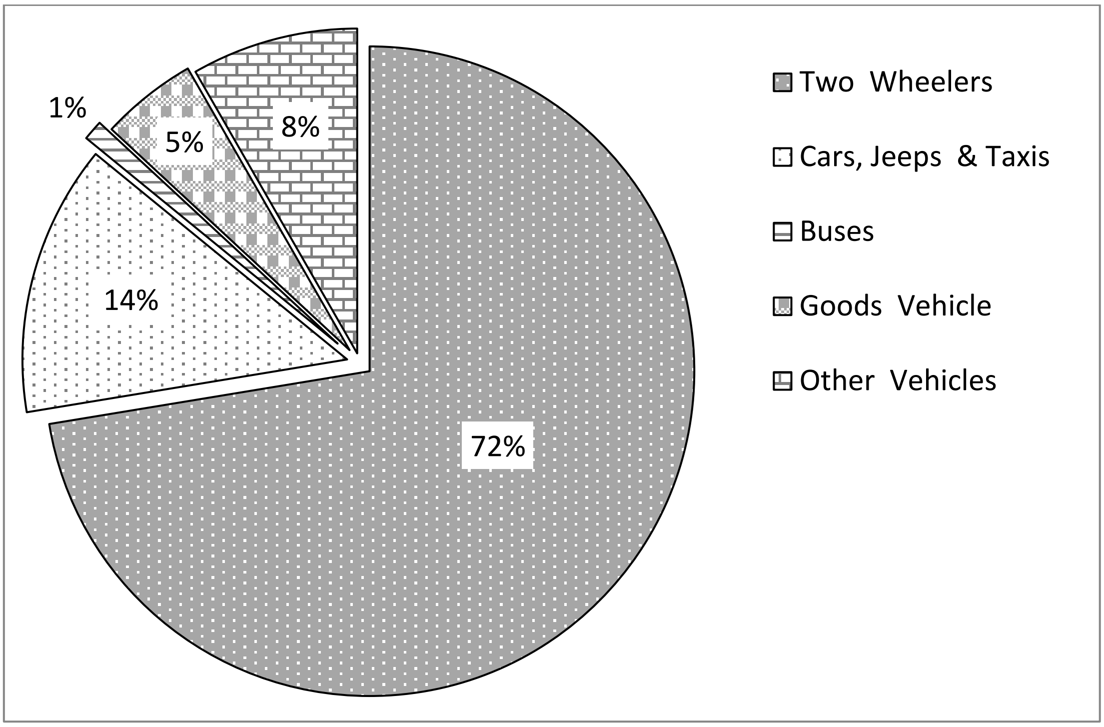


Figure 1.16 Composition of registered motor vehicles (as on 31st March, 2012)

Table 1.3 gives vehicular penetration in population in cities of developed and developing countries. It indicates that the total motor vehicle penetration in India is low compared with developed countries. The car penetration in developed countries is much higher than in the developing countries. In contrast, the penetration of two wheelers in developing countries is higher than the developed countries. Developed countries like Germany and USA have car penetration rates (car/1000 persons) higher by factors of about 34 and 42 to that of India. However, in India and few other developing countries the penetration level of two wheelers (two wheelers/1000 persons) is much higher compared to developed countries (TRW 2013). There are some developing countries which are following the trend of developed countries (like Mexico, Korea, etc.). One reason for such change may be the availability of good public transport system in some of these cities / countries.

Table 1.3 Vehicular penetration in developed & developing countries, 2010

Country	Per 1000 person			Percentage (%)	
	Cars	Two Wheelers	Total Motor Vehicles	Cars	Two Wheelers
Developed					
USA	627	27	797	78.7	3.4
UK	457	20	519	88.1	3.9
Japan	453	27	491	92.3	5.5
Germany	517	47	572	90.4	8.2
Australia	556	30	695	80.0	4.3
Japan	453	27	591	76.6	4.6
Spain	-	59	-	-	-
Italy	-	108	-	-	-
Developing					
Mexico	191	10	275	69.5	3.6
South Africa	112	7	165	67.9	4.2
Korea ,Republic	276	37	363	76.0	10.2
India**	15	96	132	11.4	72.7
Sri Lanka	-	127	-	-	-
Portugal*	-	51	-	-	-
Vietnam#	-	251	-	-	-

* Data relates to 2009; # Data relates to 2007; ** Data relates to 2012

The traffic composition as shown in Table 1.4 indicates that variety of vehicles plying on roads in most of the developing countries is much more as compared to developed countries. Mallikarjuna and Rao (2011) distinguished homogeneous and heterogeneous traffic conditions. The traffic composed of identical vehicles and following the lane discipline is termed as homogeneous. Though, the traffic comprising of motorized and non-motorized two-wheelers and three-wheelers along with several other vehicles with no-lane discipline is termed as heterogeneous.

Arasan and Krishnamurthy (2008) and Fazio et al. (1999) suggested that homogeneous traffic exists when the percentage of dominant travel mode is more than around 80 % of the total traffic mix. Traffic conditions are heterogeneous in all cities of the world, including those in the Europe and the US.

Table 1.4 Traffic composition on roads in developed and developing countries

City	Walking	Cycling	Public Transport	Private Motor Vehicle	Country
Indianapolis	0.02	0.01	0.02	**0.92**	US
San Antonio	0.02	0.00	0.03	**0.90**	US
Auckland	0.03	0.01	0.06	**0.89**	New Zealand
Dallas	0.02	0.00	0.04	**0.89**	US
Phoenix	0.02	0.01	0.03	**0.88**	US
Adelaide	0.03	0.01	0.10	**0.86**	Australia
Perth	0.03	0.01	0.10	**0.86**	Australia
San Diego	0.03	0.01	0.04	**0.85**	US
Bogota	0.15	0.02	**0.64**	0.19	Colombia
Warsaw	0.05	0.01	**0.60**	0.34	Poland
Mumbai	0.27	0.06	**0.52**	0.15	India
Tokyo	0.23	0.14	**0.51**	0.12	Japan
Delhi	0.21	0.12	**0.48**	0.19	India
Singapore	0.22	0.01	**0.44**	0.33	Singapore
Prague	0.23	0.01	**0.43**	0.33	Czech
Vienna	0.26	0.07	**0.39**	0.28	Austria
Madrid	0.36	0.00	**0.34**	0.30	Spain
Osaka	0.27	0.00	**0.34**	0.39	Japan
Barcelona	0.35	0.12	**0.33**	0.20	Spain
Shanghai	0.27	0.20	**0.33**	0.20	China
Paris	**0.61**	0.03	0.27	0.09	France

Madrid	**0.36**	0.00	0.34	0.30	Spain
Barcelona	**0.35**	0.12	0.33	0.20	Spain
Vilnius	**0.36**	0.00	0.26	0.38	Lithuania

Source: (Wikipedia 2016)

The degree of heterogeneity is different across developing and developed nations. Different types of vehicles are in use in India and other developing countries, in general, namely cars, heavy vehicles, motorized two-wheelers, motorized three wheelers, buses, non-motorized vehicles etc. These vehicles share the same road space without any physical or marked segregation, depending on the availability of the road space at a given instant of time. On similar lines the traffic stream on Indian roundabouts also comprises of variety of vehicles leading to its classification as heterogeneous. Figure 1.17 shows heterogeneous traffic condition as typically found on roundabouts in India.

Figure 1.17 Heterogeneous traffic on Indian roundabouts

There is a wide variation in the static and dynamic characteristics of different types of vehicles in developing countries. Consequently, one class of vehicles cannot be considered equal to other class of vehicle. The only way of accounting for this non-uniformity in traffic stream is to convert all vehicles into a common unit and the most accepted unit for this purpose is passenger car unit or PCU (Akçelik 1997). The concept of PCU was introduced in the 1965 Highway Capacity Manual (HCM 1965) and considerable research effort has been directed towards estimation of equivalency factors for vehicles plying on various roadway types since then.

Roundabouts provide a mechanism for merging and diverging of circulating and entering traffic flow streams. Priority of operation goes to circulating traffic flow stream and the entering traffic flow has to adjust or wait till minimum acceptable gaps in the circulating traffic flow stream are available. The capacity of the roundabouts had been estimated in the past as the capacity of the circulating (weaving) section. In such a case, the basis has been the width of the circulating roadway, the mix of the vehicle types, the length of the weaving section between adjacent entry and exit, the approach and exit widths, etc. The initial formula for the capacity estimation of a roundabout was given by Wardrop in late 1950s (Wardrop 1957). As defined before, it considered the width of the entry, length and width of the weaving section and proportion of the weaving traffic. In India, same approach was followed and documented in code IRC-65 (1976) 'Recommendation Practice For Traffic Rotaries'. Both of these basically estimated the capacity of the weaving section. Then in mid-1960's, the priority rule was implemented in Britain. The subsequent studies carried out on the validation of Wardrop's formula under new traffic control (priority) condition indicated that it was not valid and gave inaccurate results. Going by the research findings, Britain changed the estimation formula for the capacity of a roundabout (Kimber 1980). Later, the research carried by Pearce (1987) and Waddell (1997) too concluded that traffic in the weaving section does not affect the capacity of the roundabout. Such findings necessitated the shift in the estimation approach for the capacity of the roundabout. It was felt that under the given traffic flow conditions, the number of vehicles that can enter the roundabout in a given time period would depend upon the circulating traffic flow (volume) in the circulating roadway. As the traffic flow in a circulating roadway increases over the time period in a day, the traffic flow that can enter the roundabout circulating area from any approach leg would keep reducing. In other words, as the circulating traffic flow decreases, the entry traffic flow increases due to higher opportunities available to drivers for entering into the circulation area. This led to the new definition of roundabout capacity, named as 'entry capacity'. It is defined as the maximum number of vehicles that can enter the roundabout from an approach in a given time period corresponding to a circulating traffic volume. Intuitively, these two traffic flow entities would be moving in opposite directions, and the relation between the two may or may not be linear in nature. If the nature of relationship is linear then with no circulating traffic flow the entry capacity shall be maximum, whereas, with very-very heavy circulating traffic flow there shall not be any entry traffic flow. This shall result in queuing in an approach. Leaving acute traffic flow conditions, in most of the traffic flow conditions it has not been found true. There remains some flow from the approach which merges into the circulating traffic flow even under most constrained conditions.

Researchers have estimated the entry capacity of a roundabout based on three approaches, namely weaving based approach, gap-acceptance based approach and empirical approach or regression based approach. Gap-acceptance and empirical approaches describe the entry capacity as a function of the circulating traffic flow. The approach based on gap acceptance behavior of the driver inherently considers the decision making of the drivers of entering vehicles to merge within the available gaps between circulating vehicles under perceived and prevailing traffic conditions. The entry capacity of a roundabout based on gap-acceptance is considered as dependent upon critical gap and follow-up time value. The accuracy of capacity estimation is mainly dependent on the accuracy with which the critical gap is estimated. The estimation of critical gap of a vehicle type under mixed traffic situation is always a challenging task as compared to its estimation under homogeneous traffic conditions. Traffic in developed countries is homogeneous in nature, which comprises of vehicles with more or less uniform static and dynamic characteristics. The driving characteristics under peak traffic conditions become same. This makes many calculations much simpler because vehicles' size, speed and follow-up distances can be held constant. Unfortunately, this parameter cannot be obtained directly through field measurements as it varies with driver, time, intersections, movements and traffic conditions (Maurya et al. 2014). When vehicles of different sizes and operational capabilities move in a mixed mode, the lane concept and follow-the-leader concept becomes irrelevant. In all of the developing nations maximum share of the traffic is composed of small size vehicles like motorized two-wheelers. Under flow and space constraints, these vehicles try to find gaps to enter the circulating area, without following the lane discipline. They accept small gaps owing to their small size and higher maneuver flexibility. Large size vehicles force their entry in the circulating flow by accepting small gaps under high flow conditions. This causes high variation in accepted and rejected gaps by drivers of vehicles within same category or across the categories. In such a constrained and heterogeneous condition, the study of gap acceptance behavior becomes extremely complex.

The Highway Capacity Manual of US (HCM 2010) presented gap acceptance model of entry capacity for single-lane and two-lane roundabouts in U.S. This manual is extensively used in different parts of the world for estimating the capacity of a traffic facility. The approach considers lane discipline and the entry behavior of the drivers are assumed to be normal. This is not the case in Indian traffic flow condition. First of all, varieties of vehicle ply on a road in India or other developing countries. Under heavy traffic flows, single lane system converts to two-lane system and two-lane system converts to three-lane system. This happens due to formation of parallel lane in-between bigger vehicles by small size vehicles. In the case of merging operations such vehicles force their entry into the stream. The gap thus formed is, at times, utilized by bigger size vehicles also for forcing entry into the circulating traffic stream. Therefore, the entry capacity model proposed in HCM (2010) does not have direct transferability to the traffic flow conditions and driver behavior prevailing in developing countries like India. It is expected that the application of HCM models may result in unrealistic results under such conditions.

Empirical approach is usually based on the highest traffic flows observed during peak periods from an approach into a roundabout circulating area. A relationship is formed between these observed values and an influencing variable like circulating traffic flow. Such models can best reflect the local traffic flow condition and can be applied to other areas after applying corrections or adjustment factors.

1.3.1 Problem Statement

IRC guidelines for roundabouts (IRC-65 1976) were framed in 1976 for the design of rotaries in India. This guideline used static PCU values which are based on old studies prior to 1976. The static and dynamic characteristics of all vehicles in general and car in particular, have changed considerably during last three decades. Metkari et al. (2012) also reported that the PCU for vehicles in heterogeneous traffic conditions at rotaries have not been studied till date. As also discussed in previous sub-section, it arises the need to estimate the PCU values for different categories of vehicles at roundabouts in India under changed traffic conditions and vehicle operations over the years. Therefore, a method of estimating PCU values for different types of vehicles need to be developed. This may be based on traffic flow parameters and vehicle static characteristics. This will also help in revising the available guidelines for the PCU values at roundabouts in India or in developing countries.

Another aspect related to roundabout is its capacity. In India, the capacity of roundabouts is traditionally based on weaving theory. No comprehensive study is being carried out to develop a formula to estimate the entry capacity of the roundabout and thus to update and revise, a nearly 40 years old IRC code. There is a need to incorporate the new research in the estimation of the roundabout capacity based on the field study and develop statistical model for estimating roundabout entry capacity as a function of circulating traffic flow and roundabout geometric characteristics.

One approach may be based exclusively on flow characteristics like critical gap and follow-up time. The method enumerated by Highway Capacity Manual (HCM 2010) is purely based on these parameters. This document is extensively used in different parts of the world for estimating the capacity of a traffic facility. However, the direct transferability of the HCM (2010) entry capacity model to Indian traffic flow conditions is doubtful as the manual do not consider driver behavior in mixed traffic flow. It considers lane behavior and driver discipline. Therefore, the coefficients in HCM (2010) model need to be calibrated so that the modified HCM model suits the heterogeneous traffic conditions as prevailing in developing countries like India. For its use there arises the need to estimate the critical gap which is not computed directly from field data. Good numbers of estimation method are available in literature for critical gap. All of these methods use accepted / rejected gaps or lags in different forms and have some in-built assumption. Due to this there is no consensus on the use of these methods. It is proposed to develop a critical gap estimation method with rational assumptions.

The linear and non-linear regression analysis approach can be adopted to estimate entry capacity and to investigate the impact of the physical parameters of roundabouts on entry capacity under heterogeneous traffic flow condition. Models from developing countries can also be examined to see if any of those can be used directly in Indian traffic flow condition on the roundabouts.

1.4 OBJECTIVES OF THE RESEARCH WORK

The present research is taken up with the following objectives:

a) To estimate Passenger Car Unit (PCU) values for different types of vehicles typically found on a roundabout in developing countries and to propose a concept of Heterogeneity Equivalency Factor (H-factor) to convert a heterogeneous traffic stream into equivalent number of passenger cars.

b) To develop an estimation procedure for critical gap and to determine critical gaps and follow-up times for different types of vehicles.

c) To calibrate the HCM (2010) entry capacity equations for its adaptation to the heterogeneous traffic, and develop a model of entry capacity using traffic flow and geometric data.

d) To perform the sensitivity analysis on the entry capacity models with respect to the influencing variables and to compare regression model with the existing global entry capacity models.

e) To suggest the modifications, based on above analysis, in codal provisions for the roundabouts.

1.5 SCOPE OF THE WORK

The present research work is mainly focused on entry capacity analysis of roundabouts under mixed traffic conditions. The scope of the work is restricted on roundabouts with heterogeneous traffic flow. The total work is accomplished in ten parts as given below:

a) Literature review of the studies on PCU factor, gap acceptance parameters and various capacity models for roundabouts.

b) Field data collection at selected roundabouts using video camera and extraction of required data.

c) Estimation of lagging headways to develop PCU values for different types of vehicles for ready use by traffic engineers.

d) Estimation of heterogeneity equivalency factor to convert a heterogeneous traffic stream into equivalent number of passenger cars.

e) Extraction of accepted and rejected gaps to estimate the critical gaps for different types of vehicles.

f) Comparing the existing methods for estimating the critical gap and proposing a new method applicable to the complex driver behavior in Indian context.

g) Ascertaining the relationship between entry capacity and circulating traffic flow.

h) Calibrating HCM (2010) model, using gap acceptance parameters under heterogeneous traffic condition.

i) Developing a regression model between the entry capacity, circulating traffic flow and physical parameters of a roundabout.

j) Comparing the developed entry capacity model with the existing developing countries models (Jordan, Malaysian and Indian).

k) Performing sensitivity analysis of the developed entry capacity model with respect to the range of flow and physical variables of a roundabout.

1.6 ORGANIZATION OF THE THESIS

The thesis report is organized in the following manner.

Chapter 1 - The first chapter of the thesis gives the brief background of roundabouts, advantages and disadvantages, and geometric design elements of a roundabout. Characteristics of mixed traffic, need for the study, and objectives of the research work are also included in this Chapter.

Chapter 2 - This Chapter covers review of various studies done in India and abroad on the topic. This includes aspect like passenger car unit, gap acceptance and capacity of a roundabout.

Chapter 3 - This chapter deals with site selection, field data collection and extraction of required data from video film. It also discusses the methodology of research approach and includes preliminary analysis.

Chapter 4 - This chapter deals with the estimation of PCU values on roundabouts. A method to estimate PCU values is discussed and the new PCU values are compared with the existing PCU values in different countries. Further, a concept of Heterogeneity Equivalency Factor (H-factor) is introduced which gives a factor to convert heterogeneous traffic into homogeneous traffic.

Chapter 5 - This chapter deals with the extraction of accepted and rejected gap values and presents a simple procedure for the estimation of critical gaps. Prominent methods available in literature to estimate critical gap are also compared with the proposed method. Critical gap values as suggested in different countries are also compared. Follow-up time from an approach are also discussed and compared.

Chapter 6 - This chapter presents the relationship between entry capacity and circulating traffic flow. The influence of roundabout geometrics on entry flows is examined. HCM (2010) model has been modified for Indian traffic flow condition on roundabouts. This chapter also presents the development of a regression model to estimate entry capacity of an approach on a roundabout for Indian traffic flow conditions. The proposed model is compared with the existing models available in the literature to examine the suitability and applicability.

Chapter 7 - The major conclusions drawn from the study are given in this chapter. The significant contributions of the research work, limitations of the work and scope for the future work are also highlighted.

CHAPTER 2:
LITERATURE REVIEW

2.0 GENERAL

Roundabouts have been widely accepted as replacements or alternatives to conventional intersections in Europe and Australia. They have also been accepted as a traffic control measure at an intersection around the world. They control the maneuver of vehicles in two streams i.e. circulating and entering flows with respect to their merging, diverging and parallel movements. With the new concept, entering traffic has to give way to circulating traffic and can enter only when offered gaps are appropriate to accept. Many studies found that roundabouts have better performance in terms of capacity and delay than any other intersection. Flannery et al. (1998) studied five single-lane roundabouts located in Florida and Maryland, before and after their installations, which were stop-controlled prior to being converted to roundabout. The benefits of a roundabout in reducing delay were clearly demonstrated at the Ft. Walton Beach site, in which control delay was reduced on an average from 163.52 to only 3.36 s/veh. The large delay in the before period was primarily due to a significant left-turning movement from the minor approach.

These devices have helped in reducing the conflict points in an intersection area, as well as, number of crashes. The accident analysis performed on the retrofitted roundabouts located in the United States yielded encouraging results that should lend confidence in the choice of roundabouts as a measure to control traffic. The main benefit of roundabouts is their better safety performance when compared with other types of un-signalized intersections (Flannery and Datta 1996b). Persaud et al. (2001) conducted a before-after study of roundabout installation at 23 intersections (19 stop-controlled and 4 signal-controlled intersections). These intersections were located in seven states of the United States. They found 40 percent reductions for all crash severities, 80 percent reductions for all injury crashes and 90 percent reduction for fatal and incapacitating injury crashes. Crashes involving pedestrians and bicyclists were also reduced after the conversion into roundabouts. But the sample size was too small to give conclusive evidence concerning these groups. Sisiopiku and Oh (2001) compared the performance of roundabouts with those of yield-control, all-way stop controlled (AWSC), two-way stop controlled (TWSC), and signal controlled type intersections using SIDRA software. Roundabouts showed a better performance in terms of capacity and delay than any other intersection type for the cases with two-lane approaches and heavy traffic volumes or heavy left-turn volumes. Mandavilli et al. (2008) studied the traffic scenario before and after installation of six roundabouts at stop-controlled intersection. They reported a significant reduction in average delay time, queuing length, and stopping time. Vlahos et al. (2008) compared the performance of AWSC with that of signal controlled and roundabout intersections using SIDRA software. The results of the comparative analysis were in agreement with previous researchers' findings that roundabouts always offer better performance than AWSC intersections under similar traffic conditions. An average of 190% increase in capacity was recorded on roundabouts, which replaced AWSC intersections, with delay going down by 49% and queue length reducing by 41%. Fajimi and Hassan (2011) compared vehicular delay on three types of intersections under Canadian driving conditions. It was found that the roundabout produced less delay compared to AWSC and signal controlled intersections. They opined that the reduced delays at the roundabout compared to the AWSC and signalized intersections should reduce frustration and aggressiveness of the drivers for driving safe and sound. Also, the reduced delay and less opportunity for forced stop at the roundabouts would cause them to be more environmental friendly than the AWSC and signalized intersections.

In the light of recent research works, it has been observed that not significant work has been published in the context of developing countries. Considering the example of India, as already discussed in the previous chapter, the design of a roundabout / rotary is based on the research which is already around 40 years old. The concept of analysis of roundabouts have seen a drastic change in approach from weaving section based to gap acceptance based, as well as, to priority given to vehicle in circulating flow rather than in entry flow. Another point is the big leap taken in the automobile technology during these years. The driving has become easier, comfortable but agile. Therefore, traffic on roundabouts need to be re-looked with respect to its changing operational characteristics. Keeping these under consideration, the literature related to traffic heterogeneity, gap acceptance behavior of drivers (vehicle), and the capacity of a roundabout has been studied and discussed in the following sub-sections.

2.1 STUDIES ON TRAFFIC HETEROGENEITY

Traffic heterogeneity has already been discussed in chapter-1, wherein, researchers have clearly mentioned that if no vehicle dominates the composition with share above 80%, the traffic is said to be heterogeneous. Again, it is shown that traffic, especially in developing countries, is heterogeneous in nature. The wide variety of vehicles in traffic stream causes problems with respect to the design of a facility and standardization of the guidelines. Therefore, in most of the cases the heterogeneous traffic is changed to homogeneous traffic (with respect to a specific design vehicle), before dealing with the analysis of traffic flows. This is discussed in the following successive paragraph.

The analysis of a non-uniform traffic stream of vehicles is simplified if the relative effect of each vehicle type can be expressed in terms of some common units. The Highway Capacity Manual (HCM 1965) introduced the concept of passenger car unit (PCU) or passenger car equivalent (PCE) to express volume or capacity in terms of passenger cars per hour per lane. As per HCM (1965), the PCU is defined as “The number of passenger cars displaced in the traffic flow by a truck or a bus, under the prevailing roadway and traffic conditions”. The HCM (2000) defined PCU as “The number of passenger cars displaced by a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions.” HCM (2010) further modified the definition of PCU as “The number of passenger cars that will result in the same operational conditions as a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions.” These definitions clearly indicate towards the change in concept of dealing with heterogeneity. It has changed from nearly static condition to purely dynamic condition related to vehicles in traffic flow. It also indicates that traffic control measures do impact such analysis. In the present context roundabout is one such control measure.

Determination of PCU has always been an area of research interest in almost all countries. Various approaches have been used to quantify the PCU of a vehicle category by various researchers in the past. The bases used for estimation process are flow rate, density, speed, headway, delay and queue discharge flow. These are briefly discussed here.

2.1.1 Flow Rate and Density

Huber (1982) proposed a method for the estimation of PCU using equal density to relate mixed traffic flow and base traffic flow rate. Huber's basic equation is formulated as,

$$E_T = \frac{1}{P_T}\left(\frac{q_B}{q_M} - 1\right) + 1 \tag{2.1}$$

Where,

P_T = proportion of trucks in the mixed traffic flow

q_B = base traffic flow, only passenger cars (veh/h)

q_M = mixed traffic flow (veh/h)

E_T = passenger equivalent of trucks

Sumner et al. (1984) further developed Huber's model by including more than one truck type in the traffic stream.

$$E_T = \frac{1}{\Delta P}\left(\frac{q_B}{q_S} - \frac{q_B}{q_M}\right) + 1 \tag{2.2}$$

Where,

q_B = base traffic flow, only passenger cars (veh/h)

q_M = mixed flow rate (veh/h)

q_S = additional subject flow rate (veh/h)

ΔP = proportion of subject vehicles

E_T = passenger equivalent of trucks

Demarchi and Setti (2003) suggested the PCU's formula to eliminate the possible error for mixed heavy vehicles in the traffic stream, including interaction between multiple truck types:

$$E_T = \frac{1}{\sum_i^n P_i}\left(\frac{q_B}{q_M} - 1\right) + 1 \tag{2.3}$$

Where,

P_i= proportion of trucks of type *i* out of all trucks *n* in the mixed traffic flow

q_B = base traffic flow, only passenger cars (veh/h)

q_M = mixed flow rate (veh/h)

E_T = passenger equivalent of trucks

Tiwari et al. (2007) modified the density method so that more accurate passenger car unit (PCU) values can be derived for accurate capacity, safety, and operational analysis of highways carrying non-homogeneous traffic. This is given by equation (2.4).

$$PCU_i = \frac{k_{pc}/W_{pc}}{\left(q_i/u_i\right)/W_i} \tag{2.4}$$

Where,

PCU_i = PCU for vehicle of class i

k_{pc} = concentration of passenger cars (veh/km)

q_i = traffic flow of vehicles of class i (veh/h)

u_i = space mean speed of vehicles of class i (km/h)

W_{pc} = width of passenger car (m)

W_i = width of vehicles of class i (m)

2.1.2 Speed

The HCM (1994) suggests that the PCU for a vehicle can be determined directly by obtaining detailed information on speed. Chandra et al. (1995) proposed a method for the estimation of PCU value for different vehicles under mixed traffic situation. The basic concept behind the method was that the PCU value is directly proportional to the speed ratio and inversely proportional to the space occupancy ratio with respect to the standard design vehicle i.e. car. The mathematical form of the proposed model is given by Equation (2.5).

$$PCU_i = \frac{V_c / V_i}{A_c / A_i} \tag{2.5}$$

Where,

V_c = speed of the passenger car (km/h)

V_i = speed of the i^{th} vehicle (km/h)

A_c = rectangular plan area of the passenger car (m^2)

A_i = rectangular plan area of the i^{th} vehicle (m^2)

Elefteriadou et al. (1997) developed a methodology for deriving PCU values on freeway, two-lane highways and arterials in US. Speed-flow curves were generated for ‘cars only’ traffic stream and for the typical vehicle mix found on the road. The subject vehicle type and flow level are then selected, and traffic operations are simulated, after adding a certain volume of subject vehicle and removing the same volume of passenger cars. The PCU values are then determined by comparing the points on these curves with same average speed.

Basu et al. (2006) modeled passenger car equivalency at an urban midblock in India using stream speed. The PCU values were found increasing with an increase in the traffic volume. The effect of traffic volume on PCU was predominant for heavy vehicles. The PCU value of heavy vehicle increased with an increase in the compositional share of heavy vehicle in the traffic stream, whereas, the PCU value of two wheelers remained unchanged irrespective of their compositional share in the traffic stream.

2.1.3 Headways

Greenshields et al. (1947) proposed a method for the estimation of PCU using headway-based approach. The method is based on the ratio of the average time headway for the vehicles of interest to the average time headway for passenger ears. This is given by equation (2.6)

$$PCU_i = \frac{h_i}{h_c} \tag{2.6}$$

Where,

PCU_i = PCU for vehicle of class i

h_i = average headway of vehicles of class i (seconds)

h_c= average headway of passenger car (seconds)

Werner and Morrall (1976) suggested that the headway method is best suited to determine PCUs when the roadway is sufficiently congested on level terrain and pioneered an equation for determining PCUs. This is given by equation (2.7).

$$E_T = \frac{\left(\frac{H_M}{H_B} - P_c\right)}{P_T} \tag{2.7}$$

Where,

H_M = average headway for all vehicles (seconds)

H_B= average headway for passenger car (seconds)

P_T = truck proportion

P_C = passenger car proportion

E_T = truck PCU

Cunagin and Chang (1982) expressed PCU based on spatial headway. This method defines the PCU as the ratio of the mean lagging headway of a subject vehicle and the mean lagging headway of the basic passenger car and is formulated as:

$$PCU_i = \frac{H_i}{H_c} \tag{2.8}$$

Where,

PCU_i = PCU of vehicle type i

H_i = mean lagging headway of vehicle type i (seconds)

H_c = mean lagging headway of passenger car (seconds)

This approach was extended by Krammes and Crowley (1986) by introducing proportion of specific vehicle type in the spatial headway method. They suggested that PCU should be calculated as,

$$E_T = \frac{(1-P_T)*H_{TP} + P_T*H_{TT}}{H_P} \tag{2.9}$$

Where,

P_T = proportion of trucks,

H_{TP} = lagging headway of trucks following passenger cars (seconds)

H_{TT} = lagging headway of trucks following trucks (seconds)

H_P = lagging headway of cars following either vehicle type (seconds)

Molina (1987) proposed a modified headway ratio method that considered the increase in headway for vehicles queued behind the heavy vehicle. The PCU was calculated as:

$$PCU_i = \frac{(h_i + \Delta H)}{h_c} \tag{2.10}$$

Where,

PCU = PCU for a vehicle of type i

h_i = headway of vehicle of type i (seconds)

h_c = saturation flow headway of passenger car (seconds)

ΔH = total increased headway of the queue caused by the truck (seconds)

Bhattacharya and Mandal (1980) developed a generalized model to estimate PCU based on headway at intersection in Calcutta (Now Kolkata) in India. The generalized model was formed on the basis of an idealized condition in which the green interval for an approach lane was assumed to be fully loaded with passenger cars crossing the intersection while other green intervals of same duration for the same approach lane were loaded fully with other homogeneous types of vehicle like buses and trucks etc. The model can therefore, be formed considering PCU equivalent of certain type of vehicle as the ratio of number of cars that can pass in green period to the number of vehicles of a particular type that can pass in the same interval. Rongviriyapanich and Suppattrakul (2005) analyzed the effect of motorcycle traffic on urban mid-block section and signalized intersections in Bangkok. They found that the PCU of motorized two-wheeler declines with an increase in their share in the traffic flow.

2.1.4 Delay

Craus et al. (1980) proposed an equation based on delay to measure PCU of a truck as the ratio of delay caused by one truck to the delay caused by one passenger car.

$$E = \frac{d_{kt}}{d_{kp}} \tag{2.11}$$

Where,

E = truck PCU

d_{kt} = average delay time caused by one truck (seconds)

d_{kp} = average delay time caused by one passenger car (seconds)

Benekohal and Zhao (2000) suggested a method for the estimation of PCU at intersections based on delays. They suggested that the delay-based PCU value is the ratio of the additional delay caused by a heavy vehicle of type i, to the average vehicle delay of passenger-car-only stream plus one. Chitturi and Benekohal (2008) developed a method to determine PCU values for heavy vehicles for intersections which was an extension of delay-based methodology. They found that the PCU values decreased with increasing heavy vehicle percentage and increased with increasing traffic flow.

2.1.5 Simulation

Computer simulation technique has become a powerful tool to study the complex system when analytical or empirical approaches cannot adequately and accurately define the response pattern. The principal behind this technique is to observe certain characteristics of traffic such as free speed, distribution, overtaking logic, crossing logic, etc., and to build a computer model synthesizing the behavior of vehicles. The model can be used to predict the travel time and delay of individual vehicles for any volume and mix of traffic. Using the results of simulation runs, the effect of mixed traffic can be evaluated for any road and traffic conditions, and PCU values can be calculated easily.

St John (1976) developed a microscopic simulation model to derive nonlinear truck factor for two-lane highways including all important factors which can affect the flows. Results from the simulation model indicated that the truck factor should be nonlinear. Keller and Saklas (1984) developed a procedure to measure PCU using TRANSYT simulation software based on the delay caused by different categories of vehicles. Ramanayya (1988) developed a simulation model MORTAB (MOdel for depicting Road TrAffic Behavior) to study traffic behavior under mixed traffic conditions and estimated the equivalent design vehicle units for different categories of vehicles at different LOS. The estimated PCU factor for different vehicle categories showed a decreasing trend as the LOS deteriorated from A to C.

Bang et al. (1995) developed a simulation method for determining PCUs for township roads in Indonesia using the VTI (Vehicle Track Interaction) microscopic simulation model. The empirical data were used for calibration and validation process of the simulation model and for analyzing the environmental conditions on speed and capacity. The simulation model was used for determining speed-based light-vehicle units (instead of passenger-car units) and speed-flow relationships for flat, rolling, and hilly terrain. Webster and Elefteriadou (1999) estimated the PCUs for trucks on basic freeway sections using FRESIM simulation model. It was found that the PCU tends to increase with traffic flow, free flow speed, and grade/length of grade; and tends to decrease with an increase in truck percentage. Arasan and Koshy (2004) developed model for capacity and service-volume standards for mixed traffic flow condition on urban roads. The developed model was used to determine PCU values for different categories of vehicles for the traffic flow at LOS C.

Rakha et al. (2007) estimated PCU for trucks at different grades for freeway sections. PCUs are developed for broader range of 'vehicle-weight to power' ratio in the INTEGRATION software. They developed PCUs for trucks at different LOS and 2 to 6 percent grades. Aggarwal (2008) identified a number of factors affecting PCU value and developed a fuzzy MATLAB based model for the estimation of PCU. Arasan and Krishnamurthy (2008) used simulation approach to develop PCU factors for different categories of vehicles under heterogeneous traffic conditions prevailing on Indian urban roads and studied the effect of traffic volume and road width on PCU values. It was found that PCU values of all categories of vehicles followed an increasing trend at low volume levels and showed decreasing trend at high volume levels. They also reported that PCU of a vehicle type increased with increasing in road width. Dey et al.

(2008) developed the PCU factors for two lane roads in India, using simulation model. It was found that PCU of all vehicle types decreased with increasing in its own proportion in traffic stream and volume-capacity ratio, except for heavy vehicle where it increased with the increase in their own proportion. They attributed it to the large size of heavy vehicles and their poor operational efficiency.

Carrignon and Buchanan (2009) estimated PCU values for two-wheelers in heterogeneous traffic in the city of London. Microscopic traffic simulation VISSIM was used to generate the heterogeneous traffic conditions. Arasan and Arkatkar (2010) used micro-simulation technique to describe the effect of traffic volume and road width on PCU of vehicles under heterogeneous traffic conditions in India. They observed that in the case of vehicles that are larger than passenger cars, their PCU value, at low traffic volume levels, decreased with an increase in the traffic volume, while, at high traffic volume levels, it increased with the increase in traffic volume. However, this trend of variation in PCU with traffic volume was reversed for vehicles smaller than passenger cars.

Shukla and Chandra (2011) developed simulation model to analyze mixed traffic flow on 4-lane divided highways in India. Effect of traffic volume and composition showed that PCU for a vehicle type decreased with the increase in V/C ratio and its proportion in the traffic stream. However, in the case of heavy vehicles, the PCU values showed an increasing trend with the increase in its proportion. Bains et al. (2012) studied the effect of vehicle composition, mainly proportion of heavy vehicles and light commercial vehicles, on PCU values at different volume levels. The studied was carried out on expressways in India. PCU values were evaluated using micro-simulation model, VISSIM. It was found that PCU deceased with the increase in V/C ratio or proportion of the vehicle irrespective of vehicle category.

Praveen and Arasan (2013) derived PCU values for vehicles plying on urban roads in India. The HETEROSIM simulation software is used to determine the relative impedance caused by a vehicle type in the stream of all cars. The trend in PCU variation for all types of vehicle categories is found to increase with an increase in traffic volume at low traffic volume conditions, whereas under higher traffic volume conditions, the PCU value decreased with the increase in traffic volume, for any given composition. Mehar et al. (2014) determined the PCU at different level of service for capacity analysis of multilane highways in India. Microscopic simulation software VISSIM is used to generate the traffic flow and speed data. It was found that PCU of a vehicle type decreased with the increase in V/C ratio. This was similar to the results of Dey et al. (2008).

2.1.6 Miscellaneous

Indian Roads Congress code (IRC-65 1976) recommended PCU values for various categories of vehicles for the analysis of roundabouts. This document was based on old studies prior to 1976. Central Road Research Institute (CRRI 1982) made a comprehensive study on PCU estimation. The study suggested that PCU values were dependent on traffic volume and composition of traffic mix and hence there cannot be a universal PCU value for a vehicle class. Justo and Tuladhar (1984) studied the effect of various parameters on the PCU values of the vehicles. These factors were effective width of vehicle, speed of vehicle and headway. The proposed PCU model is given in Equation (2.12).

$$PCU_i = \left(\frac{W_i}{W_c}\right) * \left(\frac{U_c}{U_i}\right) * \left(\frac{t_i}{t_c}\right) \tag{2.12}$$

Where,

W_i = effective width of vehicle class i (m)

W_c = effective width of passenger car (m)

U_i = mean speed of vehicle (kmph)

U_c = mean speed of passenger car (kmph)

t_i = mean time headway of vehicle class i (s)

t_c = mean time headway of passenger car (s)

Kimber et al. (1985) found that the regression analysis of synchronous vehicle counts, Webster’s method, and headway ratio method gave appropriate PCU values whereas asynchronous regression analysis of vehicle count method gave lower results so long as there was variability in headways of vehicles of a given class. Hutchinson (1990) estimated the PCU values for different categories of vehicles based on average time required by different categories of vehicles to complete various types of intersection movements. Fan (1990) derived PCU values for different types of vehicle plying on a Singapore expressway. The study revealed that the PCU values recommended by the highway capacity manuals of US, UK etc. may not be suitable for capacity analysis in Asian countries. Tanaboriboon and Aryal (1990) deduced the PCUs for medium sized vehicles like cars as 1.0 and for large sized vehicle such as buses, trucks, tractor tailors as 1.5 on highways in Thailand.

Harwood et al. (1999) quoted PCU factor for trucks, buses as 2.0 and 1.8 respectively for level terrain at level of service ‘A’. These PCU factors increased by 100% and 67% for trucks and buses respectively for rolling terrain and again by 75% and 90% for mountainous terrain. Chandra and Sikdar (2000) developed a mathematical model for speed parameter and generated PCU values for different road widths. They showed that the PCU for a vehicle type depends on all parameters of roadway and traffic affecting behavior of vehicle in the traffic stream. Al-Kaisy et al. (2002, 2005) developed a method for capacity-based estimation of PCUs, as well as, did simulation experiment for freeways under congested flow conditions. They determined PCUs by minimizing the coefficient of variation of the queue discharge flow rate in order to show that the impact of heavy vehicles on traffic flow was greater during congestion than under saturated conditions. Chandra and Kumar (2003) studied the effect of road width on PCU of vehicles on two-lane highways and found that PCU values increased with increase in width of the road. Golias (2003) developed the Taxi Equivalence Factor (TEF) in place of PCU based on capacity and delay. Capacity based TEF values were low, while delay based TEF assumed higher values and were as high as 2.0 under specific conditions. Li et al. (2006) proposed passenger car units for different categories of vehicle using vehicle moving space (VMS) as a measure to derive PCUs for roadway and traffic conditions of China. They suggested that PCU values increase with number of lanes and level of service from A to E. Mallikarjuna and Rao (2006a) used area occupancy, as equivalence criteria to estimate the PCU values for Indian conditions. The results were similar to those of Chandra and Sikdar (2000) in Indian traffic flow conditions and of Rongviriyapanich and Suppattrakul (2005) in Bangkok.

Basu and Maitra (2006) developed a model using NN (Neural Network) by expressing traffic volume in terms of equivalent area of old technology cars on four lane divided road. Total equivalent car volume can be estimated using equation (2.13).

$$V_{Eq} = N_{HV}\frac{23}{7.82} + N_{OC} + N_{NC}\frac{5.47}{7.82} + N_{TW}\frac{1.44}{7.82} \tag{2.13}$$

Where,

V_{Eq} = total equivalent car volume

N_{HV} = number of heavy vehicles per hour

N_{OC} = number of old technology cars per hour

N_{NC} = number of new technology cars per hour

N_{TW} = number of two wheelers per hour

Dey et al. (2007) found that the PCU for a vehicle decreased with the increase in volume-capacity ratio for two-lane roads because speed differential decreases as traffic volume increases. Geistefeldt (2009) proposed the stochastic concept of determining PCU of heavy vehicles on freeways. Capacity distribution function was developed in passenger car units and then PCUs were derived such that the variance in capacity distribution function becomes minimal. PCU values suggested in the study tend to decrease with increase in road width. Cao and Sano (2012) proposed a method for estimation of Motorcycle Equivalent Unit (MEU) in place of PCU, for urban roads in Hanoi, Vietnam because of the predominance of motorcycles in the traffic stream rather than passenger car. The MEU values of cars, buses, minibuses, and bicycles were found to be 3.4, 10.5, 8.3 and 1.4, respectively.

Joshi and Vagadia (2013) derive the Dynamic Car Unit (DCU) and Dynamic Two-wheeler Unit (DTU) for urban roads in India. Data were collected in seven metropolitan cities and modified homogenization coefficients approach was used to find the variation in DCU and DTU values. DCU for 2W, 3W, LCV, Mini Bus and Standard Bus decreased with an increase in flow rate (DCU/h). DTU for 2W, 3W, LCV, Mini Bus and Standard Bus increased with the increase in flow rate (DTU/h). DTU for car increased beyond flow rate of 6500 DTU/h with the increase in flow rate (DTU/h). This happened because of ability of two wheelers to maintain relatively high speed at high traffic volume due to their less width and better maneuverability. Paul and Sarkar (2013) determined dynamic PCU for different types of vehicles on urban roads using the model suggested by Chandra et al. (1995). They found that PCU of two-wheelers increased with the increase in proportion of heavy vehicles and decreased with the increase in proportion of NMT, whereas PCU of bus decreased with the increase in heavy vehicle percentage and increased with NMT percentage.

The review of literature on PCU presented above is mainly related to the single-lane roads, two-lane roads, multi-lane roads or signalized intersections. Very few studies had been done in the area of un-signalized intersection or roundabouts. These studies reveals that different researchers have adopted different methods for the estimation of PCU values and a wide variation exists in PCU values reported in different studies. PCU values derived by different researchers vary in magnitude also. These are given in Table 2.1. It is due to varying behavior of the drivers and traffic conditions in different countries. The Indian manual for roundabouts, Indian Roads Congress (IRC-65 1976) is being used while this document is based on old studies prior to 1976. Since then old technology in vehicles has changed. Therefore, it is necessary to re-look the PCU values based on field studies.

Table 2.1 PCU values derived by different researchers

Researcher	**2W**	**3W**	**BC**	**HV**	**Method**	**Sections**
IRC-65 (1976)	0.75	1	-	2.8	***	Roundabouts
Tanaboriboon and Aryal (1990)	-	-	-	1.45-1.53	Headway	Four and six-lane roads
IHCM (1993)	0.50	-	-	1.3-2.0	***	Roundabouts
Kumarage (1996)	0.70	0.9	1.2	2.2	***	Roundabouts
Al-Masaeid and Faddah (1997)	0.50	-	-	2.0	***	Roundabouts
Chandra and Sikdar (2000)	0.25-0.30	1.24-1.75	-	3.66-5.64	Speed and Area of Vehicle	Urban roads
Al-Kaisy et al. (2005)	-	-	-	2.4-13.4	Queue Discharge Flow, INTEGRATION Microscopic Simulation	Freeways and multilane highways
Rongviriyapanich and Suppattrakul (2005)	0.2-1.0	-	-	-	Headway	Mid-block

Basu et al. (2006)	0.04-0.16	-	-	0.75-2.2	Stream Speed	Four lane divided roads
Rakha et al. (2007)	-	-	-	1.5-10.5	Flow-Density and INTEGRATION Microscopic Simulation	Freeways
Dey et al. (2008)	0.22-0.26	0.8-1.7	-	4.6-5.4	Simulation Program, Speed and Area of Vehicle	Two-lane roads
Arasan and Arkatkar (2010)	0.34-0.89	0.50-0.99	-	1.70-2.90	HETEROSIM Micro-Simulation	Four-lane divided roads
(HCM (2010)	-	-	-	2.0	***	Roundabouts
Bains et al. (2012)	-	-	-	3.5-4.4	VISSIM Micro-Simulation	Expressways
Pakshir et al. (2012)	0.75	-	-	2.8	***	Roundabouts
Dhamaniya and Chandra (2013)	0.21-0.23	0.98-1.03	1.47-1.70	5.81-7.14	Speed and Area of Vehicle	Four-lane and Six-lane roads
Praveen and Arasan (2013)	0.2-1.2	1.2-3.5	-	1.4-6.5	HETEROSIM Micro-Simulation Technique	Four-lane divided roads
Lee (2014)	-	-	-	3.5-6.0	Minimization of variation in the entry capacity	Roundabouts
Mehar et al. (2014)	0.19-0.26	0.89-1.06	1.38-1.58	3.78-4.30	VISSIM Microscopic Simulation	Multi-lane roads

***Not Given

2.2 STUDIES ON GAP ACCEPTANCE PARAMETERS

Ashworth and Green (1966) were probably the initial researchers who measured gap from the rear of one vehicle to the front of the following vehicle and reported it. Adebisi (1982) defined gap as the major stream headway wholly available to a waiting vehicle from the minor road. Polus (1983) defined it as the time interval between two successive vehicles in the major road stream.

Critical gap is an important parameter in gap acceptance behavior. Vasconcelos et al. (2012a) reported that "a 0.5 s difference in the critical gap can result in capacity difference of up to 15%". A small variation in the critical gap would result in significant variation in the entry capacity (Velan and Van Aerde 1996). Raff and Hart (1950) defined critical gap as the size of the gap whose number of accepted gaps shorter than it is equal to the number of rejected gaps longer than it. HCM (2000) defined critical gap as the minimum time interval in the circulating flow that allows intersection entry for one entry vehicle. A particular driver would reject any gap less than the critical gap and would accept gaps greater than or equal to the critical gap. All gaps less than the critical gap would be rejected and all gaps greater than or equal to the critical gap would be accepted. Another definition of the critical gap is given as the gap that has an equal probability of being accepted or rejected (Polus et al. 2005).

Several methods have been developed in last few decades to estimate critical gap for a vehicle and movement type at two-way stop controlled (TWSC) intersections or roundabouts. Prominent method among them are Ashworth's, Harders', Modified Raff's and Maximum likelihood (Ashworth 1970; Fitzpatrick 1991; Harders 1968; Raff and Hart 1950; Troutbeck 1992). Recently, Wu (2012) suggested a method to estimate critical gap based on the macroscopic probability equilibrium of the accepted and rejected gaps. The above discussed methods are now presented in the following sub-sections.

2.2.1 Harders Method

Harders (1968) developed a model for the estimation of critical gap. The time scale was divided into intervals of constant duration say Δt. The center of each interval was denoted by t_i. For each vehicle on the entry approach, all circulating vehicle gaps that were presented to the entering vehicle were observed. From these observations, the probability of acceptance (p_i) in interval 'i' and critical gap (t_c) were estimated using equation (2.14) and (2.15) respectively.

$$p_i = \frac{a_i}{n_i} \tag{2.14}$$

$$\text{Critical gap, } t_c = \sum_{i=1}^{n} t_i * (p_i) \tag{2.15}$$

Where,

p_i = probability of acceptance in interval i

n_i = number of all gaps of size i that were provided to the entering vehicle

a_i = number of accepted gaps of size i

t_i = time at the centre of interval i

2.2.2 Ashworth Method

Ashworth (1970) found that the average critical gap (t_c) can be estimated from mean and standard deviation of the accepted gaps using equation (2.16).

$$t_c = \mu_a - p * \sigma_a^2 \tag{2.16}$$

Where,

p = circulating traffic (vps)

μ_a = mean of the accepted gaps

σ_a = standard deviation of the accepted gaps

2.2.3 Modified Raff Method

It is one of the earliest methods for estimating the critical gap and easiest to use. This method estimates the mean critical gap by drawing the cumulative distribution function of accepted gaps and lags $F_a(t)$, and reverse cumulative distribution function of the rejected gaps and lags $F_r(t)$. The gap value for which both the density functions attain same value is defined as the critical gap (t_c) (Fitzpatrick 1991; Raff and Hart 1950). The concept is shown in Figure 2.1.

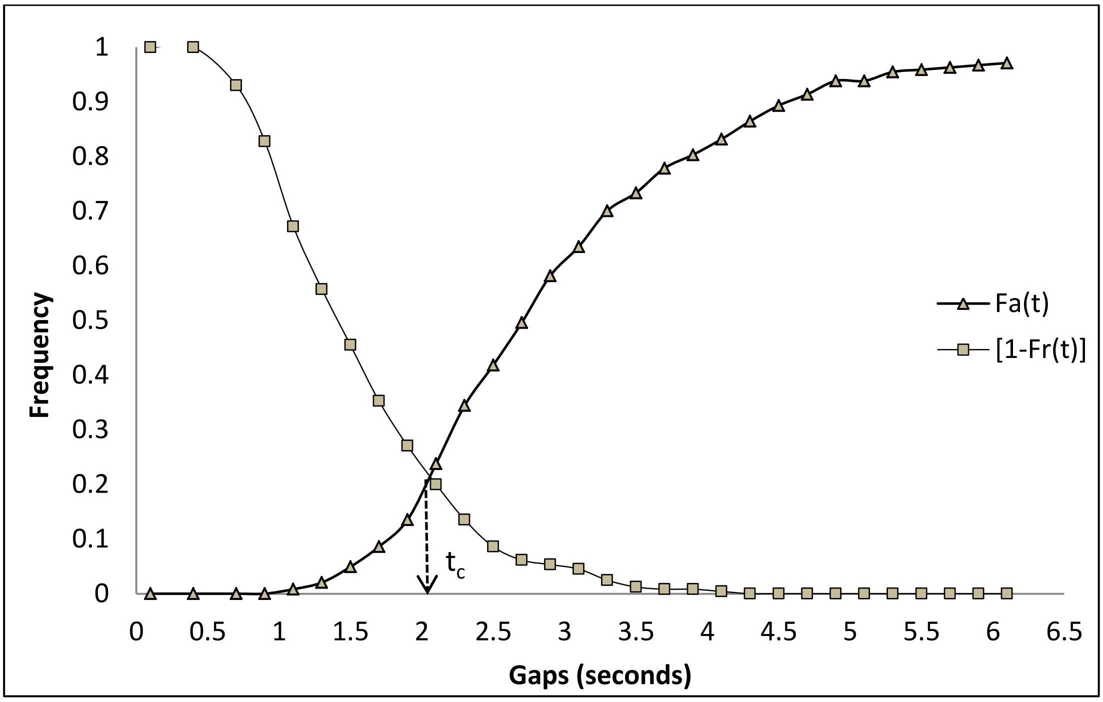


Figure 2.1 Critical gap by Modified Raff method

2.2.4 Logit Method

It is basically a weighted linear regression model with a mathematical form given by equation (2.17) and (2.18).

$$p_i(t) = \frac{1}{1+e^{\alpha+\beta_t}} \tag{2.17}$$

$$\int_0^{t_c} p_i(t)\,dt = 0.5 \tag{2.18}$$

Where,

$P_i(t)$ = probability of accepting a gap of size *i*

α and β = regression coefficients

2.2.5 Probit Method

Probit techniques for the estimation of critical gaps have been used since 1960s. The formulation for this type of models is quite similar to the logit concept. Probit transformation is given by equation (2.19). The critical gap is the value of i for which Y is 5.0.

$$Y = 5.0 + \frac{i - \mu}{\sigma} \tag{2.19}$$

Where,

Y = probit of *i*

μ = population mean

σ = standard deviation of the population

2.2.6 Maximum Likelihood Estimation Method

The maximum likelihood method of estimating critical gap is based on the fact that a driver's critical gap would lie between the range of (his) largest rejected gap and accepted gap. This method requires a pair of largest rejected gap (r_i) and accepted gap (a_i) in the circulating traffic flow for each driver sampled i; i=1 to n (Tian et al. 1999).

The following notations are used for subsequent equations:

$y_i = \ln(a_i)$ = the logarithm of the gap accepted by the i^{th} entering vehicle,

$x_i = \ln(r_i)$ = the logarithm of the largest gap rejected by the i^{th} entering vehicle,

$x_i = 0$, if no gap was rejected,

μ = mean of the distribution of the logarithms of the individual driver's critical gaps, i=1 to n.

σ^2 = variance of the distribution of the logarithms of the individual driver's critical gaps, i=1 to n.

f() = probability density function for the normal distribution

F( ) = cumulative distribution function for the normal distribution

The maximum likelihood of a sample of n drivers having a largest rejected gap, r_i, and accepted gap, a_i, is then given by equation (2.20).

$$\prod_{i=1}^{n}\left[F(y_i) - F(x_i)\right] \tag{2.20}$$

The logarithm L of the likelihood is then written as equation (2.21)

$$L = \sum_{i=1}^{n}\ln\left[F(y_i) - F(x_i)\right] \tag{2.21}$$

The likelihood, L, would be maximized when the two conditions are satisfied, i.e.

$$\frac{\partial L}{\partial \mu} = 0 \quad \text{and} \quad \frac{\partial L}{\partial \sigma^2} = 0 \tag{2.22}$$

By solving equation (2.21) and (2.22) the following two equations are established.

$$\sum_{i=1}^{n}\left[\frac{f(x_i)-f(y_i)}{F(y_i)-F(x_i)}\right]=0 \tag{2.23}$$

$$\sum_{i=1}^{n}\left[\frac{(x_i-\mu)f(x_i)-(\mu-y_i)f(y_i)}{F(y_i)-F(x_i)}\right]=0 \tag{2.24}$$

These two equations are solved iteratively to find out μ and σ^2. The mean critical gap is then given by equation (2.25).

$$t_c = e^{\mu+0.5\sigma^2} \tag{2.25}$$

2.2.7 Wu Method

Wu (2012) introduced a new model for calculating mean critical gap based on the macroscopic probability equilibrium of the accepted and rejected gaps. One aspect of this method is that the user does not need to assume a distribution type for the critical gaps and that the method does not involve an iterative process (Wu 2012). The following model is used for estimating the critical gap.

$$F_{tc}(t)=\frac{F_a(t)}{F_a(t)+\left[1-F_r(t)\right]}=1-\frac{\left[1-F_r(t)\right]}{F_a(t)+\left[1-F_r(t)\right]} \tag{2.26}$$

Where,

$F_a(t)$ = probability distribution function of the accepted gaps

$F_r(t)$ = probability distribution function of the rejected gaps

2.2.8 McGowen and Stanley Method

McGowen and Stanley (2012) proposed an alternative model for estimating the critical gap that could be used with naturalistic data. Equation (2.27) is maximized by an iterative process to find the maximum likelihood estimates of critical gap.

$$\sum_{all\_rejected} Ln\left[P\left(gap = g / rejected\right)\right] + \sum_{all\_accepted} Ln\left[P\left(gap = g / accepted\right)\right] \quad (2.27)$$

Where,

$$P\left(gap = g / rejected\right) = \frac{f_g(g)\left[1 - F_c(g)\right]}{\int_0^\infty f_g(g) F_c(g)\, dg} \quad (2.28)$$

$$P\left(gap = g / accepted\right) = \frac{f_g(g) F_c(g)}{\int_0^\infty f_g(g) F_c(g)\, dg} \quad (2.29)$$

f_g = probability density function of the circulating stream

g = gap size (s)

This proposed model was compared with the maximum likelihood method through a Monte-Carlo simulation. Both the proposed model and the maximum likelihood model assume a functional form for the distribution of the critical gap and both have a complicated mathematical solution that requires an iterative method to solve. Maximum likelihood method required using all gaps, accepted and rejected, for estimating the critical gap. The proposed method use only the gaps accepted or only the gaps rejected. This could come in handy if a data set contained only one type of gap.

2.2.9 Average Central Gap Method

This method is very simple to estimate the critical gap. The critical gap is assumed to be the midway point between largest rejected gaps and accepted gaps. So, average of all the largest rejected gaps and accepted gaps will give the critical gap value (Bunker 2014). The equation for estimating the critical gap value is given below:

$$\bar{t}_c = \frac{1}{2n} \sum_{i=1}^{n} \left(r_i + a_i\right) \quad (2.30)$$

Where,

r_i = largest rejected gaps by ith entering vehicle (s)

a_i = accepted gaps by ith entering vehicle (s)

2.2.10 Mode Central Gap (MCG) Method

MCG method considered the whole sample of drivers' critical gap bandwidths, between largest rejected gap (m_i) and accepted gap (a_i). This required determination, for a vector of possible critical gaps in Δ s increments, $t_{min}+\Delta j$; for j=0 to m. For this method, t_{min}, Δ and m need to be chosen judiciously by inspection of the data. Equation (2.31) explains the determination of the number of drivers whose critical gap bandwidth lies within each possible sample critical gap.

$$N(j)=\left[\sum_{i=1}^{n} if\left(and\left((t_{\min}+\Delta j)\geq m_i,(t_{\min}+\Delta j)\leq a_i\right),1,0\right)\right];\ \text{j=0 to m} \tag{2.31}$$

The mode number of drivers is then given by:

$$N_{\max,j}=\max{}_{j=0}^{m}\left(N(j)\right) \tag{2.32}$$

The mean critical gap is then given by:

$$t_c=t_{\min}+\Delta\left[\frac{\sum_{j=0}^{m} if\left(N_j=N_{\max j},j,0\right)}{\sum_{j=0}^{m} if\left(N_j=N_{\max j},1,0\right)}\right] \tag{2.33}$$

Where,

t_{min} = minimum gap in the circulating flow (s)

Δ = increment in the gaps (s)

Researchers have worked on these methods to examine their effectiveness, estimation accuracy and acceptability. Miller (1972) compared different methods of critical gap estimation by using simple gap acceptance model. The study found that maximum likelihood and Ashworth method gave acceptable results. Brilon et al. (1999) compared Lag, Harders, Raff, Ashworth, Logit, Probit, Hewitt, Maximum likelihood and Siegloch methods of critical gap estimation using simulation. They found that the Maximum likelihood method and Hewitt's method gave the best results. Guo (2010) found on the base of video survey of Shuma Square roundabout in Dalian, China that Ashworth's method gave the highest value and other methods had a little difference because Ashworth's method uses only accepted gap, but modified Raff's method and maximum likelihood methods use both accepted gap and rejected gap. Vasconcelos et al. (2013) suggested that Raff, Wu and maximum likelihood methods are more reliable than other methods. Troutbeck (2014) compared the Wu method and maximum likelihood method and found that maximum likelihood method was slightly better than Wu method. If drivers are inconsistent, then also the maximum likelihood method is reported to be superior. Patil and Sangole (2015, 2016) also found that the maximum likelihood method gave the consistent results.

The above discussion indicates that among various methods, only the maximum likelihood method (MLM) is suggested by researchers to be the most accurate and reliable. This method requires data on both rejected gaps and accepted gaps by a vehicle. It utilizes the data in pairs of the highest rejected gap and the next accepted gap. If there is no rejection for a particular vehicle, as in the case of limited priority or no priority condition, the maximum rejected gap would be zero and log natural of zero would be undefined. Consequently, this method may yield some trivial results in the case of limited priority condition. To deal with such a situation a very small value of gap is to be assigned in the iterative procedure while working with MLM method.

2.2.11 Critical Gap and Follow-up Time

Brilon (1988) reported that a constant ratio of 0.60 exists between follow-up time and critical gap. Tian et al. (2000) found that the major factors affecting critical gap and follow-up time include geometric layout, driver behavior, vehicle characteristics, and traffic conditions. The follow-up time to critical gap ratio was found to be approximately 0.60. Hagring et al. (2003) found that the ratio of critical gap to follow-on time ranged from 1.4 to 1.6 and close to the ratio in Sweden, which was 1.7.

Hagring (2000b) found that the critical gaps differ between the outer and inner lanes. The critical gaps were about 0.4 seconds longer in the inner lane than the outer lane due to a more difficult interaction. Hagring (2000a) developed a model for estimating the critical gap as a function of weaving section length, width of the weaving section, and inner-outer of entry lane. This is given by equation (2.34).

$$T = 3.91 - 0.0278 * L + 0.121 * W + 0.592 * (N_L - 1) \tag{2.34}$$

Where,

T = critical gap (s)

L = length of the weaving section (m)

W = width of the weaving section (m)

N_L = lane number (outer lane =1, inner lane = 2)

Chodur (2005) developed equations of basic capacity parameters, i.e., critical gap and follow-up time. Field studies of critical gaps and follow-up times were performed on 14 urban roundabouts in Poland. All of them were small-diameter and single lane roundabouts. The developed equations for follow-up time and critical gap are given as follows:

$$t_f = 0.31 * D_{ex} - 0.0044 * D^2{}_{ex} + 0.00052 * N_i - 2.59 \quad (2.35)$$

$$t_c = 1.92 * t_f + 0.316 * b_e - 0.427 * w_i - 0.126 * D_{ex} - 0.00198 * v_{ce} \quad (2.36)$$

Where,

t_f = follow-up time (s)

t_c = critical gap (s)

D_{ex} = external roundabout diameter (28–44 m)

w_i = width of the approach lane on (3.0–5.0 m)

N_i = size of town, described by the number of inhabitants (19.6–740.0 thousand inhabitants)

b_e = distance between the collision point of entering stream and circulating flow and the point where the vehicles diverge (16.2-23.0 m)

v_{ce} = circulating flow at entry *e* (134-481 veh/h)

Dahl and Lee (2012) found that the critical gap and the follow-up time were longer for trucks than for cars. To reflect the effect of trucks on the capacity, gap-acceptance parameters were determined for cars and trucks separately. Then the representative gap-acceptance parameters for the entire entry flow were calculated as a volume-weighted average of the parameters for cars and trucks. If the entry flow consists of cars and trucks only, the critical gap and the follow-up time were calculated with the following equations:

$$t_c^{'} = t_{c,car} * (1 - p_{truck}) + t_{c,truck} * p_{truck} \quad (2.37)$$

$$t_f^{'} = t_{f.cc} * (1 - p_{truck})^2 + (t_{f.ct} + t_{f.tc}) * (1 - p_{truck}) * p_{truck} + t_{f.ff} * p_{truck}^2 \quad (2.38)$$

Where,

t'_c = adjusted critical gap (s)

p_{truck} = proportion of trucks

$t_{c,car}$ and $t_{c,truck}$ = critical gaps for cars and trucks, respectively (s)

t'_f = adjusted follow-up time (s)

$t_{f,cc}$, = follow-up times for a car following a car (s)

$t_{f,ct}$ = follow-up times for a truck following a car (s)

$t_{f,tc}$ = follow-up times for a car following a truck (s)

$t_{f,tt}$ = follow-up times for a truck following a truck (s)

Many researchers from different countries reported the critical gap and follow up time values as given in Table 2.2.

Table 2.2 Critical gap and follow up time values as reported in literature

Researcher's Name	**Geometry of roundabout**	**Critical gap (s)**	**Follow-up time (s)**	**Country**
Troutbeck (1989)	1-lane	1.4 – 4.9	1.8 – 2.7	AUSTRALIA
	2-lane (dominant lane)	1.6 – 4.1	1.8 – 2.2	
	2-lane (subdominant lane)	-	2.2 – 4.0	
Flannery and Datta (1997)	-	4.0	2.5	U. S.
HCM (2000)	-	4.1-4.6	2.6-3.1	U. S.
Hagring et al. (2003)	2-lane roundabouts (L)	4.4-4.6	-	SWEDEN
	2-lane roundabouts (R)	4.0-4.3	-	
Tolazzi (2004)	-	4.8	2.9	SLOVENIA
Polus et al. (2005)	1-lane, urban/sub-urban	4.0	-	ISRAEL
NCHRP Report-572 (2007)	1- lane roundabouts	4.2-5.9	2.6-3.1	U. S.
	2- lane roundabouts (L)	4.2-5.5	3.1-4.7	
	2- lane roundabouts (R)	3.4-4.9	2.7-4.4	
Guo (2010)	2- lane roundabouts	2.65	-	CHINA
HCM (2010)	1- lane roundabouts	5.19	3.19	U. S.
	2- lane roundabouts (L)	4.29	3.19	

	2- lane roundabouts (R)	4.11	3.19	
Mensah et al. (2010)	-	2.5-2.6	-	MARYLAND
Brilon (2011)	[1/2] 40 ≤ Inscribed circle Dia ≤ 60 m	5.6	2.5	GERMANY
	[2/2] compact 40 ≤ Inscribed circle Dia ≤ 60 m	5.2	2.2	
	[2/2] large Inscribed circle Dia > 60 m	4.4	2.9	
Greibe (2011)	1-lane, urban	5.1	3.0	DENMARK
	1-lane, rural	4.7	3.0	
	2-lane, rural	4.0	2.6	
Romana (2011)	-	3.3-3.5	1.65-1.75	SPAIN
Tracz et al. (2011)	Medium 2-lane (L)	4.3	3.3	POLAND
	Medium 2-lane (R)	4.6	3.6	
	Large 2-lane (L)	3.8	2.6	
	Large 2-lane (R)	4.2	2.9	
	Semi 2-lane	4.7	2.8	
Mahesh et al. (2014)	2- lane roundabouts	2.15	1.26	INDIA

[1/2] = One lane at entry and two lanes at circulating roadway; [2/2] = Two lanes at entry and circulating roadway; (L) = Left entry lane; (R) = Right entry lane;

A look at the critical gap and follow-up time values indicate that both the values are quite high in U.S. and European countries. The critical gap values are found to be varying between 4.0 s and 5.9 s and that of follow-up time are varying between 2.2 s and 4.7 s. Relative to these, the values in Australia are found to be quite low. These are 1.4 s to 4.9 s for critical gap and 1.8 s to 4.0 s for follow-up time. In China, it is 2.65 s for critical gap. In India, it is 2.15 s for critical gap and 1.26 s for follow-up time. The values are found influenced by the number of lanes in the circulating area, as well as, with the relative location of lane. In some studies the values are also found getting influenced by the inscribed circle diameter and location of roundabout say in urban or rural area.

Critical gap and follow-up time are used as major parameters in the estimation of entry capacity. The accuracy of capacity estimation is mainly dependent on the accuracy with which the critical gap is estimated. The follow-up time was reported as 0.6 to 0.7 times of the critical gap. In Indian study it has been found to be 0.59 times of critical gap. Moreover, implementing agencies may found it difficult to estimate critical gaps and follow-up-times at different locations and in different cities because these are traffic and location specific. In such a condition a simpler approach is needed which is usable by people working in different walks of life. It has been also reported that the critical gap and the follow-up time were longer for trucks than for cars. In general, these are longer for bigger vehicles due to their maneuverability restrictions.

2.3 STUDIES ON CAPACITY ESTIMATION

Researchers have estimated the entry capacity of a roundabout based on three approaches, namely weaving based approach, gap-acceptance based approach and empirical approach or regression based approach. Now days, the gap acceptance approach and empirical approach are in use for estimating the entry capacity at a roundabout. Both approaches describe the entry capacity as a function of circulating flow. In general, as the circulating flow decreases, the entry capacity increases due to higher opportunities available to drivers for entering into the circulating flow. The gap acceptance approach estimates the entry capacity based on decision taken by the driver of entering vehicle regarding perceived traffic conditions and their ability to use the available gaps between circulating vehicles. On the other hand, the empirical approach estimates the entry capacity based on the observed capacity of the existing roundabouts which have been built in the past.

2.3.1 Weaving Section Approach

In 1957, Wardrop tested a number of different weaving sections on an artificial test track and developed a capacity model. The capacity of a traffic circle was calculated by an assumption that the intersection consisted of a series of weaving areas. Figure 2.2 shows the dimensions of weaving section. The capacity formula is given by equation (2.39) (Wardrop 1957).

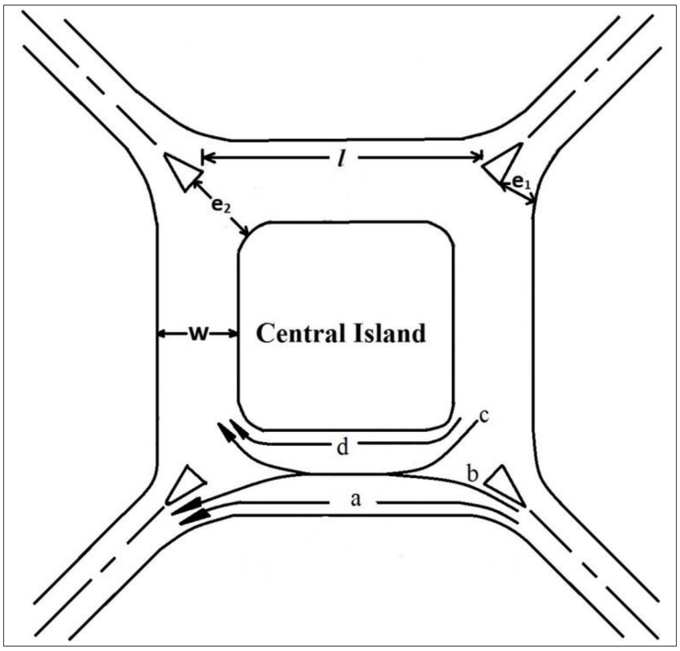


Figure 2.2 Relevant dimensions of weaving section

$$Q_p = \frac{108 * w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}} \tag{2.39}$$

Where,

Q_p = practical capacity of the weaving section of the traffic circle (pcu/h)

w = width of weaving section (ft.)

e = average entry with (ft.)

l = length of the weaving section between the ends of channeling islands (ft.)

p = proportion of weaving traffic

The formula was empirical in nature. It was based on geometric feature of the intersection and the proportion of the traffic moving in the weaving section.

The official design formula in Britain for conventional roundabouts was used until 1975. In 1966, the priority to the right rule was introduced for all roundabouts throughout Britain. Later it was noted that the adoption of the priority to the right rule in Britain has made Wardrop's formula less applicable (Pearce 1987). The new rule did not just prevented locking up, but also improved the general performance (Waddell 1997). The formula used in Britain was revised and is given by equation

(2.40) (Advisory-Manual 1968). The notations used in the formula are the same as presented before.

$$Q_p = \frac{282 * w\left(1 + \frac{e}{w}\right)\left(1 - \frac{p}{3}\right)}{1 + \frac{w}{l}} \qquad (2.40)$$

IRC-65 (1976) 'Recommendation Practice for Traffic Rotaries' used the above-mentioned concept of weaving length and weaving section. The weaving length determines the case with which the vehicles can maneuver through the weaving section and thus determines the capacity of the rotary. The formula for the practical capacity of the weaving section as given by equation (2.40) was modified and is shown as equation (2.41).

$$Q_p = \frac{280 * w\left(1 + \frac{e}{w}\right)\left(1 - \frac{p}{3}\right)}{1 + \frac{w}{l}} \qquad (2.41)$$

Where,

Q_p = practical capacity of the weaving section of the rotary (pcu/h)

w = width of weaving section in meters

$$w = \frac{e_1 + e_2}{2} + 3.5 \qquad (2.42)$$

$$e = \frac{e_1 + e_2}{2} = \text{average entry with (m)} \qquad (2.43)$$

e_1 = entry width (m)

e_2 = width of non-weaving section (m)

l = length of the weaving section between the ends of channeling islands (m)

$$p = \frac{b + c}{a + b + c + d} = \text{proportion of weaving traffic} \qquad (2.44)$$

Roundabout design in Malaysia is also based on the concept of weaving length and weaving section (Arahan Teknik 1987). The capacity is calculated as given by equation (2.45). It can be seen that no weightage is given to the traffic in weaving section. It further means that the capacity is considered as the function of roundabout geometric rather than traffic flow condition in the roundabout.

$$Q_p = \frac{160 * w\left(1+\frac{e}{w}\right)}{1+\frac{w}{L}} \tag{2.45}$$

Where,

Q_p = practical capacity of the weaving section of the roundabout (pcu/h)

w = width of weaving section (m)

$$e = \frac{e_1 + e_2}{2} = \text{average entry width (m)} \tag{2.46}$$

e_1 = entry width (m)

e_2 = width of non-weaving section (m)

L = length of the weaving section between the ends of channeling islands (m)

The method proposed in Indonesian Highway Capacity Manual (IHCM 1993) is based on calculating the base capacity (C_0) and then applying correction factors to it. The base capacity is the function of geometric features of the weaving section. Corrections are applied to take into consideration the influence of the size of the city and friction caused on the road. The capacity is calculated as given by equation (2.47):

$$C = C_0 * F_{CS} * F_{RF} \tag{2.47}$$

Where,

$$C_0 = \text{Base Capacity} = \frac{135 * w^{1.3} * \left(1+\frac{e}{w}\right)^{1.5} * \left(1-\frac{p}{3}\right)^{0.5}}{\left(1+\frac{w}{L}\right)^{1.8}} \tag{2.48}$$

w = width of weaving section (m)

$$e = \frac{e_1 + e_2}{2} = \text{Average entry width (m)} \tag{2.49}$$

L = weaving length (m)

p = proportion of weaving traffic

F_{CS} = correction factor for base capacity due to city size. This factor is decided from Table 2.3.

F_{RF} = correction factor for base capacity due to road environment and side friction type. This factor is decided from Table 2.4.

Table 2.3 City size correction factor

City Size	No. of Inhabitants (Millions)	City Size Correction Factor F_{CS}
Small	< 0.3	0.83
Medium	0.3-1.0	0.94
Large	1.0-3.0	1.00
Very large	> 3.0	1.05

Table 2.4 Road environment type and side friction correction

Road Environment Type Class	Correction Factor (F_{RF})	
	Low Side Friction	High Side Friction
Commercial	1.00	0.94
Residential	1.00	0.97
Restricted access	1.00	1.00

Basically this was re-calibration of the formula used in Britain and as given by equation (2.40). Constant was changed and power function was used to suit local traffic condition on a roundabout in Indonesia. Higher weightage is given to roundabout geometrics than traffic in weaving section.

2.3.2 Gap Acceptance Approach

The researchers have also modelled the gap acceptance behavior of drivers at an entry of an approach so as to estimate the entry capacity of that approach on a roundabout. The priority goes to the circulating flow around the inside island. In such models the geometric features of a roundabout have not being, generally, considered. Various such studies are presented and discussed in the following paragraph.

Tanner (1967) gave a formula to estimate the capacity of an un-signalized intersection. It is given by equation (2.50) as follows:

$$Q_e = \frac{3600 * q_c * (1 - \Delta.q_c) * e^{-q_c(T-\Delta)}}{1 - e^{-q_c T_0}} \qquad (2.50)$$

Where,

Q_e = entering capacity (veh/h)

q_c = circulating traffic flow (veh/s)

Δ = minimum headway in the circulating streams, in seconds

T = critical gap (s)

T_0 = follow-up time (s)

The approach indicated towards the need of estimating critical gap and follow-up time. This further necessitated the extraction of accepted and rejected gaps, and a suitable estimation method for a critical gap. Various such methods are already being discussed under section 2.2. Another shift in the approach was the consideration given to circulating traffic flow rather than to a proportion of weaving traffic. The estimation became more tedious due to involvement of micro-analysis of the traffic flow on a roundabout.

Fisk (1989) extended Tanner's model for the capacity of multilane roundabouts by differentiating between the circulating traffic stream and different critical gaps for individual circulating lanes. The form of the model was the same but without a constant value as shown in equation (2.51).

$$Q_e = 3600 * q_c \frac{\left(1 - q_{c_1} * t_m\right) * \left(1 - q_{c_2} * t_m\right) .... \left(1 - q_{c_n} * t_m\right)}{\left(1 - e^{-q_c * t_f}\right)} * e^{-\sum_{i=1}^{n} q_{c_i}(t_{ci} - t_m)} \qquad (2.51)$$

Where

Q_e = entry capacity, (veh/h)

q_{ci} = circulating traffic volume in lane i, (veh/s)

q_c = total circulating traffic volume, (veh/h)

t_{ci} = critical gap for circulating traffic stream in lane i, (s)

t_f = follow-up time (s)

t_m = minimum headway (s)

Troutbeck (1991) presented an entry capacity formula for Australian conditions and found that most vehicles travel within two stages i.e. bunched vehicle stage in which vehicles follow preceding vehicles and free vehicle stage in which vehicles travel without interaction with preceding vehicles. The entry capacity formula is given by equation (2.52).

$$Q_e = \frac{3600 * (1-\theta) * Q_c * e^{-\lambda(T-\Delta)}}{1-e^{-\lambda T_0}} \tag{2.52}$$

Where,

Q_e = entry capacity (veh/h)

Q_c = circulating flow (veh/s)

θ = proportion of bunched vehicles

$$\lambda = \text{decay constant} = \frac{(1-\theta)Q_c}{1-\Delta Q_c} \tag{2.53}$$

Δ = minimum headway in the circulating streams, in seconds

T = critical gap (s)

T_0 = follow-up time (s)

The formula indicates that as circulating flow increases for a particular value of headway and proportion of free vehicles, decay constant also increases, and consequently the entry capacity, which is the function of negative exponential of decay constant, decreases.

The French formula for the estimation of entry capacity (pcu/h), is based on the regression analysis, and is given by equation (2.54). This method considers the disturbing flow in front of entry, follow up time and roundabout geometries (Guichet 1997).

$$C = A.e^{-\frac{C_B * Q_d}{3600}} \tag{2.54}$$

Where,

$$A = \frac{3600}{T_f}\left(\frac{W_e}{3.5}\right)^{0.8} \tag{2.55}$$

T_f = follow-up time (s)

W_e = entry width (m)

C_B = coefficient (3.525 for urban areas and 3.625 for rural areas)

Q_d = disturbing flow in front of the entry (pcu/h)

The disturbing flow is the flow which affects the entry flow. It is equal to circulating flow in some capacity formulas. According to other formulas, it is equal to appropriate combinations of the circulating flow and exiting traffic flow.

$$Q_d = Q_u.k_a.\left(1 - \frac{Q_u}{Q_u + Q_c}\right) + Q_{ci}.k_{ti} + Q_{ce}.k_{te} \tag{2.56}$$

Q_u = exiting traffic flow (pcu/h)

$Q_c = Q_{ci} + Q_{ce}$ = circulating flow in front of the entry (pcu/h)

Q_{ci} = circulating flow on the far lane (pcu/h)

Q_{ce} = circulating flow on the near lane (close to the entry) (pcu/h)

$$K_a = \begin{cases} \dfrac{R}{R+W} - \dfrac{L}{L_{max}} & \text{for } L < L_{max} \\ 0 & \text{for other cases} \end{cases} \tag{2.57}$$

R = central-island radius (m)

W = circulating roadway width (m)

L = splitter island width (m)

$$L_{max} = 4.55\sqrt{R + \frac{W}{2}} \tag{2.58}$$

$$k_{ti} = \text{Min}\left\{\frac{160}{W*(R + W)} \quad \text{or} \quad 1\right. \tag{2.59}$$

$$k_{te} = \text{Min}\left\{1 - \frac{(W-8)}{W}\left(\frac{R}{R+W}\right)^2 \quad \text{or} \quad 1\right. \tag{2.60}$$

These formulae are based on exhaustive traffic studies done in France on roundabouts. It considers width of entry with respect to standard lane. But its effect is considered in reducing form. Coefficient C_B looks synonymous to critical gap. The effect of exiting flow is also considered for entry capacity. It is one of the few recommendations which considered exiting traffic also.

Hagring (1998) derived a general capacity formula for the case of more than one major lane and allowed the major lanes to differ in critical gaps and follow-up times. This distribution was used to deduce various capacity formulas. The capacity of a minor stream that has to cross *n* major lanes, having critical gaps and follow-on times that differ by lane, was expressed as equation (2.61).

$$C = 3600 * \Lambda \prod_i \frac{\alpha_i q_i}{\lambda_i} \frac{e^{-\sum_k \lambda_k T_k}}{e^{-\Lambda\Delta}\left(1 - e^{-\sum_k \lambda_k T_{0k}}\right)} \tag{2.61}$$

Where,

Δ = minimum headway between two vehicles (s)

λ = decay constant

$$\lambda = \frac{\alpha q}{1 - q\Delta} \tag{2.62}$$

q = circulating flow (veh/s)

$$\Lambda = \Sigma_i \lambda i, \tag{2.63}$$

T_k and T_{0k} are the critical gaps and follow-up times. The indices *i* and *k* refer to the different lanes.

α = proportion of free vehicles, i.e. those not driving in platoons

The work seems to be on extensions of work of Troutbeck (1991). Instead of bunched vehicles, it talks of free vehicles while computing decay constant. Other constituents are made more generalized in form so as to consider wide options.

Many researchers developed the equations for estimating the proportion of free vehicles as given in Table 2.5.

Table 2.5 Equations for proportion of free vehicles

Researcher's Name	**Model**	**Country**
Tanner (1962)	$\alpha = 1 - \Delta q$ $\quad \Delta = 2$	U.K.
Hagring (1996)	$\alpha = 0.914 - 1.54\Delta q$ $\quad \Delta = 1.8$	Sweden
Sullivan and	$\alpha = e^{-Aq}$ $\quad A = \begin{cases} 7.50 \text{ for median lane} \\ 5.25 \text{ for other lanes} \end{cases}$	Australia

Troutbeck (1997)		
Manage et al. (2003)	$\alpha = 0.9043 * e^{-2.764\,\Delta.q_c}$ $\quad \Delta = 1$	Japan
Tanyel and Yayla (2003)	$\alpha = \begin{cases} 1.25 - 1.13\Delta q & \text{if } q > 0.22 \text{ with } \Delta = 2 \\ 1 & \text{otherwise} \end{cases}$	Turkey
Akcelik (2006)	$\alpha = \dfrac{1-\Delta q}{1-(1-k_d)\Delta q}$ $\quad \Delta = 2,\ k_d = 2.2$	Australia
Çalişkanelli et al. (2009)	$\alpha = \begin{cases} 1.11 - 1.47\Delta q & \text{if } q > 0.07 \text{ with } \Delta = 2 \\ 1 & \text{otherwise} \end{cases}$	Turkey
Vasconcelos et al. (2012b)	$\alpha = \begin{cases} \dfrac{1-\Delta q}{1-A} & \text{if } q > \dfrac{A}{\Delta} \text{ with } \Delta = 2 \\ 1 & \text{otherwise} \end{cases}$	Portugal

Troutbeck and Kako (1999) used the concept of limited priority instead of absolute priority for major stream vehicles over those of the minor stream vehicles, because the entering traffic does not always completely yield to the circulating traffic. The data were collected using a videotape recorder at three locations in Brisbane, Australia. They estimated the entry capacity of the roundabout as a function of the critical gap, the follow-up time and the headway between bunched vehicles. This is given by equation (2.64).

$$Q_e = 3600 * \frac{\alpha * Q_c * C * e^{-\lambda(t_c - \Delta)}}{1 - e^{-\lambda * t_f}} \tag{2.64}$$

Where,

Q_e = entering capacity (veh/h)

Q_c = circulating flow (veh/s)

α = proportion of free vehicles in the circulating stream

Δ = headway between bunched vehicles in the circulating streams

t_c = the critical gap (sec)

t_f = the follow-up time (sec)

λ = decay constant = $\dfrac{\alpha * Q_c}{1 - \Delta Q_c}$ (2.65)

$$C = \frac{e^{\lambda * t_f} - 1}{e^{\lambda * t_f} - e^{-\lambda * \beta} - \lambda * \beta * e^{-\lambda * \beta}} \tag{2.66}$$

$$\beta = t_c - t_f - \Delta \tag{2.67}$$

Wang et al. (2001) proposed a gap acceptance model for estimating the entry capacity. It is expressed by the following equation (2.68).

$$C_e = 3600 \frac{q e^{-\lambda(t_c - t_m)}}{1 - e^{-\lambda t_f}} \left(1 - \frac{2 q_1 q_2}{q_1 + q_2} t_m\right) + 3600 \frac{\alpha_2 q_2 e^{-\lambda(t_c - t_m)}}{1 - e^{-\lambda_2 t_f}} \tag{2.68}$$

$$\lambda = \lambda_1 + \lambda_2 = \frac{\alpha_1 q_1}{1 - t_m q_1} + \frac{\alpha_2 q_2}{1 - t_m q_2} \tag{2.69}$$

$$q = q_1 + q_2 \tag{2.70}$$

C_e = theoretical entry capacity (pcu/h)

q_1, q_2 = traffic flow of inner side and outer side of the circle (pcu/s)

t_c = critical gap

t_m = minimum gap in the circulating flow

t_f = follow up time

α = ratio of free flow

α_1 and α_2 = parameters that refer to values as given in Table 2.6.

Table 2.6 Value of ratio of free flow (α)

Circulating flow (pcu/h)	**<500**	**500-600**	**600-800**	**800-1000**	**1000-1200**
α_1, α_2	1.0	0.9	0.8	0.7	0.6

Li et al. (2003) developed entry capacity model for Chinese traffic conditions which was composed of ‘r’ representative vehicles of Type 1 to Type r with the proportion of these vehicles as p_1 to p_r.

$$C = \Lambda \prod_{j=1}^{m} \frac{\alpha_j q_j}{\lambda_j} * \frac{\left\{ \left(1 - e^{-\sum_{k=1}^{m} \lambda_k T_{f1}^{k}} \right) * \left(\sum_{j=1}^{r} p_j * e^{-\sum_{k=1}^{m} \lambda_k \left(T_{cj}^{k} - \Delta \right)} \right) \right\}}{\left(1 - \sum_{j=1}^{r} p_j * e^{-\sum_{k=1}^{m} \lambda_k T_{fj}^{k}} \right)} \tag{2.71}$$

Where,

m = number of circulating lanes

Δ = minimum headway between two vehicles (sec)

$$\lambda = \frac{\alpha q}{1 - q\Delta} \quad \text{= decay constant} \tag{2.72}$$

q_j = flow rate of j^{th} circulating lane (vps)

α_j = proportion of free vehicles in j^{th} circulating lane

$\Lambda = \Sigma_j \lambda_j$, (2.73)

$T_{f1} < T_{f2} < T_{f3} \ldots.. < T_{fr}$

T_c and T_f are the critical gaps and follow-up times.

Polus et al. (2003) incorporated both the geometry and the critical gap as independent parameters in the roundabout entry capacity model. The data were collected at seven urban and suburban roundabouts in Israel. They developed an exponential entry-capacity model that depended on the outside diameter of the central-island, the circulating traffic and the critical gap. The model for entry capacity is given by equation (2.74).

$$V_e = 394\, D^{0.31}\, e^{-0.00023\, t_{cr} V_c} \tag{2.74}$$

Where,

V_e = approaching entry capacity (vph)

V_c = circulating volume around the central-island (vph)

t_{cr} = critical gap (s)

D = outside diameter (m.)

Chodur (2005) estimated capacity of entering movement based on observations collected at the 14 small roundabouts in Poland. Two different forms of equations were developed for Polish conditions. These equations gave almost identical results. These are given by equations (2.75) and (2.76).

$$C_{p,e} = \frac{3600}{t_f} * e^{-0.9*\left(v_{c,e}/3600\right)*\left(t_c - 0.5*t_f\right)} \tag{2.75}$$

$$C_{p,e} = \frac{3600}{t_f} * e^{-\left(v_{c,e}/3600\right)*\left(t_c - 0.5*t_f - 0.3\right)} \tag{2.76}$$

Where,

$C_{p,e}$ = potential entry capacity (pcu/h)

$v_{c,e}$ = circulating traffic flow (veh/h)

t_c = critical gap (s)

t_f = follow-up time (s)

Brilon and Wu (2006) proposed the model for estimation of roundabout entry capacity based on an idea from Tanner (1967) as cited by Mauro and Branco (2010). This is given by equation (2.77). The method considers circulating flow, geometry of the roundabouts and traffic flow micro characteristics like critical gap, follow-up time and headway.

$$C = 3600\left(1 - \frac{\Delta.Q_c / 3600}{n_c}\right)^{n_c} * \frac{n_e}{T_f} * \exp\left[-Q_c / 3600\left(T_c - \frac{T_f}{2} - \Delta\right)\right] \tag{2.77}$$

Where:

C = entry capacity (pcu/h)

Q_c = circulating flow in front of the entry (pcu/h)

n_c = number of circular lanes

n_e = number of lanes in the subject entry

T_c = critical gap (s)

T_f = follow-up time (s)

Δ = minimum headway between the vehicles circulating in the circle

Diah et al. (2010) developed a flow rate model in Malaysia based on the weaving section of the roundabout. The model was then validated with data of weaving section of another Malaysian roundabout.

The model is given by equation (2.78).

$$Q_{wsf} = 2.658 + 0.000027\left(Q_{ncf}^{1.5} * Q_{cf}\right) - 1.09\left(T_{ISG} * Q_{cf}\right) \tag{2.78}$$

Where,

Q_{wsf} = total weaving section flow rate (pcu/h) = $V_{11}+V_{22}+V_{12}+V_{21}$

Q_{ncf} = total non-conflicting flow rate of weaving section (pcu/h) = $V_{11}+V_{22}$

Q_{cf} = total conflicting flow rate of weaving section (pcu/h) = $V_{12}+V_{21}$

T_{ISG} = ideal safe gap (s)

V_{11} = flow rate in inner lane (pcu/h)

V_{22} = flow rate in outer lane (pcu/h)

V_{12} = conflicting flow from inner to outer lane (pcu/h)

V_{21} = conflicting flow from outer to inner lane (pcu/h)

The ideal safe gap is defined as the gap which should be adequate for merging vehicles to change lanes without making or causing any harmful disruptions to the main traffic streams.

This model is based on flow rate in weaving section unlike geometric features as used in the previous model developed in Malaysia, which gave more weight to the geometric variables.

HCM (2010) presented an exponential model of capacity for single-lane and two-lane roundabouts. It is a combination of simple, lane-based regression (exponential) and gap-acceptance model. The roundabout capacity model for an entry lane is expressed as given by equation (2.79).

$$C_e = A * e^{-B*v_c} \tag{2.79}$$

Where,

$$A = \frac{3600}{t_f} \tag{2.80}$$

$$B = \frac{t_c - 0.5*t_f}{3600} \tag{2.81}$$

v_c = circulating traffic flow (pcu/h)

t_f = follow-up headway (s)

t_c = critical gap (s)

Wei and Grenard (2012) developed a calibrated HCM model at three single-lane roundabouts in Carmel, Indiana. It was found that the critical headway and the follow-up headway were lower than those from previous roundabout research in the United States and were significantly lower than the default values given in HCM 2010. It was concluded that the calibrated HCM model is consistent with field conditions for low to medium circulating flow rates (≤800 vph) and overestimates the capacity for high circulating flow rates (>800 vph).

Mahesh et al. (2014) examined the entry capacity of a roundabout under different circulating traffic flows by measuring the field entry flows. Two roundabouts of different diameter of central-island (25m and 37m) were taken up for the study. Queue formation in the approach is taken as an indicator that the approach is operating at capacity. Relationship between entry traffic flow and circulating flow was ascertained and it was found to be negative exponential in nature. Critical gap and follow-up time was extracted in Indian condition and the same was used to modify the HCM (2010) equation. Varying adjustment factors were proposed with respect to circulating traffic flow for both the roundabouts using the adjusted HCM (2010) model. Though, this approach tried to improve upon the existing method available as per IRC:65 (1976), but this remained lacking on two accounts. Firstly, this work proposed adjustment factors to be multiplied to the already adjusted HCM (2010) model (double adjustment needed), without giving an opportunity of direct computation of entry traffic flow. Secondly, the passenger car units (PCUs) used to make the heterogeneous traffic homogenous (as per the requirement of HCM model) were those from IRC: 65, a nearly 40 years old guideline. There is definitely a need to redefine the PCU factors for different category of vehicles looking at the variability and improvement in technology used in vehicles, and the changes being observed in the driver behavior over the years.

2.3.3 Geometry and Flow based Models

Kimber (1980) developed capacity model based on experimental observations of a large number of operating roundabouts in England. It has the following linear form:

$$C = k^*\left(F - f_c^*Q_c\right) \qquad \text{(veh/h)} \tag{2.82}$$

Where,

Q_c = circulating flow (veh/h)

C = entry capacity (veh/h)

$$F = 303 * x_2 \tag{2.83}$$

x_2 = entry adjustment factor

$$f_c = 0.2108 t_D * (1 + 0.2 * x_2) \tag{2.84}$$

$$k = 1 - 0.00347 * (\Phi - 30) - 0.978 * (1/r - 0.05) \tag{2.85}$$

$$t_D = 1 + \frac{0.5}{1 + \exp\left(\frac{D - 60}{10}\right)} \tag{2.86}$$

$$x_2 = v + \frac{(e - v)}{(1 + 2S)} \tag{2.87}$$

$$S = (e - v)/l \tag{2.88}$$

Table 2.7 defines the geometric parameters as used in the above relationships and the respective symbols used in the procedure as well as their ranges.

Table 2.7 Geometric parameters and symbols used in Kimber (1980) model

Parameter	**Description**	**Range values**
e	Entry width	3.6 – 16.5 m
v	Entry lane width	1.9 – 12.5 m
u	Circulatory roadway width	4.9 – 22.7 m
l	Flare length	1 – ∞ m
S	Sharpness of the flare	0 – 2.9
r	Entry bend radius	3.4 – ∞ m
Φ	Entry angle	$0 – 77^o$
$D = D_{ext}$	Inscribed circle diameter	13.5 – 171.6 m
W	Width of the weaving section	7.0 – 26.0 m
L	Weaving section length	9.0 – 86.0 m

Louah (1988) developed the entry capacity model for conditions prevalent in France. This is given by equation (2.89).

$$C = (1330 - 0.7Q_d)\left[1 + 0.1(w_e - 3.5\right] \tag{2.89}$$

Q_d is the disturbing traffic which is determined as given by equation (2.90).

$$Q_d = \left[Q_c + \frac{2}{3}Q_u\left(1 - \frac{w_m}{15}\right)\right]\left[1 - 0.085(w - 8)\right] \tag{2.90}$$

Where,

C = entry capacity (vph)

Q_u = exiting traffic flow (vph)

Q_c = circulating traffic flow (vph)

w_e = approach width (m)

w_m = width of entry island (m)

w = width of circulating area (m)

An entry capacity formula is recommended for Swiss roundabouts considering exit traffic, entering traffic and circulating traffic in front of the exit being considered by Bovy et al. (1991). This is given by equation (2.91).

$$C = \frac{1}{\gamma}.\left\{1500 - \frac{8}{9}\left(\alpha.Q_u + \beta.Q_c\right)\right\} \qquad \text{(pcu/h)} \tag{2.91}$$

Where,

Q_u = exiting traffic (pcu/h)

Q_e = entering traffic (pcu/h)

Q_c = circulating traffic (pcu/h)

Coefficients α, β and γ are related to the geometry of the roundabout, as given below.

γ = influence of number of entry lanes

$$\gamma = \begin{cases} 1 & \text{for single entry lane} \\ 0.6\text{–}0.7 & \text{for double entry lane} \\ 0.5 & \text{for triple entry lane} \end{cases} \tag{2.92}$$

α = influence of outgoing traffic

The value of ‘α’ are taken as given below:

$$\alpha = \begin{cases} 0.6 & \text{for} \quad 0 < l \le 9 \\ 0.6\text{- } 0.0416*(l\text{-}9) & \text{for} \quad 9 < l \le 21 \\ 0.1 & \text{for} \quad 21 < l \le 27 \\ 0.1 - 0.1*(l - 27) & \text{for} \quad 27 < l \le 28 \\ 0 & \text{for} \quad l > 28 \end{cases} \tag{2.93}$$

l = distance between the exiting conflicting point (A) and entering point (B) in meters as shown in Figure 2.3.

β = influence of number of roundabout lanes

$$\beta = \begin{cases} \text{0.9–1.0 for single circulating lane (default = 1.00)} \\ \text{0.6–0.8 for double circulating lane (default = 0.66)} \\ \text{0.5–0.6 for triple circulating lane (default = 0.55)} \end{cases} \tag{2.94}$$

Various factors as considered above are shown in Figure 2.3.

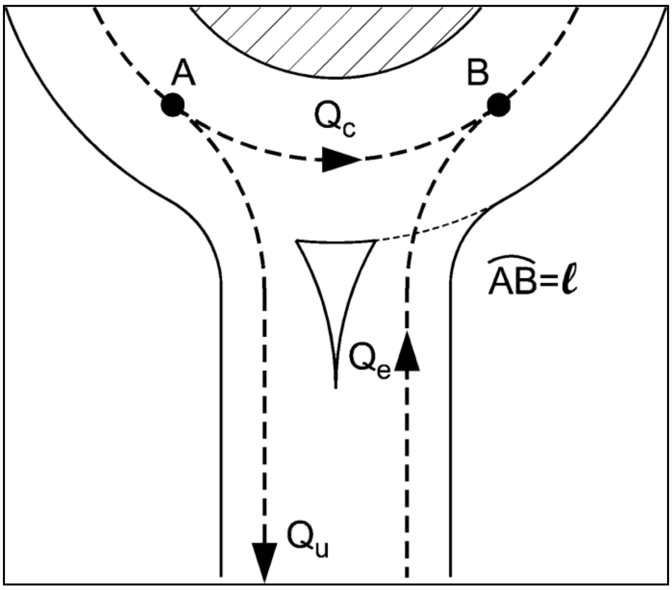


Figure 2.3 Capacity attributes for a roundabout

Al-Masaeid and Faddah (1997) developed an empirical model for estimating roundabout entry capacity in Jordan. The data was collected at ten roundabouts and regression analysis was used for developing the model. Model was a function of entry width, circulating width, diameter of the central-island, circulating traffic volume, and distance between the entry and its near exit. It is given by equation (2.95).

$$Q_e = 168.2\, D^{0.312}\, S^{0.219}\, e^{0.071\,EW + 0.019\,RW}\, e^{-\frac{5.602\, Q_c}{10000}} \tag{2.95}$$

Where,

Q_e = entry capacity (pcu/h)

Q_c = circulating traffic flow (pcu/h)

D = central-island diameter (m)

S = distance between the entry and near-side exit (m)

EW = entry width (m)

RW = circulating roadway width (m)

It was concluded that entry width and central-island diameter produced greatest effect while the circulating roadway width was found to have the least relative effect on the estimated entrance capacity.

Brilon et al. (1997) developed a model of an entry capacity based on data collected in Germany, which is represented by the simple linear relationship as given by equation (2.96).

$$C = A - B.Q_c \text{ (pcu/h)} \qquad (2.96)$$

Where,

A and B are parameters which depend on the numbers of entry and circulating lanes.

Q_c = Circulating flow (pcu/h)

Equation (2.96) is valid for roundabouts with inscribed circle diameters that range from 28 to 100 meter. Values of parameters A and B are given in Table 2.8.

Table 2.8 Parameters values for capacity formula (Brilon et al. 1997)

Circulatory lanes	Entry lanes	A	B
3	2	1409	0.42
2	2	1380	0.50
2–3	1	1250	0.53
1	1	1218	0.74

Polus and Shmueli (1997) developed an entry capacity model based on their studies of six roundabouts in Israel. It was an exponential model. The entry capacity was dependent upon circulating flow and inscribed circle diameter and is given by equation (2.97).

$$V_e = 394 * D^{0.31}\ e^{-(0.00095 * V_c)} \tag{2.97}$$

V_e = entry capacity (veh/h)

V_c = circulating flow (veh/h)

D = inscribed circle diameter of the roundabout (m)

The Australian capacity formula is published by Akcelik et al. (1998) and it is calculated lane by lane which are given as follows:

$$Q_e = f_{od} * Q_g \tag{2.98}$$

$$f_{od} = 1 - f_{qc}(p_{qd} * p_{cd}) \tag{2.99}$$

$$Q_g = \frac{3600}{\beta}\left(1 - \Delta_c * Q_c + 0.5 * \beta * \varphi_c * Q_c\right) e^{-\lambda(\alpha - \Delta_c)} \tag{2.100}$$

For single lane stream, the circulating flow is estimated as given by equation (2.101).

$$f_{qc} = \begin{cases} 0.04 + 0.00015 Q_c & \text{for } Q_c < 600 \\ 0.0007 Q_c - 0.29 & \text{for } 600 \le Q_c \le 1200 \\ 0.55 & \text{for } Q_c > 1200 \end{cases} \tag{2.101}$$

For multi-lane, the stream circulating flow is estimated as given by equation (2.102).

$$f_{qc} = \begin{cases} 0.04 + 0.00015 Q_c & \text{for } Q_c < 600 \\ 0.00035 Q_c - 0.08 & \text{for } 600 \le Q_c \le 1800 \\ 0.55 & \text{for } Q_c > 1800 \end{cases} \tag{2.102}$$

Where,

Q_e = capacity of the entry lane (veh/h)

Q_g = capacity estimate using the gap acceptance capacity method (veh/h)

f_{od} = factor to adjust the basic gap acceptance capacity for the O-D pattern

f_{qc} = a calibration parameter

p_{cd} = proportion of the total circulating flow that originated from the dominant approach

$$p_{cd} = Q_{cd} / Q_c \tag{2.103}$$

p_{qd} = proportion of the queued (stopped) vehicles

Q_c = total circulating flow rate (pcu/h)

Q_{cd} = part of the total circulating stream flow that originated from the dominant approach

Δ_c = minimum headway in circulating traffic

$$\left.\begin{array}{ll} \Delta_c = 2.0 \text{ and } \varphi_c = e^{-5Q_c} & \text{for single lane circulating flow} \\ \Delta_c = 1.2 \text{ and } \varphi_c = e^{-3Q_c} & \text{for two lane circulating flow} \\ \Delta_c = 1.0 \text{ and } \varphi_c = e^{-2.5Q_c} & \text{for multi-lane circulating flow} \end{array}\right\} \tag{2.104}$$

λ = arrival headway distribution factor, estimated as given by equation (2.105)

$$\lambda = \left\{\begin{array}{ll} \dfrac{\varphi_c Q_c}{1-\Delta_c Q_c} & \text{for } Q_c \leq 0.98/\Delta_c \\ \\ \dfrac{49\varphi_c}{\Delta_c} & \text{for } Q_c > 0.98/\Delta_c \end{array}\right. \tag{2.105}$$

For the dominant entry lane (lane at multi lane roundabout with the largest entry flow), parameter β is estimated by equation (2.106)

$$\left.\begin{array}{l} \beta = \beta_d = \beta_0' - 3.94 * 10^{-4} Q_c \\ \qquad \text{subject to } \beta_{min} \leq \beta_d' \leq \beta_{max} \\ \beta_0' = 3.37 - 0.0208 D_i + 0.889 * 10^{-4} D_i^2 - 0.395 n_e + 0.388 n_c \\ \qquad \text{subject to } 20 \leq D_i \leq 80 \end{array}\right\} \tag{2.106}$$

Where,

D_i = inscribed diameter (m)

n_e = number of entry lanes

n_c = number of circulating lanes

$\beta_{min} = 1.2\,s$

$\beta_{max} = 4.0\,s$

For the subdominant entry lane (lane at multi-lane roundabout with the smallest entry flow), parameter β is estimated by equation (2.107)

$$\beta = \beta_s = 2.149 + (0.5135\beta_d - 0.8735)r_{ds} \tag{2.107}$$

subject to $\beta_d \leq \beta_s \leq \beta_{max}$

Where,

$r_{ds} = \dfrac{Q_d}{Q_s} =$ raio of dominant (Q_d) and subdominant (Q_s) flow in the entry (2.108)

$\alpha =$ critical headway(s)

$$\alpha = \begin{cases} (3.6135 - 3.137 * 10^{-4} Q_c - 0.339 w_L - 0.2775 n_c)\beta & \text{for } Q_c \leq 1200 \\ (3.2371 - 0.339 w_L - 0.2775 n_c)\beta & \text{for } Q_c > 1200 \end{cases} \tag{2.109}$$

Subject to $3.0 \geq \dfrac{\alpha}{\beta} \geq 1$ and $2.2 \leq \alpha \leq 8.0$

w_L = average entry with (m)

Hossain (1999) developed the entry capacity model for Dhaka, Bangladesh by using regression analysis. The entry capacity is related with circulating flow, road width, inscribed circle diameter, percentage of non-motorized vehicles and percentage of heavy vehicles.

$$Q_e = -0.82Q_c + 300W_d + 4.7D_i + 3.8P_{nmv} - 19.6P_{hv} \tag{2.110}$$

Q_e = entry capacity (veh/h)

Q_c = circulating flow (veh/h)

W_d = width of entry and circulating road widths (m)

D_i = inscribed circle diameter (m)

P_{nmv} = percentage of non-motorized vehicles

P_{hv} = percentage of heavy vehicles

Chik et al. (2004) identified the correlation between the circulating flows, entry flows and the entry width based on statistical analysis using data collected in Malaysia. The entry flows for both single and multilane entries were found to be highly dependent on the circulating flow. Following linear regression relationships between the entry and circulating flow were derived for both the single and multi-lane entry:

For multi-lane entry:

$$Q_e = -0.7743Q_c + 2044.9 \qquad (2.111)$$

For single-lane entry:

$$Q_e = -0.6481Q_c + 1061.2 \qquad (2.112)$$

Where,

Q_e = maximum entry flow (pcu/h)

Q_c = maximum circulating flow (pcu/h)

Bared and Afshar (2009) proposed planning capacity models for two- lane and three-lane roundabouts by separate entry-lane and separate circulatory-lane traffic volumes. Capacity models have been developed by using U.S. data from NCHRP Report-572. All capacity models were sensitive to the circulating volumes in each circulatory lane. In addition, the capacity of the right lane is also a function of R_t, the ratio of right-turning vehicles to total entering flow. However, R_t is not significant in the capacity models for the left and middle lanes, and is therefore omitted from entry capacity model estimations for left lane and middle lane. A comparison of the capacity of entry lanes reveals that the right lane has the highest capacity and the middle lane has a higher capacity than the left lane. The developed capacity models are given below:

$$E_L = e^{\left(7.0754 - \frac{1.1864*c_1}{1000} - \frac{1.0813*c_2}{1000} - \frac{0.9479*c_3}{1000}\right)} \qquad R^2 = 0.955 \qquad (2.113)$$

$$E_M = e^{\left(7.0754 - \frac{0.6758*c_1}{1000} - \frac{1.1556*c_2}{1000} - \frac{0.9049*c_3}{1000}\right)} \qquad R^2 = 0.980 \qquad (2.114)$$

$$E_R = e^{\left(7.0754 - \frac{0.5569*c_1}{1000} - \frac{0.9044*c_2}{1000} - \frac{1.0258*c_3}{1000} + 0.2795*R_t\right)} \qquad R^2 = 0.955 \qquad (2.115)$$

Where,

E_L= entry capacity for left lane (vph)

E_M = entry capacity for middle lane (vph)

E_R = entry capacity for right lane (vph)

c_1 = circulating flow of inner lane (vph)

c_2 = circulating flow of middle lane (vph)

c_3 = circulating flow of outer lane (vph)

R_t = ratio of right-turning vehicles to total entering flow in the desired entry approach

Prakash (2010) suggested an empirical equation for estimating entry capacity of roundabouts using circulating flow and geometric parameters (Central island diameter, number of entry lanes and number of circulating lanes) of the roundabouts in India. It was found that number of entry lanes and diameter of central island positively affect the entry capacity while circulating flow and number of circulating lanes has negative effect. Linear regression was performed to have a relationship between these parameters and entry capacity which is given by equation (2.116).

$$C_e = 1116 – 0.429*Q_c + 5.79*D + 842.18*N_e – 426.33*N_c \qquad (2.116)$$

Where,

C_e = entry capacity (pcu/h)

Q_c = circulating flow (pcu/h)

D = central island diameter (m)

N_e = number of entry lanes

N_c = number of circulating lanes

Wei et al. (2011) developed entry capacity model based on actual flow rates at three congested roundabouts in Carmel, Indiana. The capacity was estimated using equation (2.117).

$$C = -0.8698 * V_c + 1503 \qquad (2.117)$$

Where,

C = capacity of the approach (veh/h)

V_c = circulating traffic flow (veh/h)

Chandra and Rastogi (2012) proposed a method to determine the entry capacity of a roundabout which considers only the circulating flow. The data were collected at four roundabouts in the suburban area of Chandigarh city. . The peak one hour period was selected to compute the entry volume and circulating volume. The volume counts were taken at each leg of the roundabout for consecutive 5 min intervals. The 5 min counts were converted to the hourly volumes and plotted against each other as shown in Figure 2.4. To determine the entry capacity, tangents were drawn from two limbs of the parabola and the intersection points of these tangents gave the capacity of the approach at respective circulating flow.

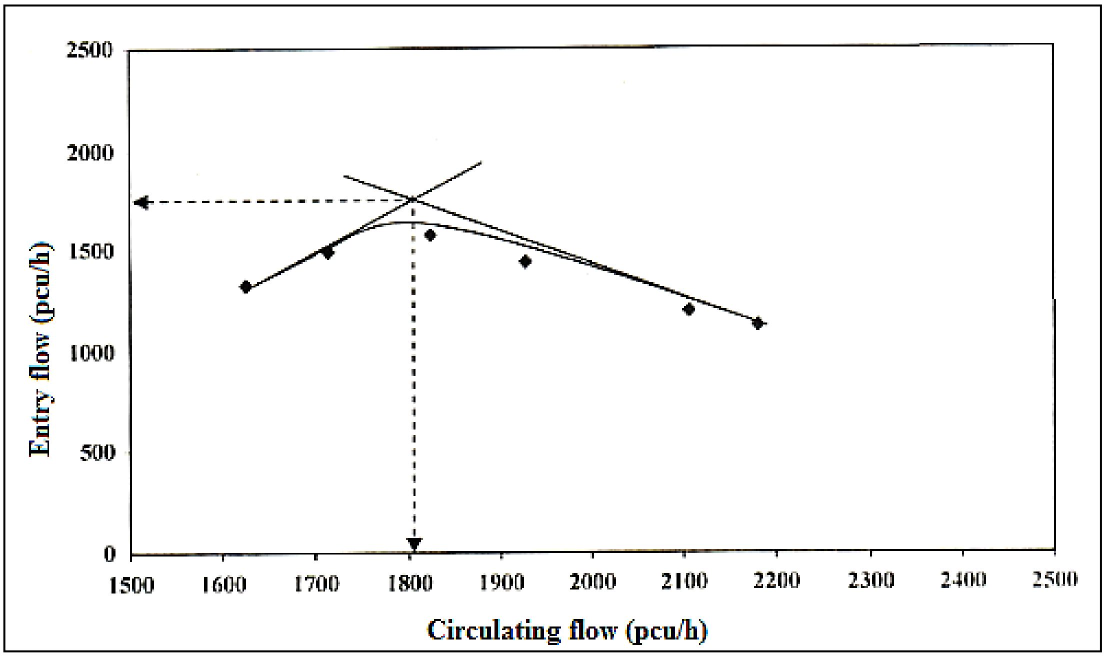


Figure 2.4 Entry capacity based on circulating flow and entry flow

Piras and Pinna (2013) developed entry capacity model for standard and non-standard leg of the roundabouts. A roundabout is non-standard when it has not followed the design rules, for example, one of legs without deflection. Traffic volume data were collected for each leg of several roundabouts. Only urban roundabouts in Cagliari, Italy were taken into account.

For standard legs, the entry capacity model is given as equation (2.118) below:

$$Q_e = 1366.49 * e^{-(0.001Q_c)} \tag{2.118}$$

For non-standard legs, the entry capacity is given as equation (2.119) below:

$$Q_e = 713.37 * e^{-(0.00003Q_c)} \tag{2.119}$$

Q_e = entry capacity (pcu/h)

Q_c = circulating traffic flow (pcu/h)

2.3.4 Miscellaneous

Çalişkanelli et al. (2009) applied regression analysis method to compare the capacity models. The data was collected at four multi-lane and five single-lane roundabouts in Izmir, Turkey. They found that the gap acceptance method gives more accurate results than the other models. Mazzella et al. (2011) considered a geostatistical approach to establish the relationship between entry capacity and circulating flow. It was emphasized that the relationship between entry capacity and circulating flow cannot be expressed by one trend only but by two or three trends. Dahl and Lee (2012) found that the observed capacity was lower for the roundabout with a higher truck percentage. As truck percentage increased, the critical gap and the follow-up time for the roundabout increased. This resulted in lower capacities. The results showed that the capacity decreased as truck percentage increased, but the amount of capacity reduction is less at higher circulating flow. The method by Chandra and Rastogi (2012) gave the capacity, quite comparable to the German entry capacity model. Indian model gave the highest capacity amongst all the methods i.e. UK, Swiss, HCM and German model. However, Indian model is based on the capacity of weaving section which can accommodate the least traffic. Among other four methods, UK model gave the highest entry capacity and US model gave the lowest capacity.

Based on the review of literature, it has been appeared that the initial formula for the capacity estimation of a roundabout was given by Wardrop in 1957. It was based on the weaving section of the roundabout and considered the width of the entry, length and width of the weaving section and proportion of the weaving traffic. The IRC code (IRC-65) estimates the weaving capacity of the roundabout based on the modified empirical formula that was used in Britain in 1950's to 1970's. The formula is more than around 40 years old. In mid-1960's, the priority rule was implemented in Britain. The subsequent studies carried out on the validation of Wardrop's formula under new traffic control condition indicated that it was not valid and gave inaccurate results. It was also concluded in some of the studies that traffic in weaving section also does not affect the capacity of the roundabout. The design formula in Britain was modified in 1980 and the capacity of the roundabout was modeled as a function of geometric parameters and circulating flow. Since mid-1990's the focus of the researchers shifted to the estimation of the entry capacity of the roundabout leg. The studies were based on the vehicle gap available and the circulating flow around the central-island. The variables mostly used were critical gap, circulating flow, headways and follow-up time. Some researchers also incorporated the physical parameters of the roundabout like number of circulating lanes and entry lanes, the inscribed diameter, the diameter of central-island, etc. The impact of exiting traffic and presence of heavy vehicles on entry capacity was also examined as an individual parameter.

Most of the studies carried out in the UK, USA, Europe and Australia have been on homogeneous traffic movements. The studies with heterogeneous traffic moving around the roundabout were carried out in Malaysia, Indonesia, Israel and Jordan. The direct transferability of the methods reported in literature to Indian conditions is doubtful. Few attempts have been made in India to analyze the traffic flow and estimate the entry capacity of the roundabout. In one of the studies, it was solely dependent on the circulating flow, whereas, in another study it was correlated with circulating flow, central island diameter and number of lanes at the entry and in circulating area. No comprehensive study is being carried out to develop a formula to estimate the capacity of the roundabout and thus to update and revise, a nearly 40 years old IRC code. There is a need to incorporate the new research in the estimation of the roundabout capacity. The impacts of the mix of variables under mixed traffic condition need to be studied. A general capacity estimation formula needs to be developed on the basis of the results and conclusions.

2.4 SUMMARY

The following points emerged from the review of literature on passenger car units, gap acceptance parameters and roundabout capacities:

i. Many studies found that Roundabouts have better performance in terms of capacity and delay than any other intersection. It has been also reported that the crashes involving pedestrians and bicyclists were also reduced after the conversion into roundabouts.

ii. The estimation of passenger car unit (PCU) for different vehicles to convert heterogeneous traffic into homogeneous is a well-accepted procedure. The researchers have adopted different methods for the estimation of PCU values. These studies are mainly related to the single-lane roads, two-lane roads, multi-lane roads or signalized intersections. Very few studies had been done in the area of un-signalized intersection or roundabouts. Suggested PCU values for Indian conditions are not recent as given in IRC-65 (1976), and needs a re-look based on the field studies.

iii. There are many methods to estimate critical gaps and the applicability of one out of all always remains questionable. The above studies of the research articles explain that best suited method for estimating the critical gap is maximum likelihood method (MLM). This method requires data on both rejected gaps and accepted gaps by a vehicle. The follow-up time to critical gap ratio is reported to be approximately 0.60.

iv. In India, the capacity of roundabouts is based on weaving theory, as discussed already. No comprehensive study is being carried out to change the weaving section based formula of roundabout capacity by an entry capacity formula. Giving consideration to the outdated capacity formula of the roundabout, which is based on 40 years old research, there is a need to replace it by an entry capacity formula, in consonance with the new research in this field. Based on the cited research, factors like entry width, circulating roadway width, circulating traffic flow, central-island diameter, number of entry / circulating lanes, exiting traffic flow, etc. were considered while developing the entry capacity model. The research in the recent year has demonstrated the applicability of circulating flow and gaps along with the physical parameters in the estimation of the roundabout entry capacity. A proper mix of the effect of circulating traffic flow, physical parameters of the roundabout and gaps need to be examined on the capacity of the roundabout.

CHAPTER 3:
RESEARCH METHODOLOGY

3.0 GENERAL

The objectives laid for this study, based on the study of literature; the local traffic conditions prevailing in developing countries; the guidelines laid for the analysis of roundabouts / rotaries in different countries with specific context of India; and the shortcomings being observed in the related approaches; were achieved by formulating a research methodology. This is being discussed in this chapter. It has following main parts apart from study of literature: site selection and data collection, data extraction, data analysis, model development and inferring.

Data collection is a very important part of any traffic engineering study and the success of the effort is heavily dependent on the quality of data. This chapter explains the sites selected based on the needs arising to satisfy the objectives and the roundabouts having typical mixed traffic flow and geometric characteristics. The technique used for data collection, as well as, of data extraction for further analysis is also being described. The present research work is mainly focused on entry capacity analysis of the roundabouts under mixed traffic conditions. The critical gap and follow-up time are used for estimating the entry capacity in gap acceptance approach; hence the gap acceptance analysis is studied under heterogeneous condition. The PCU of a vehicle type is also required in quantifying the entry capacity and hence the PCU values are studied under varying traffic conditions on the roundabouts. The statistical procedure used in the development of entry capacity model and the sensitivity analysis with respect to influencing variables is also enumerated. The sequential procedural steps involved in the whole research work, are given in Figure 3.1.

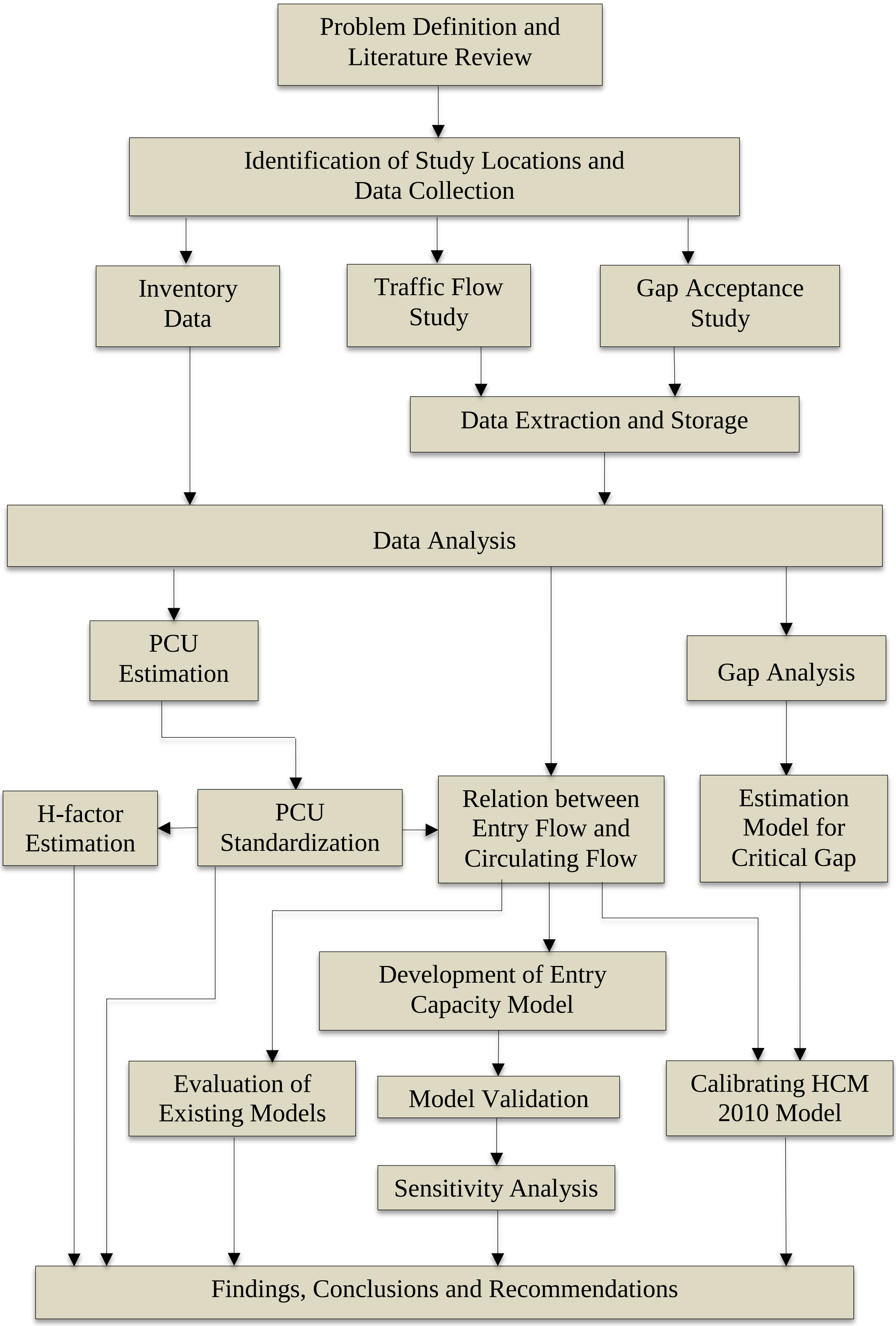


Figure 3.1 Flow chart for research methodology

3.1 APPROACH TO FIELD DATA COLLECTION

3.1.1 Selection of Study City

Proper identification of study areas is the core of any field study. The roundabouts for this study were selected in three cities namely, Chandigarh, Noida and Panchkula. All of these cities are in North India and are known for roundabouts at intersection. They have been developed as urban areas with specific aims. A brief outline of urban form used in these cities is discussed in successive subsections.

3.1.1.1 Chandigarh

Chandigarh city is known as ‘City Beautiful’ because of its unique concept. It is one of the greenest city of India with its 1400 nos. green belts / parks/ gardens. It is expanded in 114 km^2 area. As per provisional reports of census India, population of Chandigarh city is in 2011 is 1,055,450. It is a union territory and is planned in a grid pattern. One unique feature in the layout of Chandigarh is its roads, classified in accordance with their functions. The urban space is divided into 56 identical sectors with roads crossing each other. An integrated system was designed to ensure efficient traffic circulation. They intersect at right angles, forming a grid of network for movement. This arrangement of road-use leads to a remarkable hierarchy of movement, which also ensures that the residential areas get segregated from the noise and pollution generated by traffic. Most of the crossings at the interface of four sectors have roundabouts.

3.1.1.2 Noida

Noida is located in Gautam Buddh Nagar district of Uttar Pradesh state and is developed as a commercial and information technology hub adjacent to the national capital New Delhi. It is expanded in 203 km^2 area. As per provisional reports of census India, population of Noida in 2011 is 637,272. Noida has been planned in a grid pattern. The major roads have been planned horizontally from southwest to northeast interconnected by perpendicular roads forming a grid and dividing the area into sectors. The township is planned on the concept of self-contained integrated township. The high-density residential areas are located close to the work places. The commercial centers are well distributed over space with the main commercial hub in the City Centre.

3.1.1.3 Panchkula

Panchkula was planned as a satellite city of Chandigarh in Haryana state and is just 10 kilometers away from the city. Panchkula district is expanded in 816 km^2 area. As per provisional reports of census India, population of Panchkula city is in 2011 is 211,355. This city is also planned in a grid pattern. The township has been sub-divided into residential sectors, industrial sectors, parks and areas for regional recreation, major institutions, markets, and government and semi-government offices. Major roads spanning length and breadth of this city have roundabouts at their intersections.

3.1.2 Selection of Roundabouts

Following features are given consideration while selecting the roundabouts in a city:

a) Each roundabout should have four approaches which are mutually perpendicular,

b) The traffic operation is uncontrolled i.e. not having a traffic signal or deployment of a police personal to control or regulate traffic,

c) All roundabouts are at ground level,

d) Roundabouts are free from the effect of bus stop locations, parked vehicles or any other side friction which may cause any restriction to traffic movement on it, and

e) Roundabouts are free from the interference by pedestrian and cycle flows.

Based on the above considerations, nine roundabouts are selected in Chandigarh city, and one each in Noida and Panchkula. The locational identification for these roundabouts is given in Table 3.1. Location of these cities in India and location of roundabouts in a city are shown in Figure 3.2. The photograph of these roundabouts is given in Figure 3.3 for better understanding of flow spaces and patterns on these roundabouts.

Table 3.1 Locational identification of selected roundabouts

S.N.	**City**	**Location of roundabout**	**Identification code used in study**
1	Chandigarh	Sector 35-36	R_1
2	Chandigarh	Sector 42-43	R_2
3	Chandigarh	Sector 46-47	R_3
4	Noida	Shashi Chowk (Sector 36-39)	R_4
5	Chandigarh	Sector 41-42-53-54	R_5
6	Chandigarh	Sector 7-19-26-27	R_6
7	Chandigarh	Sector 29-30-31-32	R_7
8	Chandigarh	Sector 7-8-18-19	R_8
9	Chandigarh	Sector 42-43-52-53	R_9
10	Panchkula	Sector 5-6-7-8	R_{10}
11	Chandigarh	Sector 16-17-22-23	R_{11}

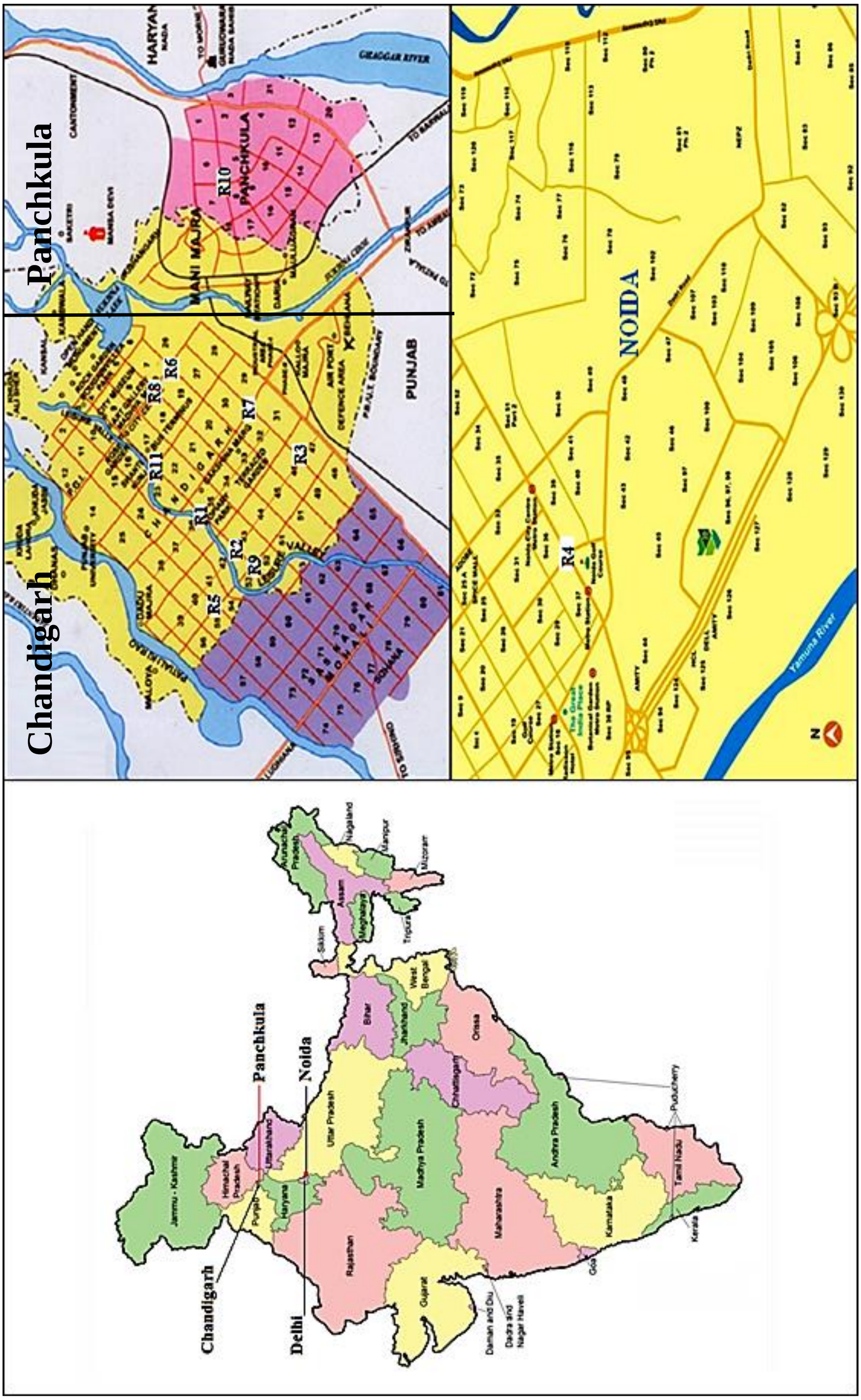


Figure 3.2 Location of cities in India and location of roundabouts in a city

a) R_1

b) R_2

c) R_3

d) R_4

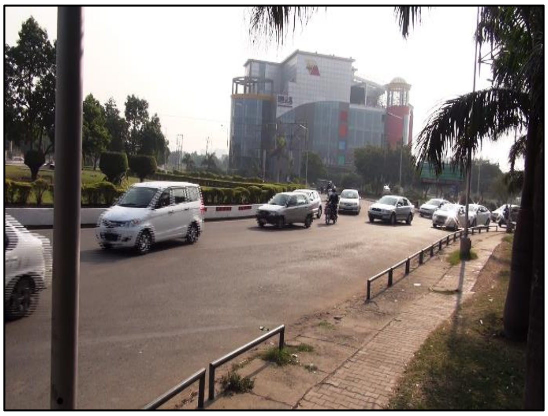
e) R_5

f) R_6

g) R_7

h) R_8

i) R_9



j) R_{10}

k) R_{11}

Figure 3.3 The photograph of selected roundabouts

3.1.3 Data Collection

For any traffic study, data collection is extremely important and it is to be carried out very carefully. The accuracy and care with which the data collection is being carried out in turn greatly affects the results. The data collected can be broadly classified into two categories, inventory data and traffic flow data. The inventory data includes the geometric details of the roundabout like circulating roadway width, entry width, and central-island diameter, weaving length, etc. The inventory data are collected using a measurement tape and a measuring wheel. The geometric features of the roundabout are given in Table 3.2.

Table 3.2 Inventory details at different roundabouts

Intersection ID	Central island diameter	Circulating roadway width	Entry width	Exit width	Weaving length
R_1^*	25.0	8.0	7.0	7.0	28.0
R_2^*	37.0	7.0	8.5	8.5	33.0
R_3^*	37.0	8.0	8.0	8.0	36.0
$R_4^\#$	48.0	16.0	12.0	12.0	48.0
R_5^*	49.0	9.5	13.0	12.0	40.0
R_6^*	50.0	9.5	12.0	11.0	45.0
R_7^*	50.0	10.0	14.7	15.5	43.0
R_8^*	50.0	13.0	12.5	13.0	50.0
R_9^*	51.0	10.5	12.0	12.0	40.0
$R_{10}^\$$	76.0	10.0	9.7	9.7	55.0
R_{11}^*	85.0	12.5	9.3	10.5	65.0

*Chandigarh City # Noida, Near Delhi $ Panchkula

The physical parameters of the selected roundabouts are found varying as – 25 m to 85 m for central island diameter, 7 m to 16 m for circulating roadway width, 7 m to 14.7 m for entry width, 7 m to 15.5 m for entry width, and 28 m to 65 m for weaving length. All the selected roundabouts have more than one-lane in entry approach, as well as, in the circulating area of a roundabout.

The video recording technique is used to collect the traffic flow data at roundabouts. This technique of data collection has the following advantages.

a) Video camera acts as a real time data source and enables extraction of data whenever required.

b) Many round of data retrieval can be done for each vehicle by using the software, as many times as needed.

c) The manpower requirement drastically reduces as compared to manual method of data collection.

d) The data can be extracted with sufficient level of accuracy.

e) Actual driver behavior can be captured using video camera, as the drivers of moving vehicles are unaware of the data collection process.

A video camera installed at a sufficiently high location, usually on a 12 feet tripod stand or at the top of a road-side high rise building. The data collection has been done in the months of September to November 2013, which are considered to be the normal months as the traffic flow is least affected by the environmental influences during this period. The video has been captured either from 8 a.m. to 12 a.m. or from 4 p.m. to 7 p.m. on a typical clear weekday.

3.2 DATA EXTRACTION

As video-graph technique has been used in this study, the traffic related data were extracted by playing the video at later point of time in the office. This was done especially for the following:

a) Traffic flows from entry of approaches and within circulation area around the the central-island

b) Micro traffic flow data like lagging headway between the vehicles on circulating roadway, and

c) Micro traffic flow data related to vehicular gaps within circulating roadway being accepted or rejected by the entering vehicles

d) The extraction of these data is now discussed in the subsequent sub-sections. The type of data extracted and its use are shown in a chart given in Figure 3.4.

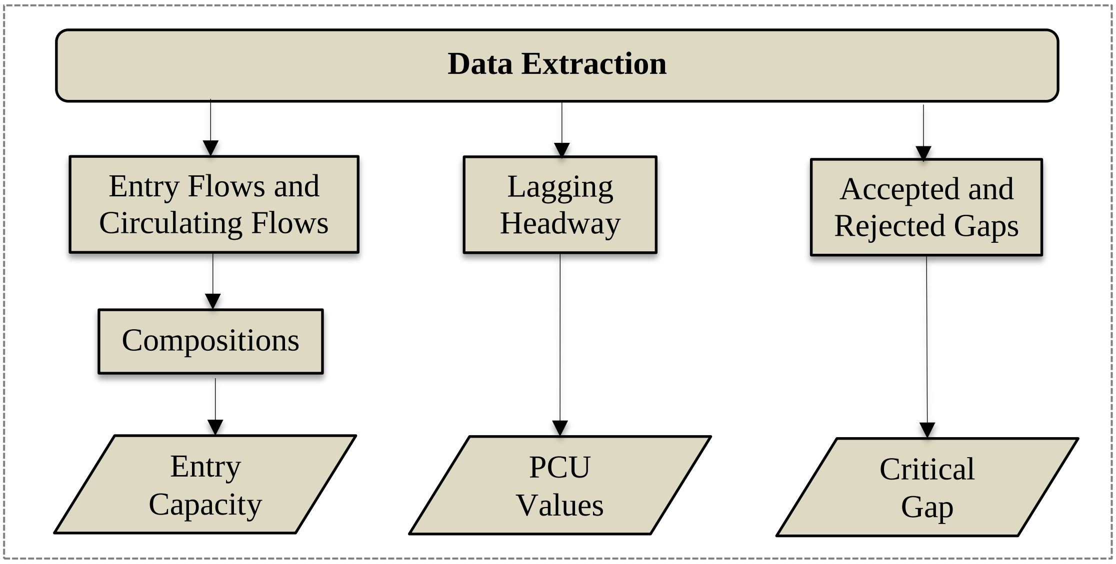


Figure 3.4 Flow chart for data extraction

3.2.1 Traffic Flow at Entry and Circulating Roadway

The video was replayed to examine the time period during which queue of vehicles was formed on an approach of a roundabout was noted. The flow during this period would give the estimation of the field entry capacity i.e. the maximum number of vehicles that can enter the roundabout while having a stable queue and the corresponding circulating traffic flow that allows the vehicle entry as well as queue dissipation. The entering vehicles and circulating vehicles are counted at entry and circulating section of a roundabout marked by **A** and **B**, respectively, as shown in Figure 3.5.

Figure 3.5 Markings at entry and circulating section of a roundabout for flow measurement

The range of time period for which stable queue was formed at an approach of a roundabout, the traffic flow that was measured while entering the roundabout from that approach (synonymous to field entry capacity) and the corresponding traffic flow in the circulating area are all recorded in Table 3.3. The traffic flow has been measured in vehicles during the period of queue discharge and is multiplied with the appropriate factor to convert it into equivalent hourly flow.

Table 3.3 Range of queue discharge period and traffic flow from entry and on circulating roadway

Roundabout ID	**Queue discharge period (Sec.)**	**Entry Flow* (Veh/h)**	**Flow on Circulating Roadway* (Veh/h)**
R_1	34 - 124	857 - 1500	1101 - 2304
R_2	42 - 142	1317 – 2152	643 - 1758
R_3	32 - 133	1420 - 2046	667 - 1515
R_4	41 - 157	898 - 3104	370 - 2906

R_5	32 - 137	1293 - 2124	786 - 1737
R_6	34 - 143	1596 - 2690	234 - 1493
R_7	30 - 118	1226 - 2263	642 - 2328
R_8	35 - 141	1477 - 2625	397 - 1671
R_9	34 - 125	1221 - 1834	1070 - 1972
R_{10}	38 - 127	1817 - 2697	653 - 1645
R_{11}	36 - 154	1532 - 2470	1063 - 2368

*Equivalent hourly flow

With the propose of data extraction, the vehicles are divided into five different categories as motorized two-wheeler (2W), motorized three-wheeler (3W), small car (SC) or standard car, big car (BC) and heavy vehicle (HV). Physical dimensions and rectangular plan area of these vehicles are given in Table 3.4. Cars are also divided into two categories as small car (standard car) and big car. Small car represents all cars having length of 3.72 m, width 1.44 m and engine power of up to 1400 cc. The big car is the one having engine power of above 1400 cc and up to 2500 cc, average length and width being 4.58 m and 1.77 m, respectively (Dhamaniya and Chandra 2013).

Table 3.4 Vehicle categories and their dimensions

Vehicle type	Vehicles included	Length (m)	Width (m)	Rectangular plan area(m^2)
2W	Scooter, Motorcycles	1.87	0.64	1.20
3W	Auto rickshaw*	3.20	1.40	4.48
SC	Car, Van below 1400cc	3.72	1.44	5.36
BC	Car, SUV, MUV	4.58	1.77	8.11
HV	Truck, Bus	10.10	2.43	24.54

Source: Dhamaniya and Chandra (2013) * Motorized Three-wheeler

The average compositional share of each category of vehicle and traffic flow in the entry and circulating traffic stream on different roundabouts is given in Table 3.5.

Table 3.5 Average flow and traffic composition details at different roundabouts

Roundabout ID - Flow Entity		**Proportion of Vehicles, %**					**Average Flow (Veh/h)**
		2W	**3W**	**SC**	**BC**	**HV**	
R_1	Entry flow	42	4	41	12	1	1361
	Circulating flow	40	14	37	7	2	1932
R_2	Entry flow	53	7	36	2	2	1721
	Circulating flow	51	9	31	6	3	2795
R_3	Entry flow	45	4	41	8	2	2028
	Circulating flow	39	7	39	11	4	1220
R_4	Entry flow	32	11	44	10	3	1543
	Circulating flow	34	15	41	7	3	3215
R_5	Entry flow	40	8	37	10	5	1953
	Circulating flow	36	6	41	11	6	2055
R_6	Entry flow	41	17	33	6	3	2536
	Circulating flow	46	15	30	8	1	1309
R_7	Entry flow	36	10	40	11	3	1872
	Circulating flow	42	13	32	8	5	2367
R_8	Entry flow	38	14	34	10	4	1656
	Circulating flow	41	9	40	7	3	3175
R_9	Entry flow	48	5	33	10	4	1909
	Circulating flow	41	10	36	8	5	2405
R_{10}	Entry flow	38	7	42	10	3	2100
	Circulating flow	40	6	47	6	1	3058
R_{11}	Entry flow	48	4	35	11	2	2354
	Circulating flow	43	7	39	8	3	2569
Average	Entry flow	42	8	38	9	3	1912
	Circulating flow	41	10	38	8	3	2373

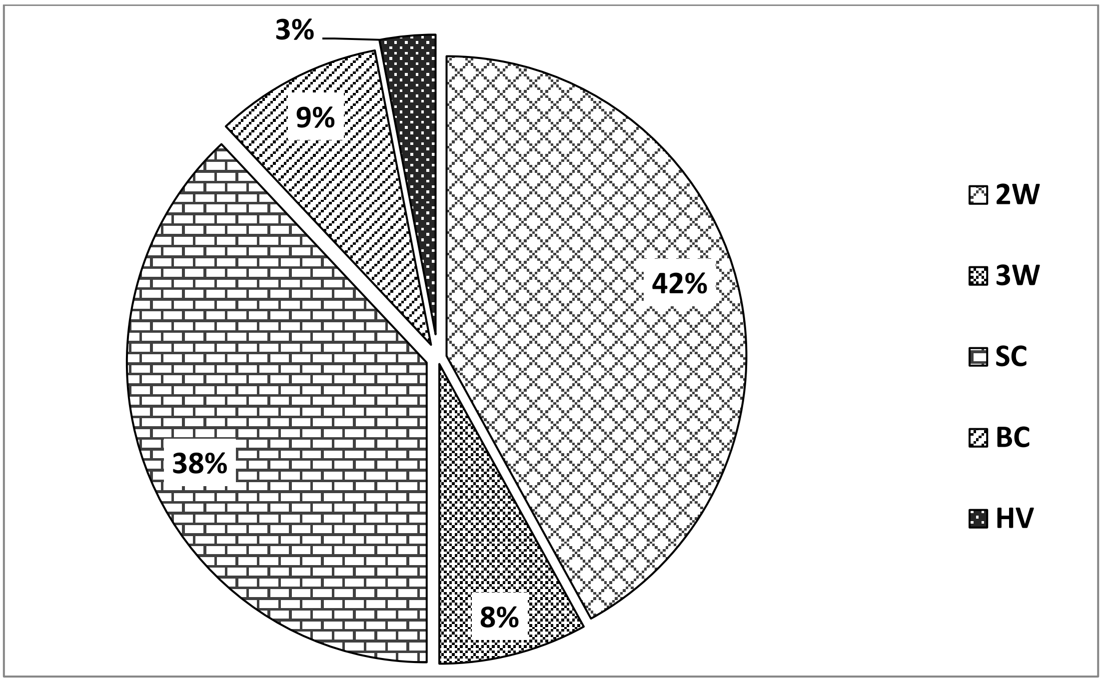


Figure 3.6 Average traffic composition of entry flow

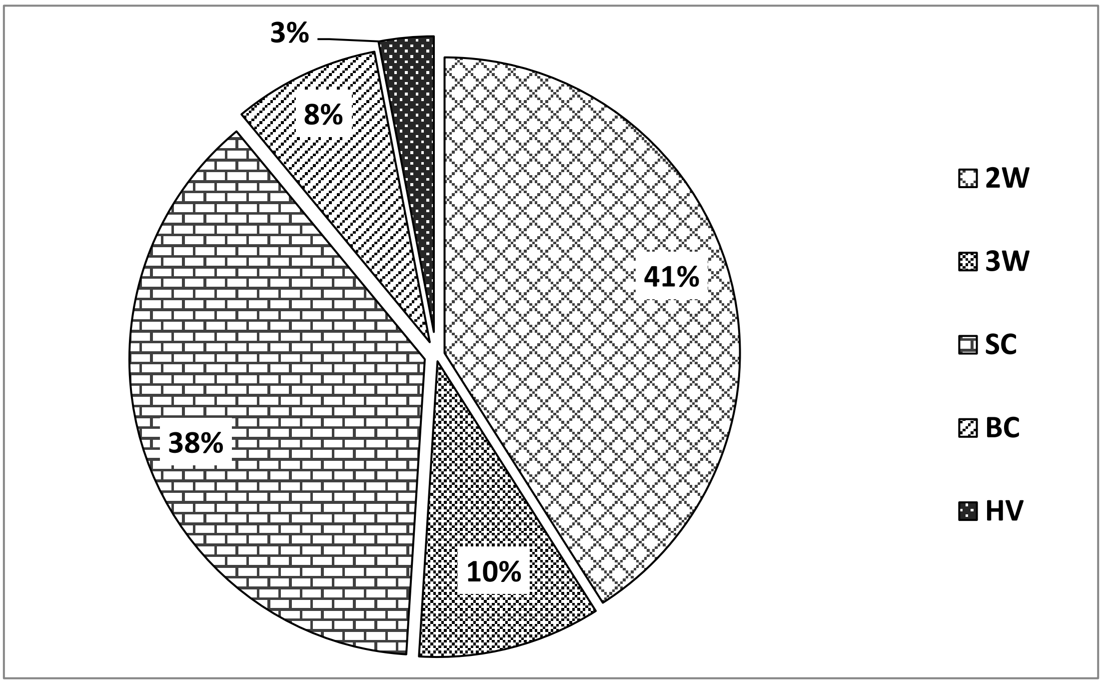


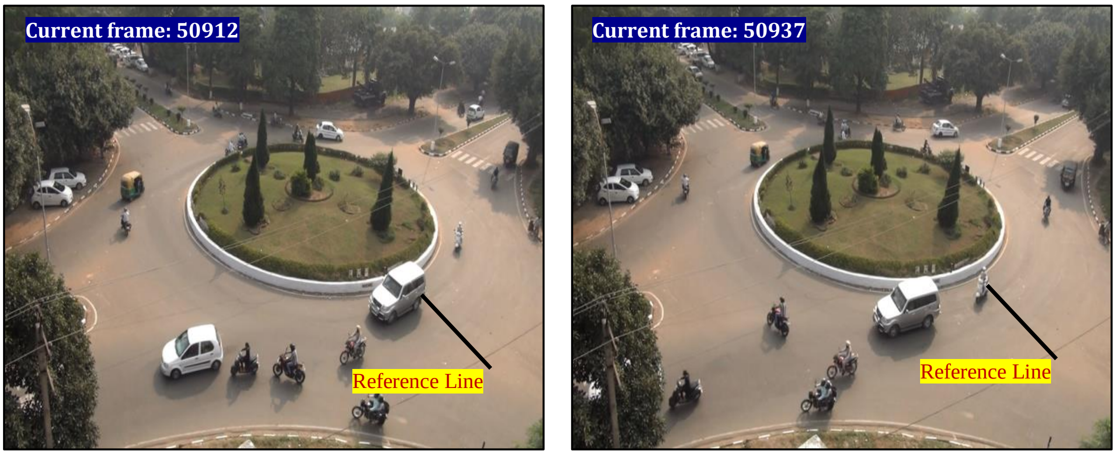
Figure 3.7 Average traffic composition of circulating traffic flow

The average traffic composition of entry flow and circulating traffic flow is almost same as shown in Figure 3.6 and Figure 3.7, respectively. The average traffic composition of entry flow is found varying as – 32% to 53% for motorized 2W, 4% to 17% for motorized 3W, 33% to 44% for small car, 2% to 12% for big cars, and 1% to

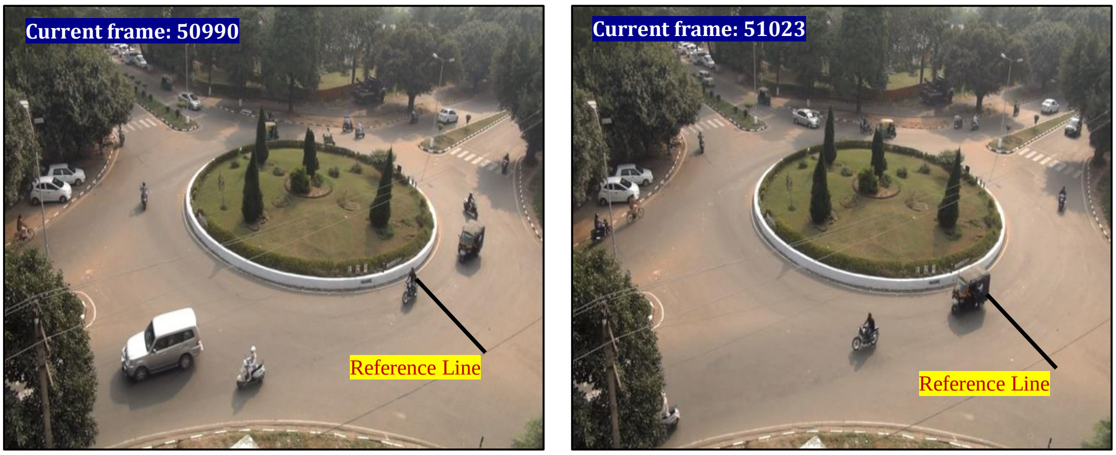
5% for HV. The average traffic composition on circulating road of a roundabout is found varying as – 36% to 51% for motorized 2W, 6% to 15% for motorized 3W, 30% to 47% for small car, 6% to 11% for big cars, and 1% to 6% for HV.

3.2.2 Lagging Headway

The recorded film was played on the computer screen to extract the data of lagging headway for following vehicle. The lagging headway is defined as a difference of times at which the rear bumper of the lead vehicle and the rear bumper of the following vehicle crosses the reference line as shown in Figure 3.8. In accordance to the concept shown in Figure 3.8, a reference line was marked on the screen covering circulating roadway. The movement of vehicles was observed with respect to this reference line.



(a) Lagging Headway for 2W



(b) Lagging Headway for 3W

(c) Lagging Headway for SC

Figure 3.8 Frame-by-frame sample data for lagging headway

The lagging headway is estimated using 25 frames per second data for all categories of vehicles in the circulating area of a roundabout. A sample sheet of lagging headway is shown in Table 3.6. Here, vehicle category means that the vehicle which is following and for which lagging headway is measured.

Table 3.6 Sample sheet for measuring lagging headway (in sec)

Vehicle category	Current frame		Difference	Lagging headway (sec)
2W	50912	50937	25	1.00
2W	50937	50990	53	2.12
3W	50990	51023	33	1.32
2W	51023	51083	60	2.40
SC	51083	51148	65	2.60
2W	51240	51280	40	1.60
SC	51290	51322	32	1.28
BC	51519	51562	43	1.72
3W	51581	51689	107	4.28
HV	51796	51890	94	3.76
2W	51930	51951	21	0.84

SC	51951	52076	125	5.00
2W	52298	52314	16	0.64
SC	52314	52336	22	0.88
SC	52336	52417	81	3.24
3W	52989	53048	59	2.36

Based on the above-mentioned approach, the lagging headway data was extracted for different vehicles at different roundabouts. The size of the data has been given in Table 3.7.

Table 3.7 Sample size for different type of vehicles on selected roundabouts

Roundabout ID	**2W**	**3W**	**SC**	**BC**	**HV**	**Total**
$\mathbf{R_1}$	418	165	674	151	53	1461
$\mathbf{R_2}$	657	150	969	182	90	2048
$\mathbf{R_3}$	358	59	474	126	57	1074
$\mathbf{R_4}$	474	133	1179	253	80	2119
$\mathbf{R_5}$	384	68	530	128	46	1156
$\mathbf{R_6}$	314	110	641	123	53	1241
$\mathbf{R_7}$	567	171	679	157	88	1662
$\mathbf{R_8}$	286	79	632	154	65	1216
$\mathbf{R_9}$	392	106	514	90	77	1179
$\mathbf{R_{10}}$	762	112	1272	180	82	2408
$\mathbf{R_{11}}$	954	120	1147	225	102	2548

3.2.3 Extraction of Gaps

Gap can be defined as the time between the rear bumper of the first vehicle and the front bumper of the second vehicle to reach the common reference point and is usually measured in seconds. While, headway is the time interval between two successive vehicles which pass a common reference point on the roadway in the same direction. It is measured from the front bumper of the first vehicle and the front bumper of the second vehicle to pass a common reference point. The difference between headway and gap is shown in Figure 3.9.

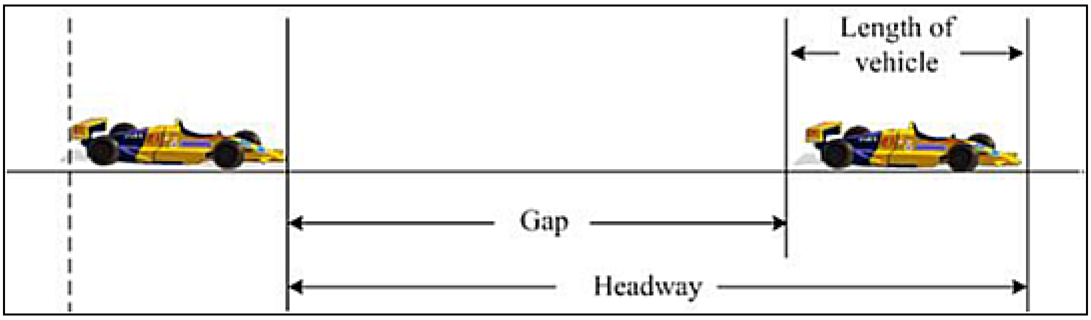


Figure 3.9 Concept of Gap and Headway

The arrival of a vehicle at a particular entry is taken as the starting point for the measurement of lags. Lag is defined as the first gap that the entering vehicle faces in circulating traffic stream. The lag is measured from the time when an approaching vehicle arrives at the entry line until the next circulating stream vehicle passes the conflict line. The conflict line is fixed on the circulating area of a roundabout where vehicles are separating to their respective directions. The gap and lag is shown in Figure 3.10. The accepted and rejected gap data by an entering vehicle is extracted for estimating the critical gap. These data is extracted for five categories of vehicles. An example set of lag and gap data is shown frame by frame in Figure 3.11.

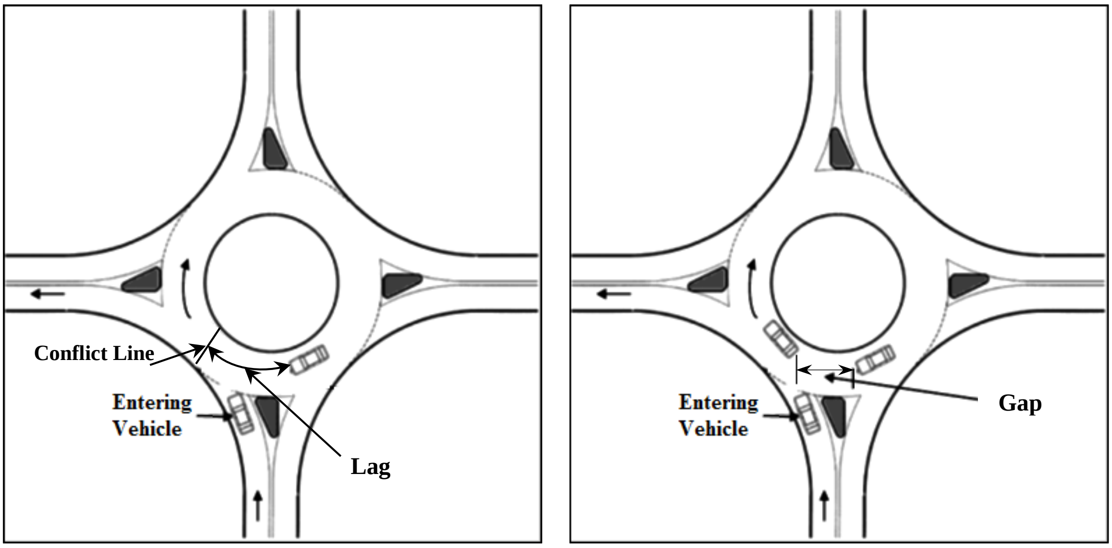
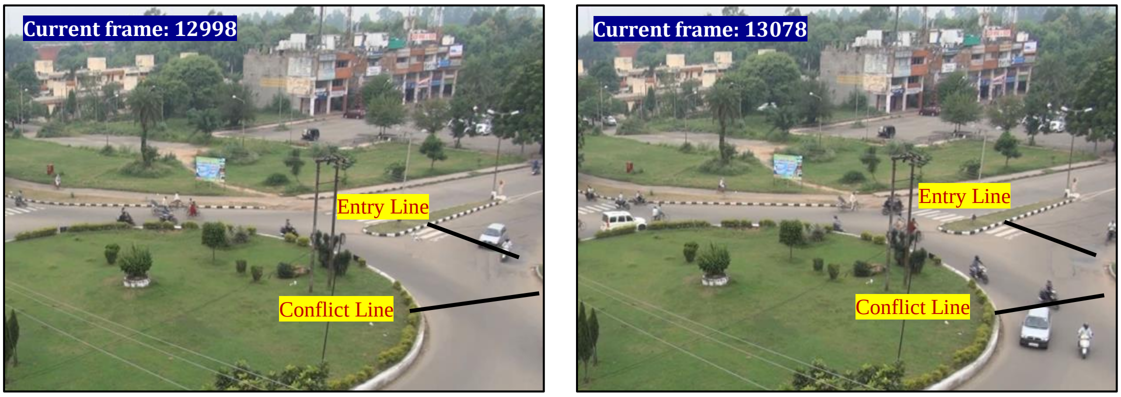


Figure 3.10 Lag and gap in the circulating stream

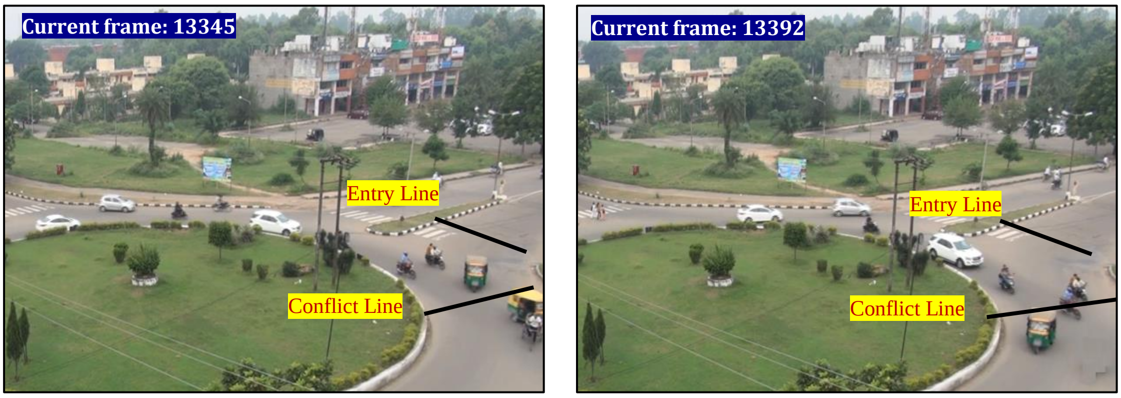


(a) Lag accepted by entering vehicle (SC)

Lag = (13088-12998)/25 = 3.20 s

(b) Gap between 2W and 3W rejected by entering vehicle (3W)

Gap 1 = (13326-13302)/25 = 0.96 s



(c) Gap between 3W and 2W accepted by entering vehicle (3W)

Gap 2 = (13392-13345)/25 = 1.88 s

Figure 3.11 Frame-by-frame sample data for lag and gap

The extracted data are recorded in MS-Excel work sheets for further processing. Table 3.8 shows the data sheet used to record the gap acceptance data of each category of vehicle. Here, vehicle category means that the vehicle which is entering from an approach of a roundabout. The lag in Table 3.8 is measured from the time when an entering vehicle arrives at the entry line until the next circulating stream vehicle passes the conflict line. If entering vehicle accepts this lag then this lag will be considered as accepted then second entering vehicle will be analyzed. If entering vehicle rejects this lag then it will be considered as rejected and next gap (Gap 1, Gap 2, Gap 3) will be recorded until entering vehicle accept the gap offered by the circulating traffic.

Table 3.8 Datasheet for recording accepted and rejected lags and gaps

Vehicle category	Lag (s)	Gap 1 (s)	Gap 2 (s)	Gap 3 (s)
SC	3.20 (A)	-	-	-
3W	0.34 (R)	0.96 (R)	1.88 (A)	-
SC	1.42 (R)	2.20 (A)	-	-
2W	1.78 (A)	-	-	-
2W	0.28 (R)	0.90 (R)	1.66 (A)	-
BC	1.24 (R)	2.54 (R)	3.58 (R)	3.88 (A)
SC	2.48 (R)	2.26 (R)	3.04 (A)	-
3W	1.98 (R)	2.14 (A)	-	-
2W	2.04 (A)	-	-	-
SC	1.28 (R)	1.00 (R)	2.58 (A)	-
2W	0.34 (R)	0.76 (R)	1.38 (R)	2.10 (A)
SC	0.60 (R)	0.68 (R)	3.38 (A)	-
BC	0.46 (R)	1.52 (R)	3.00 (A)	-
HV	0.86 (R)	0.90 (R)	2.46 (R)	3.72 (A)
SC	0.34 (R)	0.52 (R)	2.32 (A)	-

(R) = Rejected; (A) = Accepted

Based on the above-mentioned approach, the accepted and rejected lags and gaps were extracted for different categories of vehicles. The lags and gaps could be extracted only on five roundabouts, namely R_1, R_2, R_3, R_5, and R_6. It was problematic to estimate the critical gap on other roundabouts due to their exceptionally large circulating roadway width, entry width and weaving length. The gap acceptance approach is not working on these roundabouts due to propensity of merging and diverging maneuvers of entering and circulating traffic vehicles. In this situation, entering vehicles do not use gap between the circulating vehicles. However, they merge into circulating traffic stream then diverge to their respective directions. The ease of operation has made it difficult to estimate the critical gap on these roundabouts. The size of the data for lags and gaps has been given in Table 3.9.

Table 3.9 Size of data extracted for gaps and lags

Roundabout ID	Rejected Lags and Gaps					Accepted Lags and Gaps				
	2W	**3W**	**SC**	**BC**	**HV**	**2W**	**3W**	**SC**	**BC**	**HV**
$\mathbf{R_1}$	389	159	465	198	85	235	68	247	93	42
$\mathbf{R_2}$	525	264	585	238	118	297	127	274	108	48
$\mathbf{R_3}$	238	138	315	178	124	205	79	193	83	63
$\mathbf{R_5}$	245	148	362	124	116	174	62	181	59	56
$\mathbf{R_6}$	228	112	339	186	96	188	54	196	67	47

3.3 PRELIMINARY ANALYSIS

The preliminary analyses have been taken up in the following parts.

a) PCU estimation
b) Gap analysis
c) Analysis of entry flows and circulating flows

3.3.1 PCU Estimation

Passenger car unit (PCU) of different types of vehicles are required to convert a mixed traffic stream into a homogeneous equivalent, and thereby to express the mixed traffic flow in terms of equivalent number of passenger cars. Analysis of mixed traffic is often simplified by converting the different types of vehicles into equivalent number of passenger cars. As mentioned in the previous section, five vehicle categories have been considered on roundabouts, based on the field data collected. Out of these, small car is taken as the standard car in the present study and PCU factors were estimated for all other vehicles with respect to this car.

The researchers have adopted different methods for the estimation of PCU values and a wide variation exists in PCU values reported in different studies. A detailed overview of the estimation parameters and methods, as well as, the PCU values estimated by different researchers in different countries is already presented in sub-section 2.1 of previous chapter. These studies are mainly related to the single-lane roads, two-lane roads, multi-lane roads or signalized intersections. Very few studies had been done in the area of un-signalized intersection or roundabouts. These studies reveal that PCU values are specific to certain roadway and traffic conditions prevailing at a location. It is due to varying behavior of the drivers and prevailing traffic conditions in different countries. The possible traffic parameters that influence the estimation of PCU values of vehicles on roundabout are discussed in the next paragraph.

In general the parameters used are vehicle speed, vehicle area (space occupied), and headway. Number of researchers has used simulation technique to arrive at the PCU values. Some have used speed and occupancy area as base parameter, but in others it is not clear. These parameters are looked at in terms of operations on roundabouts. The speed of all entering vehicles and circulating vehicles is controlled by the geometry of the roundabout. There is a lower speed differential between the users of roundabouts since the road users travel at similar speeds through the roundabout (NCHRP Report-672, 2010). As the variation in speed is not large, it is not advisable to consider vehicular speed in the estimation of PCU values on roundabouts. Therefore, speed based method goes out of consideration. The factors like the average length and width of each vehicle type (defining area occupied) and the average gap maintained between the vehicles on circulating roadway looks appropriate. The area occupancy relatively increases, whereas, headways decrease during the congested conditions. Combining the two parameters indicate that width of the vehicle can be considered as a separate parameter, because the length of the vehicle gets included in the lagging headway estimation. In this respect, the lagging headway is estimated for vehicles on circulating roadway. The lagging headway is defined as a difference of times at which the rear bumper of the lead vehicle and the rear bumper of the following vehicle crosses the marked or reference line on the carriageway as shown in Figure 3.12. Based on these, the following equation has been proposed to determine PCU for a vehicle type '*i*'.

$$PCU_i = f_i * \frac{H_i}{H_c} \quad (3.1)$$

Where,

PCU_i = PCU of vehicle type *i*

H_i = mean lagging headway of vehicle type '*i*' in the circulating stream, seconds

H_c = mean lagging headway of standard passenger car in the circulating stream, seconds

f_i = width factor for vehicle '*i*'

$$f_i = \frac{w_i}{w_c} \quad (3.2)$$

Where,

w_i = width of the vehicle type *i*, meters

w_c = width of standard passenger car, meters

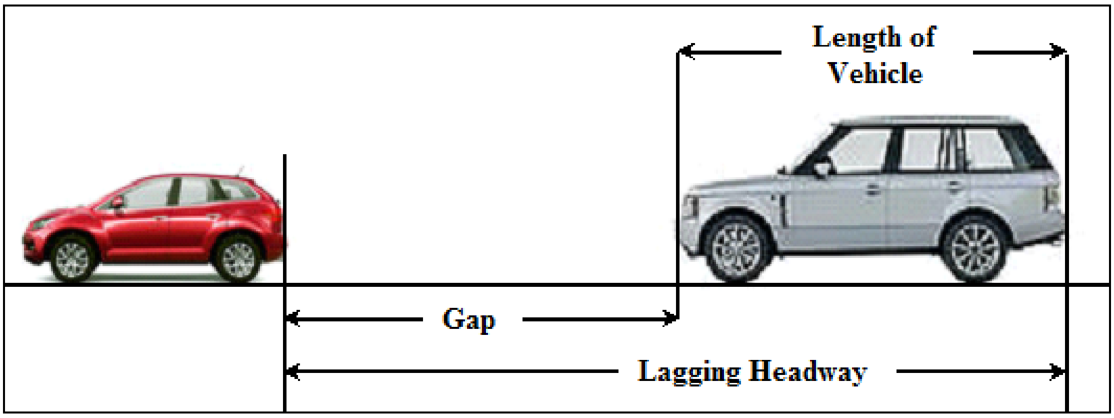


Figure 3.12 Concept of gap and lagging headway

The variation of headway values among five different categories of vehicles is shown using boxplot in Figure 3.13. In boxplot, top of the box represents the 75th percentile, bottom of the box represents the 25th percentile, and the line in the middle represents the 50th percentile or median. The whiskers (the lines that extend out of the top and bottom of the box) represent the maximum and minimum. An outlier lies on more than the maximum value and less than the minimum value. These outliers can be seen in Figure 3.13 and have been taken off from the database for further analysis.

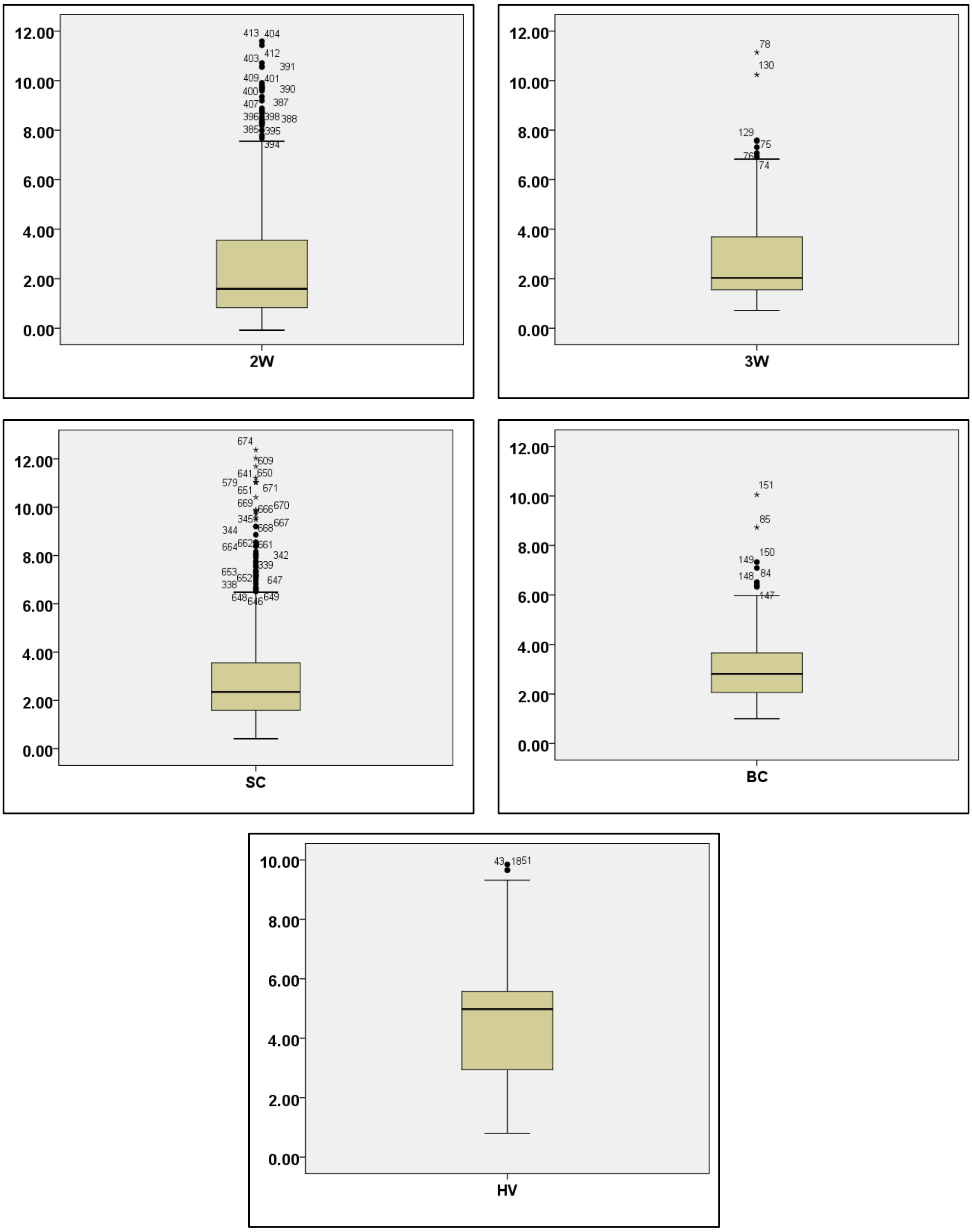


Figure 3.13 Boxplot chart of headway values for different category of vehicles

3.3.2 Gap Analysis

In a mixed traffic situation, vehicles do not respect yield sign and enter the circulating flow without waiting for a suitable gap. Small size vehicles, like two-wheelers, sometimes force their entry into the circulating flow and thus cases of rejected gaps become very few. Wide variation in the acceptance and rejection of gaps by variety of vehicles makes the analysis complex. A single and representative gap value is needed for the analysis. Therefore, a critical gap needs to be estimated. For this the gap acceptance behavior shall be examined when there is no forceful entry to the roundabout. The literature has highlighted that the maximum likelihood method (MLM) is generally found to be the most acceptable method. But all such studies are reported from urban environment of western and developed worlds. It was therefore decided to estimate critical gap using methods highlighted in the literature. This will allow to examine their suitability to urban traffic condition in India. But all of these methods are based on certain assumptions. It is reported that MLM method utilizes the data in pairs of highest rejected gap and the next accepted gap. If there is no rejection for a particular vehicle, as in the case of limited priority condition, the maximum rejected gap would be zero and log natural of zero would be undefined. Keeping such constrained traffic condition under consideration it was felt that an estimation technique for critical gap needs to be developed, which can deal with such types of traffic situations. A method based on minimizing the sum of absolute difference in a gap with respect to the accepted and rejected gaps has been used. The iterative procedure provides a value of gap that is termed as the critical gap under mixed traffic conditions.

The procedures for the estimation of critical gap by existing methods are given below.

a) Ashworth Method

Spreadsheet for estimating the critical gap by using Ashworth method is given in Table 3.10.

Table 3.10 Spreadsheet for estimating the critical gap by Ashworth method

	A	**B**
1		**Accepted Gaps (A_i)**
2		2.08
3		4.56
4		2.08
5		2.22
6		3.54
…		…
…		…
…		…
123		3.94
124		3.20
125		2.62
126		3.04
127		3.76
128		2.92
129	**Mean (μ_a)**	**4.18**
130	**Standard Deviation (σ_a)**	**1.82**
131	**Circulating Traffic (vps)**	**0.7**
132	**Critical Gap ($t_c = \mu_a - p * \sigma_a^2$)**	**1.86**

b) Harders Method

Spreadsheet for estimating the critical gap by using Harders method is given in Table 3.11.

Table 3.11 Spreadsheet for estimating the critical gap by Harders method

Gap Size	Gap Accepted (A_i)	Total Gaps Offered (N_i)	t_i(s)	$R_i = A_i/N_i$	Probability $= R_i - R_{i-1}$	t_i*Probability
0.00-0.50	0	37	0.25	0.000	0.000	0.000
0.51-1.00	2	43	0.75	0.047	0.047	0.035
1.01-1.50	28	103	1.25	0.272	0.225	0.282
1.51-2.00	36	86	1.75	0.419	0.147	0.257
2.01-2.50	37	69	2.25	0.536	0.118	0.265
2.51-3.00	38	47	2.75	0.809	0.272	0.749
3.01-3.50	38	45	3.25	0.844	0.036	0.117
3.51-4.00	21	23	3.75	0.913	0.069	0.257
4.01-4.50	16	17	4.25	0.941	0.028	0.120
4.51-5.00	17	17	4.75	1.000	0.059	0.279
5.01-5.50	8	8	5.25	1.000	0.000	0.000
5.51-6.00	5	5	5.75	1.000	0.000	0.000
6.01-6.50	3	3	6.25	1.000	0.000	0.000
6.51-7.00	2	2	6.75	1.000	0.000	0.000
7.01-7.50	1	1	7.25	1.000	0.000	0.000
7.51-8.00	1	1	7.75	1.000	0.000	0.000
$t_c = \sum t_i * (\text{Probability})$						**2.360**

c) Maximum Likelihood Method

Spreadsheet for estimating the critical gap by using maximum likelihood method is given in Table 3.12. The mean and standard deviation of all the gaps are in cell D91 and D92, respectively. The values in the column D are calculated using the formula:

LN(NORM.DIST(LN(B2),D$91,D$92,TRUE)-NORM.DIST(LN(C2),D$91,D$92,TRUE))

For estimation of the critical gap, the sum of the column D in cell D90 was -35.29. Using solver function in MS EXCEL, the mean and standard deviation converged to 0.887 and 0.256 respectively and the sum (logarithm of the likelihood) was increased to -1.58.

Table 3.12 Spreadsheet for estimating the critical gap by MLM

A	**B**	**C**	**D**
1	**Max Rejected Gap (r)**	**Accepted Gap (a)**	**Ln(F(a)-F(r))**
2	0.90	9.14	0.000
3	1.60	3.80	-0.096
4	1.58	5.48	-0.049
5	1.98	5.14	-0.242
6	0.94	5.46	-0.001
…	…	…	
…	…	…	
86	0.98	4.34	-0.012
87	1.94	4.68	-0.218
88	1.70	4.58	-0.093
89	1.20	2.94	-0.262
90	**Sum**		**-1.58**
91	**Mean (μ)**		**0.887**
92	**Standard Deviation (σ)**		**0.256**
93	**Critical Gap,** $t_c = e^{\mu+0.5\sigma^2}$		**2.509**

d) Wu Method

Table 3.13 presents an example of the procedure for estimating the critical gap by Wu method with a spreadsheet.

Table 3.13 Spreadsheet for estimating the critical gap by Wu method

(0)	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
(0)	(1)	(2)	if (2)="r", $n_r = n_r+1$	if (2)="a", $n_a = n_a+1$	$\frac{(3)}{n_{r,max}}$	$\frac{(4)}{n_{a,max}}$	$\frac{(6)}{(6)+1-(5)}$	$(7)_i-(7)_{i-1}$	$\frac{(1)_i+(1)_{i-1}}{2}$	$(8)*(9)$
S.N. (i)	Gaps (t_i)	Acc. or Rej.	n_r	n_a	F_r	F_a	$F_{tc} = \frac{F_a}{F_a+1-F_r}$	$P_{tc_i} = F_{tc_i}-F_{tc_{i-1}}$	$td_j = \frac{t_i+t_{i-1}}{2}$	$P_{tc_i}*td_i$
1	0.50	r	1	0	0.019	0.019	0.019	0.000	0.250	0.000
2	0.64	r	2	0	0.037	0.019	0.019	0.000	0.567	0.000
3	0.74	r	3	0	0.056	0.019	0.019	0.000	0.683	0.000
4	0.78	a	3	1	0.056	0.037	0.038	0.019	0.750	0.014
5	0.78	r	4	1	0.074	0.037	0.038	0.001	0.767	0.001
6	0.84	r	5	1	0.093	0.037	0.039	0.001	0.800	0.001
…	…	…	…	…	…	…	…	…	…	…
…	…	…	…	…	…	…	…	…	…	…
…	…	…	…	…	…	…	…	…	…	…
104	4.38	a	52	52	0.963	0.963	0.963	0.001	4.300	0.003
105	4.64	a	52	53	0.963	0.981	0.964	0.001	4.517	0.003
106	4.68	a	52	54	0.963	1.000	0.964	0.001	4.667	0.003
107	4.68	r	53	54	0.981	1.000	0.982	0.018	4.667	0.082
108	6.70	r	54	54	1.000	1.000	1.000	0.018	5.683	0.103
$t_c = \sum P_{tc_i} * td_i$										**2.340**

e) Modified Raff Method

Spreadsheet for estimating the critical gap by Modified Raff's method is given in Table 3.14. This method describes the critical gap as the intersection of two cumulative curves F'_a and F'_r as shown in Figure 3.14.

Table 3.14 Spreadsheet for estimating the critical gap by Modified Raff's method

Gap Size	Accepted	Cumulative Accepted (F_a)	Cumulative % Accepted (F'_a)	Rejected	Cumulative Rejected (F_r)	Cumulative % Rejected (F'_r)
0.00-0.50	0	0	0.00	1	20	100
0.51-1.00	0	0	0.00	2	19	95
1.01-1.50	0	0	0.00	11	17	85
1.51-2.00	1	1	1.10	3	6	30
2.01-2.50	9	10	10.99	2	3	15
2.51-3.00	13	23	25.27	1	1	5
3.01-3.50	12	35	38.46	0	0	0
3.51-4.00	10	45	49.45	0	0	0
4.01-4.50	9	54	59.34	0	0	0
…	…	…	…	…	…	…
…	…	…	…	…	…	…
…	…	…	…	…	…	…
12.50-13.00	0	88	96.70	0	0	0
13.01-13.50	0	88	96.70	0	0	0
13.51-14.00	2	90	98.90	0	0	0
14.01-14.50	1	91	100.00	0	0	0
Total	**91**			**20**		

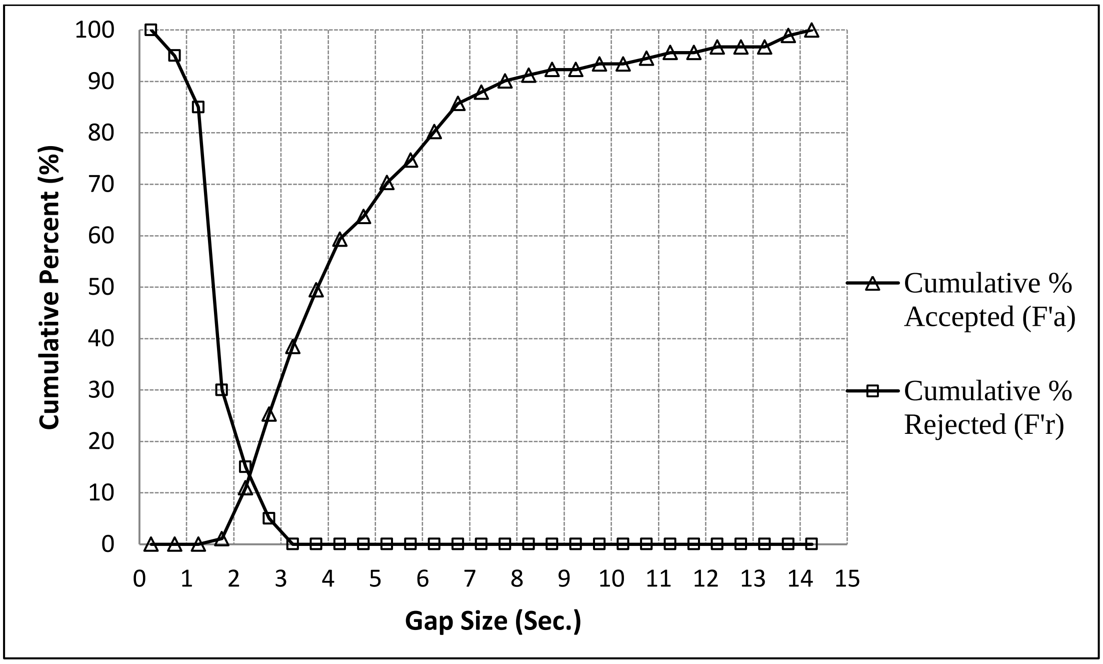


Figure 3.14 Critical gap estimated by Modified Raff's method

3.3.3 Analysis of Entry Flows and Circulating Flows

The entry traffic flow from an approach and circulating traffic flow are extracted during the period of stable queue. The process of their measurement has already been discussed in previous section. The scatter plots have been plotted between the entry flow and circulating flow so as to examine their relationship. The linear, exponential, logarithmic, polynomial and power functions have been used to ascertain the best relation between entry flow and circulating flow. The statistical parameters such as: *Significance F* and coefficients of determination have been used to find the best fit of models between entry flow and circulating flow.

3.4 DETAILED ANALYSIS

As defined in the flowchart of the methodology, the data analyses would be taken up in the following five parts.

a) PCU standardization and H-factor estimation

b) Estimation of critical gap and follow-up time

c) Calibrating HCM (2010) model

d) Development of entry capacity model

e) Sensitivity analysis for variables

These are briefly discussed here.

3.4.1 PCU Standardization and H-Factor Estimation

PCU for a vehicle on a roundabout has been given for three category of vehicles with respect to the car in IRC:65-1976. These categories are motorized two-wheelers, motorized three-wheelers and heavy vehicles. The suggestive PCU values are 0.75, 1.0 and 2.8 respectively for the above-mentioned three categories of vehicles. Since 1976, the vehicle technology has changed manifolds. The vehicles have become operationally more efficient, safe and maneuverable. The driver behavior and traffic patterns or conditions of roads have also changed over the years. All these variables have influence on the PCU values of the vehicles. There certainly arises the need to estimate the PCU values for these new technology vehicles under different categories plying on roundabouts in developing countries. As enumerated before, the PCU values have been estimated with respect to the small car. With recent technological developments, a range of vehicles are available even within car category. As mentioned before, cars have been considered in two categories, namely small cars and big cars. The passenger car units for five categories of vehicles have been estimated using lagging headway and width of the vehicle. The values have been examined with respect to the roundabouts of varying geometric features to standardize them. Statistical examination of the values has been carried out. Based on these, final values are suggested. The effect of traffic flow and geometric parameters of the roundabout on estimated PCU values have been examined for different categories of vehicles.

Once the PCU values were standardized based on new information available, it was thought to extend the work to estimate heterogeneity equivalency factor (H-factor). It is the ratio of traffic volume in PCU/h and volume in veh/h. This can be used for converting the heterogeneous traffic (veh/h) into homogeneous traffic (pcu/h). This is defined as a factor which when multiplied with the traffic flow, measured in veh/h, will give directly the flow in pcu/h. Therefore, the need to estimate traffic flow using PCU factor get omitted, till these factors get changed under changed traffic condition in future.

3.4.2 Estimation of Critical Gap and Follow-up Time

A driver who is waiting for a gap may accept a gap after rejecting some gaps. The size of the accepted gaps and the rejected gaps for different drivers do not provide information about what the smallest gap they would accept or the biggest gap they would reject. Therefore it is necessary to calculate critical gap for different types of vehicles entering the circulating traffic at a roundabout.

Accepted and rejected gaps have been extracted to estimate the critical gap for five categories of vehicles plying on a roundabout. Various methods are available in literature to estimate critical gap, such as: Harders, Ashworth, Modified Raff, MLM and Wu methods. The suitability of these methods to mixed traffic conditions is judged by the criterion of explaining the percent violators. There are two sets of data, one rejected gaps and the other accepted gaps. The critical gap value should be greater than all the rejected gaps and smaller than all accepted gaps. Some values of rejected gaps and accepted gaps would not satisfy this criterion because of inconsistent drivers and these are termed as violations. The outcome of methods will also be examined with the outcome of the proposed method of estimation of critical gap. The variation in critical gap value with other influencing variables will also be taken-up.

Follow-up times represent the process by which multiple vehicles that are queued at an approach can enter the roundabout using the same gap in the circulating flow. The follow-up time is the time between the departure of one vehicle from the entry approach and the departure of the next vehicle using the same gap under a condition of continuous queuing. The follow-up time has been measured for different types of vehicles on selected roundabouts.

3.4.3 Calibrating HCM (2010) Model

The HCM (2010) suggests use of the equation (3.3) to estimate entry capacity (C_e) at roundabouts.

$$C_e = A * e^{-B*V_c} \tag{3.3}$$

Where,

V_c = circulating traffic flow (pcu/h)

Parameters A and B, and the corresponding gap-acceptance parameters (t_f, t_c) are given in Table 3.15.

Table 3.15 HCM (2010) roundabout parameters (A, B, t_f and t_c)

	A	**B**	**t_f**	**t_c**
Single lane circulating stream (n_c=1)				
Single lane entry (n_e=1, n_c =1)	1130	0.00100	3.19	5.19
Multilane entry (n_e > 1, n_c=1); apply to all entries	1130	0.00100	3.19	5.19
Multilane circulating stream (n_c>1)				
Single lane entry (n_e=1, n_c>1)	1130	0.00070	3.19	4.11
Multilane entry (n_e>1, n_c>1)				
Right lane entry (US driving condition)	1130	0.00070	3.19	4.11
Left lane entry (US driving condition)	1130	0.00075	3.19	4.29

Parameters A and B can also be calculated using equation (3.4) and (3.5) as given below.

$$A = \frac{3600}{t_f} \tag{3.4}$$

$$B = \frac{t_c - 0.5 * t_f}{3600} \tag{3.5}$$

Where,

t_f = follow-up time (s)

t_c = critical gap (s)

The original equation as given in HCM 2010 for the estimation of entry capacity cannot be used directly in the traffic flow conditions prevailing in developing countries. Two factors are identified behind this: one, traffic flow heterogeneity in developing countries as compared to the US and European countries, and second, the driver behavior which is governed by lane discipline in the developed countries but is governed by the flow characteristics and opportunities arising for merging from approaches in the developing countries. Therefore, HCM (2010) equation for estimating the entry capacity has to be modified for its adaptation to heterogeneous traffic conditions. Parameters A and B of the HCM equation have been calibrated based on critical gap and follow-up time obtained from the field data.

3.4.4 Entry Capacity Model

Over the years, researchers have developed an entry capacity estimation model for roundabouts. The entry capacity is defined as the maximum number of vehicles that can enter the roundabout in a given period of time, with a given amount of circulating traffic volume and given width of entry as well as circulating area. The physical parameters of the roundabout had been incorporated in entry capacity model as already reported in many studies like number of circulating lanes and entry lanes, the inscribed diameter, the diameter of central-island, distance between the entry and near-side exit, circulating roadway width, entry width, width of entry island, approach width, etc. The circulating traffic flow, circulating roadway width, entry width, number of circulating lanes and entry lanes are used in most of the existing entry capacity models. At the same time there are some models which purely use traffic flow data only.

Regression analysis has been used to develop a general entry capacity model for Indian traffic flow conditions. Physical parameters of roundabout such as central-island diameter, entry width, exit width and circulating roadway width have been analyzed for developing the entry capacity formula. Based on the statistical characteristics of the developed model, the statistically significant parameters at 95 percent level of confidence have been included in the entry capacity model. Then, the developed entry capacity model has been compared with the existing regression models used in developing countries.

3.4.5 Sensitivity Analysis of Model

The sensitivity analysis has been done to examine the effect of influencing variables on the entry capacity. The entry capacity of an approach has been influenced by physical parameters of a roundabout. It has been decided based on the sensitivity analysis, which physical parameter has highly influenced the entry capacity of an approach.

3.5 CONCLUSIONS AND RECOMMENDATIONS

The significant conclusions will be drawn on chapter-8 based on the analysis of passenger car unit, critical gap and follow-up time, analysis of entry flow, calibrating HCM (2010) model, and entry capacity model. The significant contributions of the research work, limitations of the work and scope for the future work will also be defined on chapter-8.

CHAPTER 4:
PCU AND H-FACTOR

4.0 GENERAL

Sufficient literature is available on approaches to deal with heterogeneous traffic on mid-blocks in developing countries, but not much similar work is reported on roundabouts. The use of passenger car unit (PCU) or passenger car equivalent (PCE) for different category of vehicles to convert heterogeneous traffic into homogeneous one is a well-accepted and documented procedure. The parameters used for the estimation of PCU or PCE factors at mid-blocks may or may not be influencing in similar manner on roundabouts. This may be due to difference in the traffic flow characteristics at these two locations. A look on the suggested PCU values on roundabouts from developing countries indicates that these are not recent, and needs a relook. The literature also reports the use of static or dynamic PCU values. Many researchers have advocated the use of dynamic PCU values on account of possible temporal and spatial variations at / across locations. This chapter presents the estimated PCU values for different category of vehicles plying on roundabouts in developing countries like India. It also statistically examines the possible influence of geometric and flow variables on the estimated PCU values for different category of vehicles on different roundabouts. Before suggesting PCU values for a category of vehicles plying on different size of roundabouts, a check is made to ascertain whether or not the roundabouts are statistically different (operationally). A concept of Heterogeneity Equivalency Factor (H-Factor) is proposed to avoid the re-estimation of PCU values on different locations, due to possible traffic flow variations. The factor is multiplicative and converts heterogeneous traffic (veh/h) into homogeneous traffic (pcu/h), without a need to estimate PCU values under given traffic flow conditions. The variation in H-factor due to proportion of different category of vehicles in a traffic flow and with variations in circulating traffic is also presented. Statistical examination, wherever felt needed, is also done and presented.

4.1 PASSENGER CAR UNIT

Traffic in India is heterogeneous in nature wherein varieties of vehicles, having different static and dynamic characteristics, occupy same space on mid-blocks and at intersections. The width of these vehicles varies from around 0.60 m (bicycle) to 2.50 m (bus and truck). Their operational performance, say vehicle speed, too varies from as low as 12 km/h to 60 km/h or more. Consequently, one type of vehicle cannot be considered similar in space occupied on carriageway and operational performance to other types of vehicles. To account for the non-uniformity or heterogeneity in traffic stream as well as to make them comparable across locations, the measurement unit for vehicles needs to be converted into a common unit. With this purpose, passenger car unit or PCU is used widely. PCU for heavy vehicle can be defined as "the number of passenger cars that will result in the same operational conditions as a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions" (HCM 2010). PCU is also discussed by some researchers as passenger car equivalent (PCE).

These PCUs or PCEs have been estimated as static as well as dynamic value. Static PCU values are based on the relative size of the vehicles and some flow parameter. These remain constant for varying composition of the traffic flow and road geometry. Static PCU values for a vehicle have been determined by many researchers in developed and developing countries. Results of few of such studies on roundabouts are given in Table 4.1. As can be seen, the PCU for a vehicle is different in different countries. It might be due to differences in operational traffic conditions and variation in the driver behavior. Traffic characteristics and operations at the roundabouts of developing countries are significantly different from those at the similar roundabouts in developed countries. The population and vehicle ownership pressures in developing countries just do not allow the drivers to follow lane based driving behavior, thus resulting in a higher breed of impatient or aggressive drivers. Under the scenario where they are expected to do merging and diverging maneuvers (like on a roundabout), the drivers accept low gaps between the vehicles in the circulating roadway in a bid to reduce the waiting time. It can also be noted from the data that the static PCU values, used in India, for roundabouts are based on old traffic studies being conducted prior to 1976. Values for other countries are from 90s or 2000. Since 1976, the vehicle technology has changed tremendously. They have become operationally (kinematic, turning and speed) more efficient and maneuverable. The car category has evolved over the years and need to be categorized based on either their sizes or engine power as done in Sri Lanka. Sri Lankan values have resemblance to the traffic flow condition observed in India. Values used in U.S. are only for two categories of vehicles in a traffic stream. This is relatively more homogeneous than that in India. Looking at the data from other developing countries it was felt necessary *to estimate the PCU values for different categories of vehicles on roundabouts in Indian traffic flow condition and to examine the same in the light of changed traffic conditions and vehicle operations over the years and vis-à-vis other countries having similar traffic operation conditions.*

Table 4.1 Existing PCU values for vehicles on roundabouts

Country Name	**2W**	**3W**	**SC**	**BC**	**HV**	**Source**
India	0.75	1	1	-	2.8	(IRC-65 1976)
Indonesia	0.50	-	1	-	1.3-2.0	(IHCM 1993)
Sri Lanka	0.70	0.9	1	1.2	2.2	(Kumarage 1996)
Jordan	0.50	-	1	-	2.0	(Al-Masaeid and Faddah 1997)
U.S.	-	-	1	-	2.0	(HCM 2010)
Malaysia	0.75	-	1	-	2.8	(Pakshir et al. 2012)

Note: 2W = motorized two-wheeler; 3W = motorized three-wheeler; SC = small car; BC = big car; HV = heavy vehicle.

Next question is whether to use static or dynamic PCU values. Joshi and Vagadia (2013) had reported that the research efforts in last decade have been towards the estimation of dynamic PCU values for various vehicles. Dynamic PCU values have been estimated based on attributes namely traffic flow characteristics (macro or micro), vehicle physical characteristics like width or length or area and, volume and composition of traffic flow (Sharma et al. 2014). One point propagated in favour of the dynamic estimation of PCUs is that it considers the effect of varying behavior of traffic in temporal frame. But it makes the estimation as well as implementation difficult as the traffic scenario at a time will be different across the locations. It also makes the PCU values varying spatially. Keeping an account of temporal and spatial variations in PCU values is not an easy task. Keeping the above pros and cons of the two types of PCU values, *it is decided to estimate PCU values for vehicle categories on roundabouts and to examine the effect of traffic volume in circulating roadway or diameter of central-island or width of the circulating roadway, etc. on the PCU values, if any. It was also decided to statistically examine the operation ability of roundabouts, for any differences. This would give reply to the question whether or not to use static or dynamic PCU values.*

4.2 ASSESSMENT OF PCU VALUES

An effort has been made to estimate PCU values for different categories of vehicles at roundabouts. As already mentioned, the factors like the average length and width of each vehicle type and the average gap between the vehicles while circulating around the central island are considered in the estimation of PCU value for different vehicle categories. The lagging headway by each vehicle type was measured by the time displayed on the computer screen. The procedure adopted in the measurement is already discussed in the research methodology chapter at appropriate location. The average lagging headway for different type of vehicles and roundabouts are given in Table 4.2.

Table 4.2 Average lagging headway for different type of vehicles

Roundabout ID	**Lagging Headway (Seconds)**				
	2W	**3W**	**SC**	**BC**	**HV**
R_1	2.31	2.70	2.85	2.97	4.64
R_2	1.96	2.72	2.68	3.12	4.94
R_3	2.78	3.52	3.19	3.36	4.86
R_4	2.32	3.04	2.94	3.32	5.09
R_5	3.10	3.70	3.78	4.24	6.26
R_6	1.72	2.54	2.58	2.81	4.25
R_7	1.79	2.40	2.08	2.28	3.63

R_8	1.67	2.43	2.35	2.70	4.69
R_9	1.70	2.60	2.57	2.98	5.11
R_{10}	1.53	2.00	1.96	2.14	3.15
R_{11}	1.69	2.24	2.10	2.24	3.40
Average	2.05	2.72	2.64	2.92	4.55

The overall average lagging headway of 2W is 2.05s which is less than the average lagging headway of SC i.e. 2.64s. Similarly, the overall average lagging headway of 3W is 2.72s which is greater than that of SC. This is probably impacted by the relative length of the vehicles and acceleration and deceleration characteristics of respective vehicles as given in Table 4.3. The size of 3W is not too different than SC but their kinematic characteristics are lower than those of SC. The overall average lagging headway of BC is 2.92s, which is probably attributed to their size (width especially) and higher safety margins adopted to keep the vehicle safe. The kinematic characteristics of small and big car remain more or less similar under controlled path system (circulating roadway). In case of HV, the overall average lagging headway is 4.55s. This is impacted more by the larger dimensions of HV and lower kinematic characteristics, making the drivers cautious while accepting gaps, as well as, while maneuvering (merging ⁄ diverging).

Table 4.3 Acceleration and deceleration rates of vehicles

Vehicle Type	**Acceleration (m/s^2)**		**Deceleration (m/s^2)**	
	Maximum	**Desired**	**Maximum**	**Desired**
2W	2.2	1.6	1.5	0.5
3W	1.1	0.9	1.1	0.8
SC	2.7	1.3	1.8	1.2
BC	2.7	1.3	1.8	1.2
HV	2.5	1.5	1.7	1.2

Source: Mehar et al. (2014)

The IRC-65 (1976) considers cars as single category. In the present study it has been divided into two categories i.e. SC and BC. Z-test analysis has been carried out to see whether there is any similarity between the lagging headway of SC and BC. The null hypothesis (H_0) was: *There is no significant difference between the lagging headway of SC and BC.* The lagging headway of SC and BC are normally distributed (Refer Appendix A). The result of z-test analysis is given in Table 4.4. It shows that the p-value < 0.01, and z > 2.576. Therefore, the null hypothesis is rejected at the 99% level of confidence and there is significant difference between the lagging headway of SC and BC. The lagging headway of BC is higher than SC.

Table 4.4 Z-test: comparison of lagging headway of SC and BC

	Lagging Headway (s)	
	SC	**BC**
Mean	2.64	2.92
Variance	0.3367	0.4727
Observations	144	
Hypothesized Mean Difference	0	
z	3.662	
P(Z<=z) two-tail	0.000	
z Critical two-tail	2.576	

Similarly, z-test analysis has been carried out to see whether there is any similarity between the lagging headway of 3W and SC. The null hypothesis (H_0) was: *There is no significant difference between the lagging headway of 3W and SC.* The lagging headway of 3W and SC are normally distributed (Refer Appendix A). The result of z-test analysis is given in Table 4.5. It shows that the p-value > 0.01, and z < 2.576. Therefore, the null hypothesis is not rejected at the 99% level of confidence and there is no significant difference between the lagging headway of 3W and SC.

Table 4.5 Z-test: comparison of lagging headway of 3W and SC

	Lagging Headway (s)	
	3W	**SC**
Mean	2.72	2.64
Variance	0.3155	0. 3367
Observations	144	

Hypothesized Mean Difference	0
z	1.169
P(Z<=z) two-tail	0.243
z Critical two-tail	2.576

The average estimated PCU values for different type of vehicles relating to roundabout ID are given in Table 4.6.

Table 4.6 Average estimated PCU values for different type of vehicles

Roundabout ID	**2W**	**3W**	**SC**	**BC**	**HV**
R_1	0.36	0.92	1.00	1.28	2.75
R_2	0.32	0.99	1.00	1.43	3.11
R_3	0.39	1.07	1.00	1.29	2.57
R_4	0.35	1.01	1.00	1.39	2.92
R_5	0.36	0.95	1.00	1.38	2.80
R_6	0.30	0.96	1.00	1.34	2.78
R_7	0.38	1.12	1.00	1.35	2.95
R_8	0.32	1.01	1.00	1.41	3.37
R_9	0.29	0.98	1.00	1.42	3.35
R_{10}	0.35	0.99	1.00	1.34	2.72
R_{11}	0.36	1.03	1.00	1.31	2.73
Average	0.34	1.00	1.00	1.36	2.91

Statistical mean, percentiles and skewness of PCU values for different categories of vehicles are given in Table 4.7. Mean PCU values for different categories of vehicles are almost same as median PCU values. The median is a measure of central tendency. The mean is only representative if the distribution of the data is symmetric, otherwise it may be heavily influenced by outlying measurements. Based on the skewness value as given in Table 4.7, it can be said that though the distribution of PCU factor is skewed towards right of the mean but is not significant. In such a condition either median or mean can be used for the homogenization of traffic data on roundabouts. Mean PCU values are finally suggested. It is 0.34 for 2W, 1 for 3W and SC, 1.36 for BC and 2.91 for HV.

Table 4.7 Statistical PCU values for different categories of vehicles on roundabouts

Statistical Values	2W	3W	SC	BC	HV
Mean	0.34	1.00	1.00	1.36	2.91
15th Percentile	0.31	0.81	1.00	1.16	2.26
25th Percentile	0.32	0.85	1.00	1.24	2.42
50th Percentile (Median)	0.36	0.97	1.00	1.37	2.85
75th Percentile	0.38	1.07	1.00	1.47	3.27
85th Percentile	0.40	1.09	1.00	1.56	3.49
Skewness	0.19	0.24	0.00	0.12	0.37

The estimated values of PCU for different vehicles are found to be quite similar to the values already presented in Table 4.1. The comparison of the suggested values with those given in IRC-65 (1976) indicates that there is no change in the PCU value for 3W. The estimated PCU value for 3W is more or less equal to one as reported in literature. It may be attributed to the lagging headway which is significantly similar to that of SC even though width of the vehicle is little bit smaller as compared to SC. Consequently, it is decided to merge motorized three-wheeler with standard car and consider them as one category, SC. The PCU value for 2W is found to be lower (almost half) than that given in the IRC-65 (1976) code. This means that in the operational space occupied by a standard car, around 3 motorized 2-wheeler may maneuver (lagging gap effect) instead of 4 (relative space occupied effect). In the case of HV, the estimated PCU value is not too different than that given in the IRC-65 (1976) code. PCU value for HV got marginally increased. The comparison with those used in different countries (Refer Table 4.1) indicates towards wider difference. It probably also indicates towards different driver behavior in India as compared to other countries. It may also be the effect of time gap in previous studies and this study during which technology and behavior have changed a lot. These need to be ascertained by conducting a separate study.

Before recommending the final PCU values for roundabouts, it was felt necessary to examine whether the selected roundabouts behave homogenously irrespective of the diameter of central-island or the width of the circulating roadway. It was observed that there is wide variation in the circulating traffic even on similar sized roundabouts. These roundabouts may be considered operationally similar if the composition of the circulating traffic on each of the selected roundabouts does not differ statistically. To examine the above hypothesis statistical checks were made on the data. Two-Way-ANOVA analysis has been used to check the homogeneity between different roundabouts (i.e. R_1 to R_{11}) based on the traffic composition. The null hypothesis (H_0) is: *There is no significant difference in the traffic compositions between roundabouts R_1 to R_{11}*. The results of two-way ANOVA analysis are given in Table 4.8. It shows that the p-value for roundabouts R_1 to R_{11} = 1.0 > 0.05 (or F = 0 < 2.1 = F_{crit}). *Therefore, the null hypothesis is not rejected at the 95% level of confidence and there is statistically no significant difference in the operational condition of roundabouts R_1 to R_{11}*. Hence, the suggested static PCU values in the preceding paragraph may be recommended for use on roundabouts in the developing countries.

Table 4.8 Results of two-way ANOVA analysis

Source of Variation	**SS**	**DOF**	**MS**	**F**	**P-value**	**F_{crit}**
Roundabouts (R_1 to R_{11})	0.0	10	0.0	0.0	1.0	2.1
Traffic Composition of Vehicles	14087.6	4	3521.9	217.3	1.5E-26	2.6
Error	648.4	40	16.2			
Total	14736.0	54				

4.2.1 PCU v/s Flow and Geometric Variables

Further, the effect of traffic flow and geometric parameters of the roundabout is examined on the estimated PCU values for different category of vehicles. The plots of variation in PCU values with different variables are shown in Figure 4.1 to Figure 4.5. The equations of PCU with variables are embedded in these figures. The statistical parameters (coefficients of determination and p-value) of the related equations are given in Table 4.9.

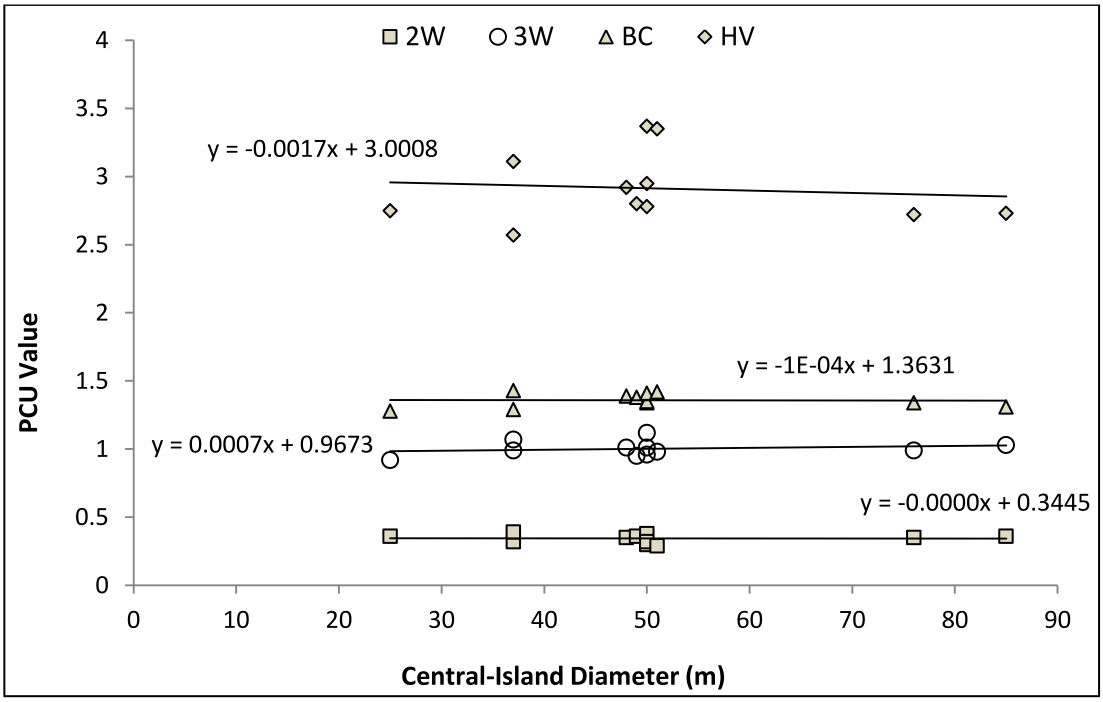


Figure 4.1 Variation in PCU values with central-island diameter

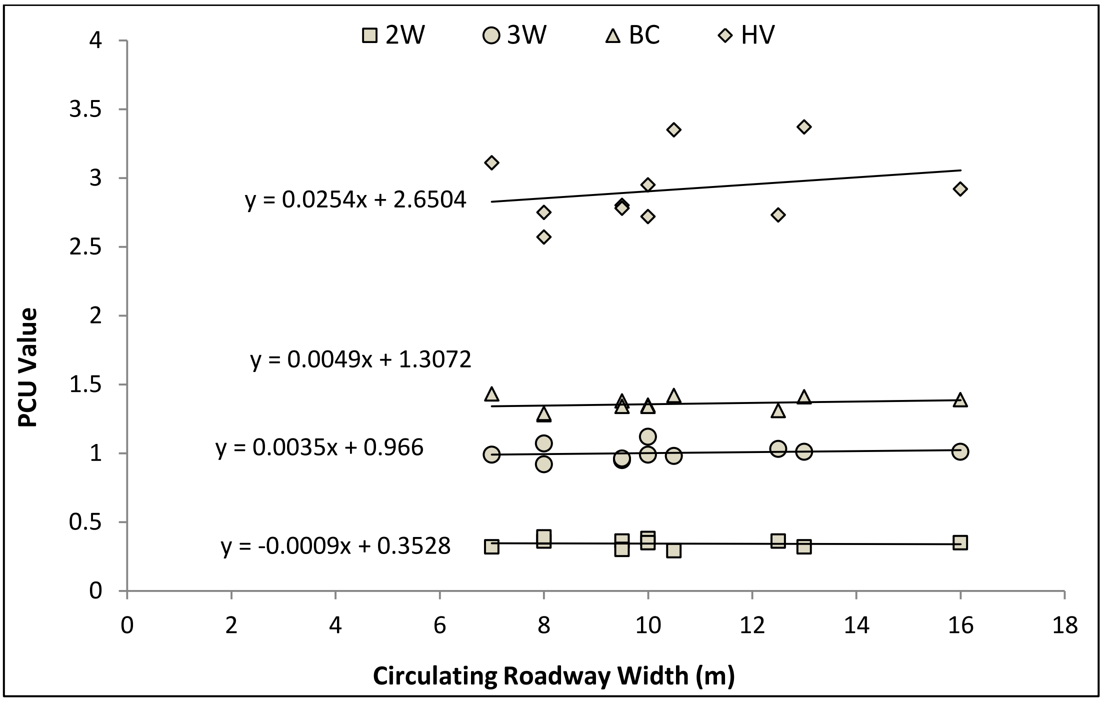


Figure 4.2 Variation in PCU values with circulating roadway width

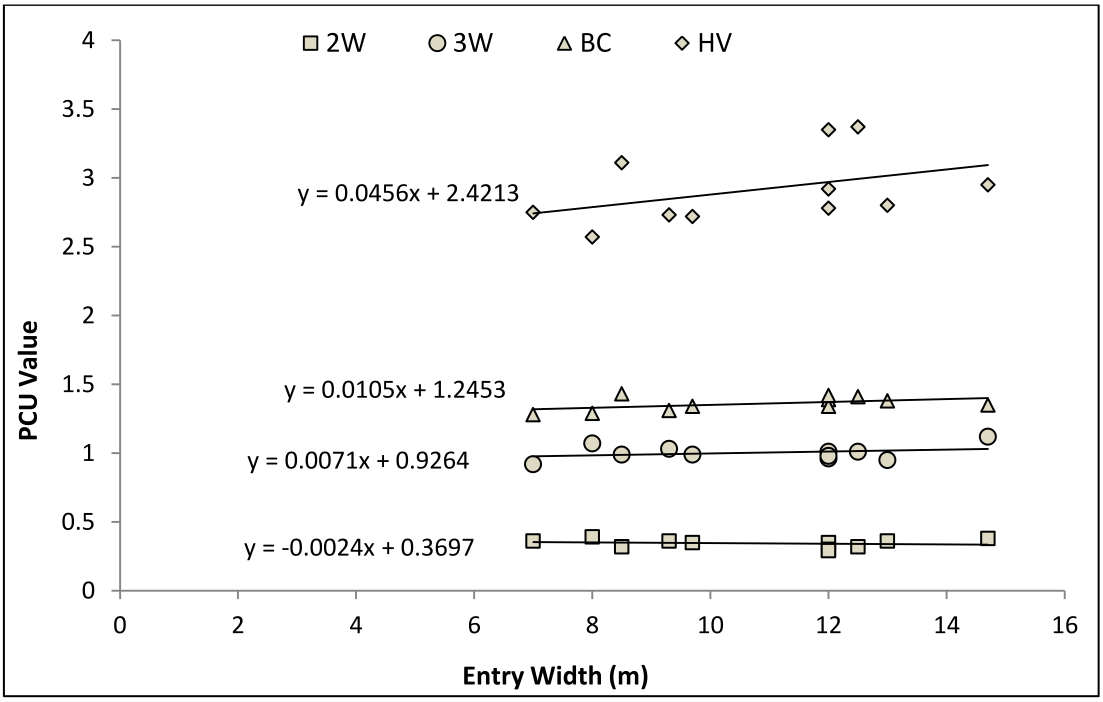


Figure 4.3 Variation in PCU values with entry width

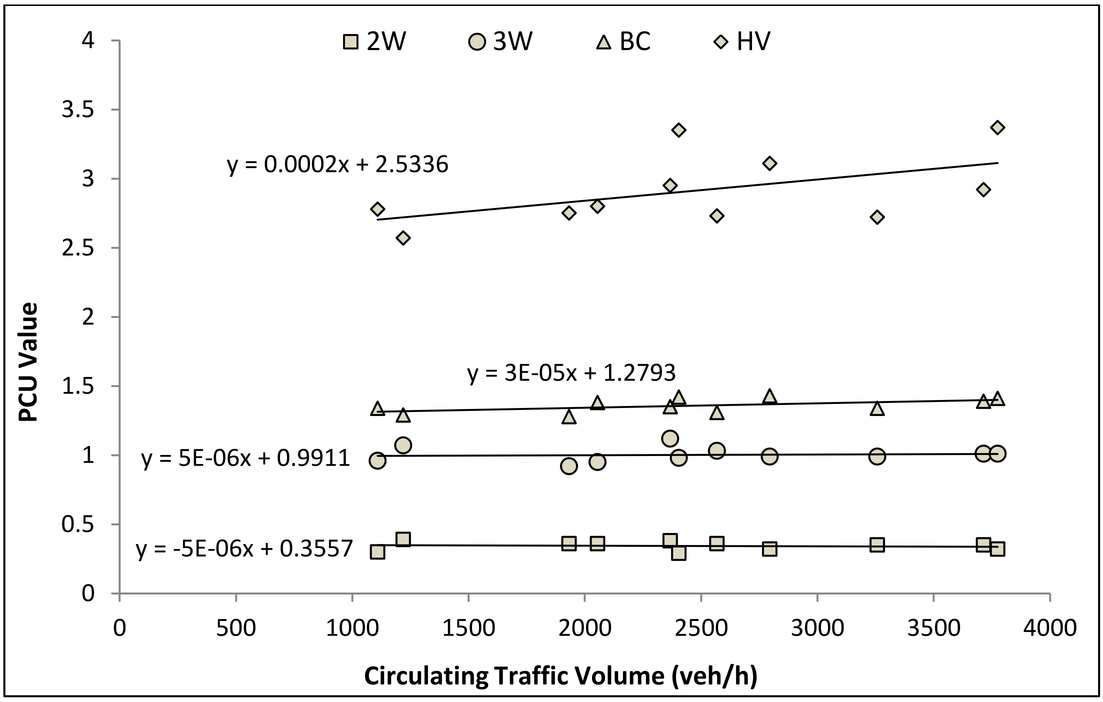


Figure 4.4 Variation in PCU values with circulating traffic volume

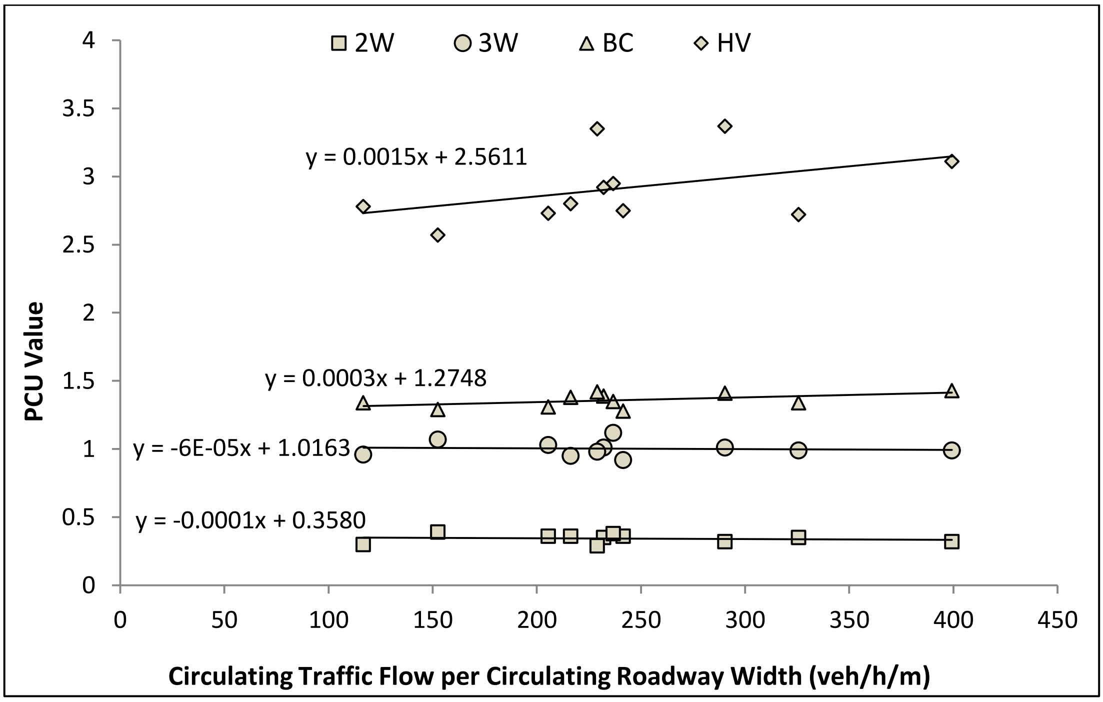


Figure 4.5 Variation in PCU values with circulating traffic flow per circulating roadway width

The coefficients of determination (R^2) for all the equations of PCU for different category of vehicles are very small (lower than 0.3) as given in Table 4.9. P-values for the coefficient in all the equations are not statistically significant at 95 percent level of confidence (p-values are greater than 0.05). Based on the coefficients of determination (R^2) value and p-value, it has been concluded that there is a weak or no relationship between PCU values of different vehicle categories, and geometric and traffic flow parameters of the roundabouts. This further indicates that the PCU values for different category of vehicles plying on a roundabout shall not be considered as dynamic and should be used as static.

Table 4.9 R^2 and p-values for the equations of PCU with different variables

		Central island diameter	**Circulating width**	**Entry width**	**Circulating flow**	**Circulating flow per circulating roadway width**
PCU_{2W}	**R^2**	0.0001	0.0051	0.0331	0.0182	0.0211
	P-value	0.9789	0.8344	0.5924	0.6922	0.6707
PCU_{3W}	**R^2**	0.0444	0.0269	0.0928	0.0055	0.0061
	P-value	0.5341	0.6297	0.3623	0.8283	0.8200
PCU_{BC}	**R^2**	0.0011	0.0606	0.2366	0.2972	0.2678
	P-value	0.9271	0.4655	0.1292	0.0829	0.1030
PCU_{HV}	**R^2**	0.0123	0.0636	0.1771	0.2715	0.1882
	P-value	0.7452	0.4545	0.1976	0.1003	0.1825

Apart from the dependence of PCU values on independent variables, information that becomes available is the possible influence of these variables on estimation of PCU values. It is visible that increase in the entry width and circulating roadway width will influence the PCU values of all categories of vehicles positively except 2W. The increase in circulating traffic flow will impact the PCU values of 2W negatively. This is due to acceptance of lower gaps by motorized two-wheeler in such congested conditions. The impact of circulating traffic flow is opposite for 3W, BC and HV, as their occupancy times will increase in the constrained conditions. The influence of circulating traffic flow, entry width and circulating width has been similar in nature. The increase in the circulating traffic flow per circulating roadway width clearly shows the division on both sides of the small car. The impact is negative in the case of 2W and 3W, and positive for BC and HV.

4.3 HETEROGENEITY EQUIVALENCY FACTOR (H-FACTOR)

Proper care is taken to arrive and suggest the static PCU values for different category of vehicles plying on roundabouts, but still there may be some influence of spatial and temporal conditions of traffic prevailing at a location. Analysis with respect to spatial condition has not shown any significant influence on estimation of PCU values. Further, when comparison is made between locations, the measurement unit of vehicle/hour may not give clear picture of traffic scenario at or across location(s). One location with high percent of cars and motorized two wheelers may get equated in terms of vehicle/hour with another location having high percent of heavy vehicles but lower percent of motorized two-wheelers. In such conditions, the measurement of traffic flow in veh/h becomes questionable as it may not represent the actual traffic scenario at a location. Mallikarjuna and Rao (2006b) have also reported that the determination of PCU value for each type of vehicle under heterogeneous traffic condition is a difficult task. Therefore, an approach is needed which obviates the need to re-estimate PCU values again and again. It is suggested that the heterogeneous traffic may be converted into an equivalent homogeneous traffic by using a Heterogeneity Equivalency Factor (H-factor). This is defined as a factor which when multiplied with the traffic flow, measured in veh/h, will give directly the flow in PCU/h. The use of this factor will omit the need to look back again on PCU values of individual vehicles.

Heterogeneity Equivalency Factor is estimated as the ratio of traffic volume in PCU/h (using new estimated PCU values as mentioned above) and volume in veh/h, and is denoted as ‘H-factor’. This is given by equation (4.1). The effort and cost of data collection would get reduced on using this concept. This concept is very easy to use and the problem of estimating the PCU values for each type of vehicles gets eliminated.

$$H-\text{factor} = \frac{\text{Flow in pcu / hr.}}{\text{Flow in veh / hr.}} = \frac{Q_{PCU}}{Q_{vph}} \tag{4.1}$$

The 10 minute flow values, in veh/h and in PCU/h, for combined data from eleven roundabouts are plotted in Figure 4.6. As may be seen, there is a good relation between flow in PCU/h (Q_{PCU}) and flow in veh/h (Q_{vph}). The straight line relation for roundabouts is given as equation (4.2). The straight line relation with zero intercept (since, H-factor must be zero for no flow condition) suggest the average value of H-factor is 0.8166 for flow on roundabouts. The value also indicates towards lower share of heavy vehicles in the flow. Value higher than 1.0 indicates that proportion of heavy vehicles is higher in traffic composition.

$$Q_{PCU} = 0.8166 * Q_{vph} \qquad (R^2 = 0.8786) \tag{4.2}$$

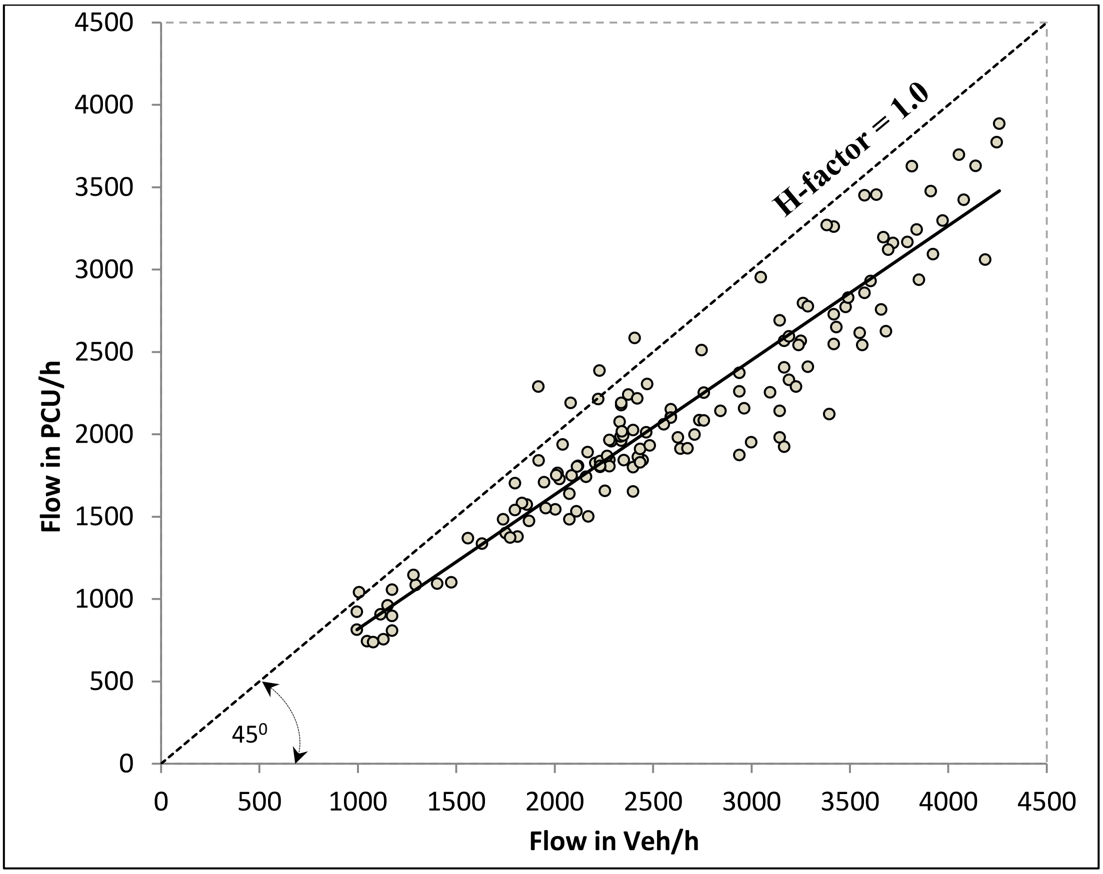


Figure 4.6 Plot between flow in veh/h and pcu/h at roundabouts

H-factor is then correlated with the circulating traffic flow, circulating traffic flow per circulating roadway width, traffic composition and geometric parameters of the roundabout. Correlation and regression analysis method has been used to estimate H-factor based on the above variables. It has been found that the circulating traffic flow and geometric parameters of the roundabout are not statistically significant at 95 percent level of confidence. However, the traffic composition and circulating traffic flow per circulating roadway width are found to be a significant parameter at 95 percent level of confidence. H-factor is expected to be higher for traffic flow with high proportion of HVs and would be lower for traffic flow with high proportion of 2Ws. The mathematical form of this relation is given by equation (4.3).

$$H = 1 + a * P_{2W} + b * P_{BC} + c * P_{HV} + d/V_{CW} \tag{4.3}$$

Where,

P_{2W} = Proportional fraction of two wheeler

P_{BC} = Proportional fraction of big car

P_{HV} = Proportional fraction of heavy vehicle

V_{CW} = Circulating traffic flow per circulating roadway width (veh/h/m)

The proportion of standard cars and motorized three-wheelers are merged together as mentioned under PCU estimation. These are not kept in the equation to avoid the problem of co-linearity. The values of regression coefficients ‘a’, ‘b’, ‘c’, and ‘d’ are estimated using field data and are given in equation (4.4).

$$H = 1 - 0.676 * P_{2W} + 0.508 * P_{BC} + 2.718 * P_{HV} - 6.081/V_{CW} \quad (R^2 = 0.853) \tag{4.4}$$

The statistical characteristics of developed model are given in Table 4.10. In Table 4.10, ‘t’ statistics show that all the coefficients are significant at 95 percent level of confidence as all values lie outside the critical value of -1.98 to 1.98. Signs of coefficients are also logical as the value of H-factor would increase for all vehicles larger than car and would reduce for all vehicles smaller than car. The coefficient of determination (R^2) for equation (4.4) is 0.853, which also indicates toward good strength of the model in predicting H-factor. Table 4.10 also indicates that the traffic composition of motorized two wheeler, big car and heavy vehicles had a strong effect on H-factor as the value of *Significance F* or p-value of the model (1.04E-187) is much less than 0.05. This also signifies that the regression output is not merely a chance occurrence.

Table 4.10 Statistical characteristics of the developed model for H-factor

Independent Variable	**Coefficients**	**Std. Error**	**t stat**	**P-value**	**Lower Bound**	**Upper Bound**
2W	-0.676	0.021	-31.852	5.19E-66	-0.717	-0.634
BC	0.508	0.132	3.836	0.00019	0.246	0.770
HV	2.718	0.167	16.230	5.57E-34	2.387	3.049
V_{CW}	-6.081	2.430	-2.503	0.01347	-10.885	-1.278
Analysis of Variance						
	DF	**Sum of Squares**	**Mean Squares**	**F**	**Significance F (p-value)**	
Regression	4	98.75	24.688	17410.78	1.04E-187	
Residual	140	0.20	0.001			
Total	144	98.95				

2W= Motorized two-wheelers; 3W= Motorized three-wheelers; BC= Big cars; V_{CW} = Circulating traffic flow per circulating roadway width

4.3.1 Effect of Traffic Composition on H-factor

H-factor would depend on proportional composition of different types of vehicles in the circulating traffic flow. To show the variation in H-factor with composition of traffic flow, the proportion of two types of vehicles was kept constant and proportion of remaining two types of vehicles was varied. Figure 4.7 shows the effect of 2W on H-factor at roundabouts, when the proportion of BC and HV was kept fixed at 10% and 5% respectively. The circulating traffic flow per circulating width was also kept constant as 200 veh/h/m. Similarly, the variation in H-factor with proportion of BCs, a curve was drawn by fixing the proportion of 2Ws and HVs at 25% and 5% respectively (Figure 4.8). Similar plot for variation in H-factor with proportion of HVs is shown in Figure 4.9 by fixing the proportion of 2Ws and BCs at 30% and 10% respectively. The trend in these three figures is on expected lines. H-factor is the weighted average PCU for the entire traffic flow and its value shall increase with an increase in the proportion of vehicles in the traffic flow which have their PCU greater than unity (like HVs and BCs), and shall decrease with an increase in the proportion of the vehicles in the traffic stream which have their PCU smaller than unity (like 2Ws).

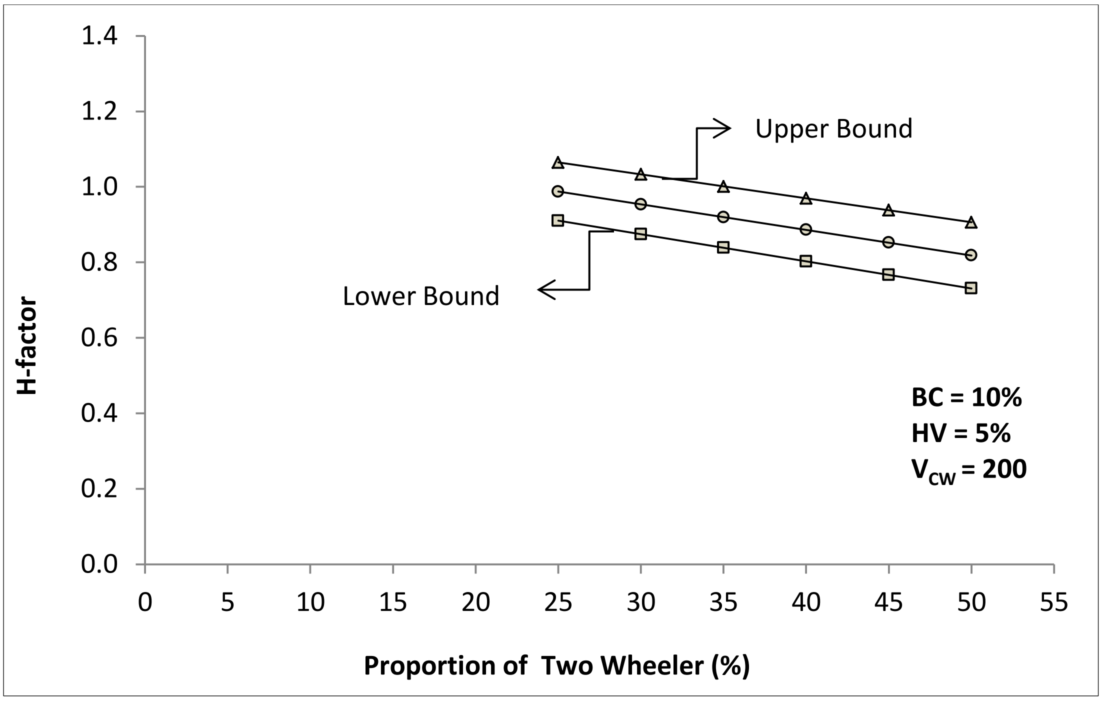


Figure 4.7 Effect of proportion of two wheelers on H-factor

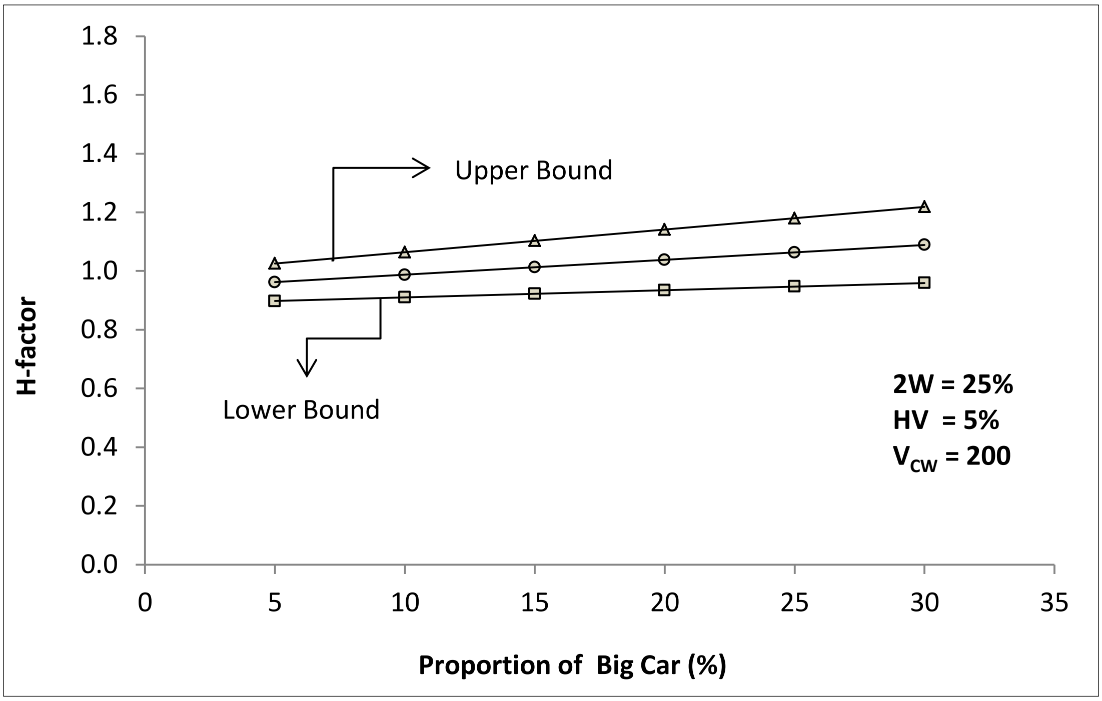


Figure 4.8 Effect of proportion of big cars on H-factor

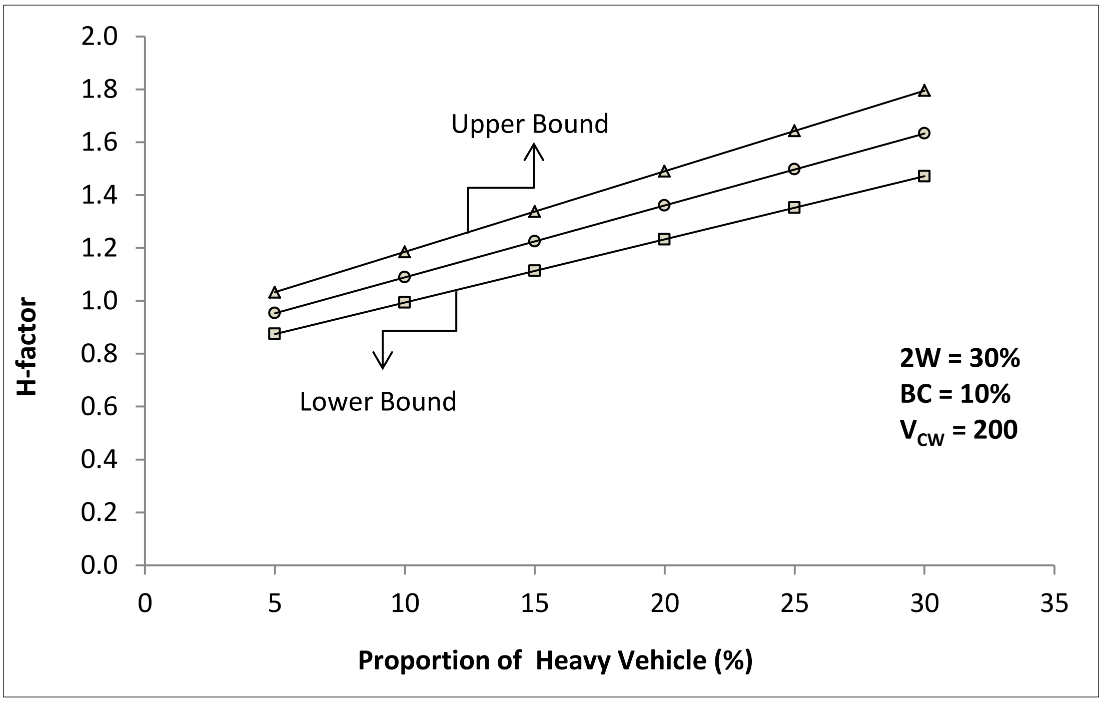


Figure 4.9 Effect of proportion of heavy vehicle on H-factor

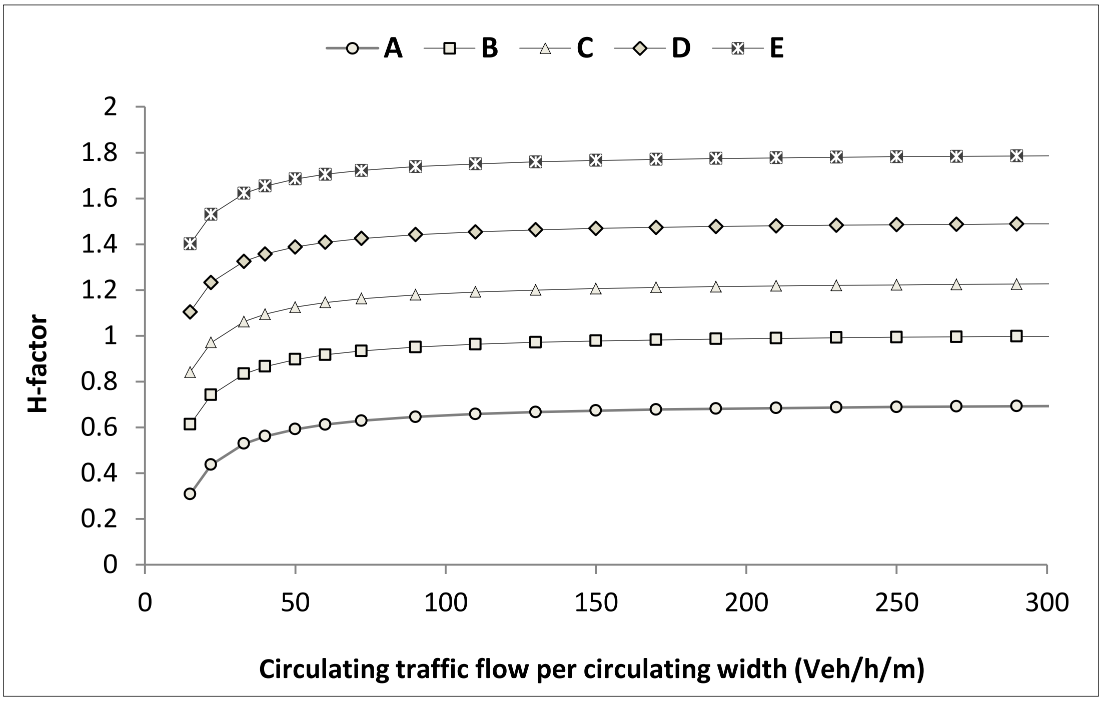


Figure 4.10 Effect of circulating traffic flow per circulating width on H-factor

The effect of circulating traffic volume on H-factor is shown in Figure 4.10. The curves are drawn for a traffic stream with five groups of different compositions of vehicles. These are:

Group A: 40% SC, 10% BC, 0% HV, and 50% 2W,

Group B: 60% SC, 10% BC, 5% HV, and 25% 2W,

Group C: 60% SC, 15% BC, 10% HV, and 15% 2W,

Group D: 65% SC, 20% BC, 15% HV, and 0% 2W,

Group E: 50% SC, 25% BC, 25% HV, and 0% 2W.

H-factor is found increasing with an increase in the circulating traffic flow per circulating roadway width and tends to become constant at higher circulating traffic flow, irrespective of the composition of vehicles. It shows more homogeneity in the circulating traffic stream at higher circulating traffic flow. H-factor also increases with an increase in the percentage of HVs in the traffic stream, whereas, it increases with a decrease in the percentage of 2Ws in the traffic stream.

4.4 SUMMARY

Analysis was done to estimate the PCU factors for different categories of vehicles on roundabouts to express heterogeneity in traffic in a common unit of standard car. Lagging headway and width of the vehicle were used in its estimation. Lagging headway is a traffic parameter and expected to vary with type of vehicle and traffic culture. This is already evident from the data given in Table 4.2. But the variation is relative and therefore, when used as a proportion it makes no significant impact. The PCU values obtained by the proposed approach are found partly comparable with those used in different countries. Pressure on roads is causing higher impatience behavior on account of motorized two-wheelers. Impact of technological changes in cars and the entry of their variants in developing countries have made it necessary to divide car category in two, as small car and big car. This is evident from the PCU value for big car which is substantially up than small car. Heavy vehicles are impacted more in Indian traffic flow condition as compared to other developing and developed countries. Their PCU values are comparable only to Malaysian. For the analysis of any traffic study related to roundabouts in India or other developing countries, the static PCU values are recommended as: 0.34 for motorized 2-wheeler, 1 for motorized 3-wheeler and small car, 1.36 for big car and 2.91 for heavy vehicle. The PCU values suggested here are applicable to other developing countries or other cities within India. It has already been shown that PCU values are not getting influenced by traffic and physical variables in the roundabout. Hence, these can be considered as stand-alone values. The main difference with respect to the already in use IRC code is that car category got divided as small and big car; and PCU values for 2W have become almost one-half of the existing one.

An attempt was also made to avoid the re-estimation of PCU values for different categories of vehicles for transforming heterogonous traffic into a homogenous one. The concept of heterogeneity equivalency factor is used for transforming heterogeneous traffic volume in veh/h to homogenous traffic volume in PCU/h without using PCU values for different category of vehicles. This concept is very easy to use and the problem of estimating PCU values for each type of vehicles under varying traffic flow conditions gets eliminated. The developed regression model for heterogeneity equivalency factor is applicable only on roundabouts. Similar model may be tried to develop for other types of intersections and roads.

CHAPTER 5:

CRITICAL GAP AND FOLLOW-UP TIME

5.0 GENERAL

The procedure for determination of entry capacity at an uncontrolled intersection like roundabout is generally based on gap-acceptance models. Critical gap is one of the major traffic parameter which is used in such models (Amin and Maurya 2015, 2016). The accuracy of capacity estimation is mainly dependent on the accuracy with which the critical gap is estimated. Estimation of critical gap for a vehicle type under mixed traffic conditions prevailing in developing countries has always been a challenging task. This is due to drivers not following lane discipline during congestion periods and very limited priority being followed by the vehicles at priority intersections like roundabouts. There are different methods available in literature for the estimation of critical gap. These methods will be examined for vehicles plying on different roundabouts. The results of the proposed method will be compared with the other methods. Literature also indicates that maximum likelihood method gives most reliable and accurate results. The proposed method is also compared with maximum likelihood method to examine the relative efficiency of the methods. Concept like percent violation has been used for such comparison. Follow-up time are also extracted and reported for selected roundabouts and their ratio with respect to critical gap is also reported. Comparison with findings from literature is also reported.

5.1 NEW APPROACH FOR CRITICAL GAP ESTIMATION

A new approach is proposed for estimating the critical gap. The approach is based on minimization of the sum of absolute difference between a gap value and accepted / rejected gap. Figure 5.1 shows the distribution function of rejected, accepted and critical gaps. The distribution function $F_c(t)$ of the critical gaps must be positioned between the distribution functions of rejected gaps $F_r(t)$ and the distribution function of accepted gaps $F_a(t)$ so that the difference of all accepted gaps and all rejected gaps from critical gap remains minimum.

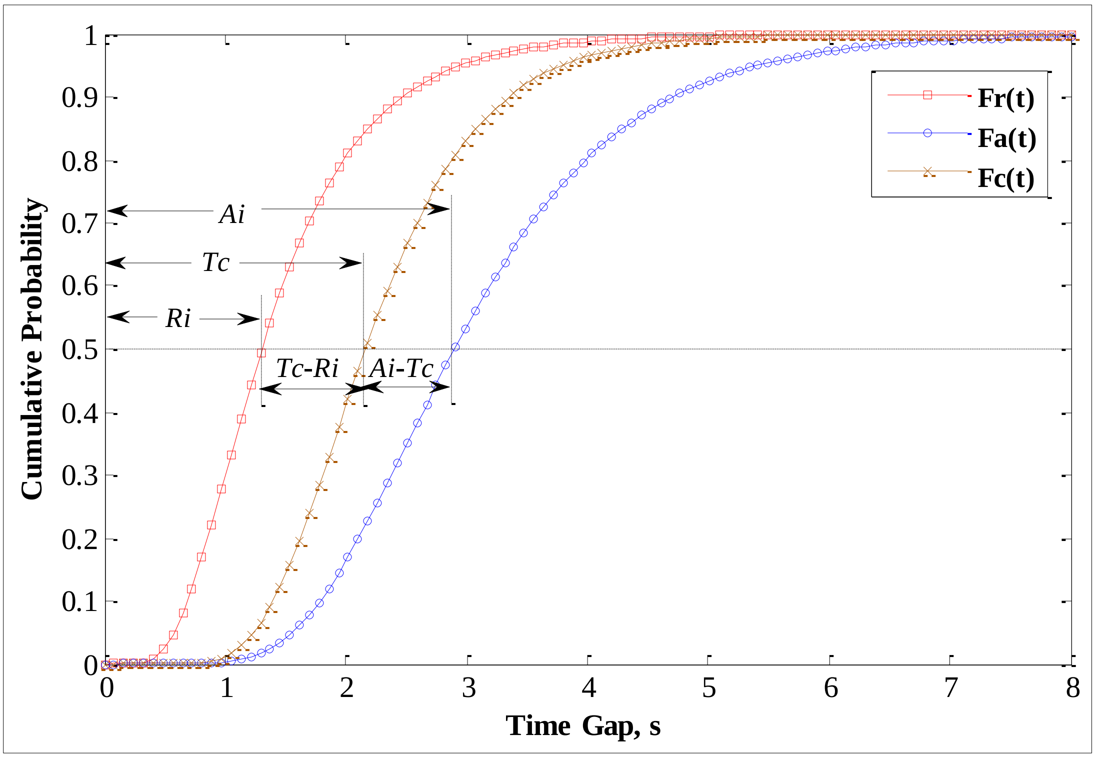


Figure 5.1 Cumulative distribution function of highest rejected and accepted gaps

The difference of rejected gap (R_i) from critical gap (T_c) is (T_c – R_i), and the difference of accepted gap (A_i) from critical gap is (A_i – T_c). Therefore, the total difference in the two values can be written as equation (5.1).

$$f = \left\{ \mathrm{Abs}\left(T_c - R_i\right) + \mathrm{Abs}\left(A_i - T_c\right) \right\} \tag{5.1}$$

Where,

A_i = Accepted gap by the ith entering vehicle (seconds),

R_i = Highest rejected gap by the ith entering vehicle (seconds),

T_c = Critical gap value (seconds)

For estimating the critical gap (T_c), the total difference (the sum of absolute values of differences) should be minimized. This is a mathematical optimization problem and can be written as equation (5.2).

$$\mathrm{Min}\left[\sum_{i=1}^{n} \left\{ \mathrm{Abs}\left(T_c - R_i\right) + \mathrm{Abs}\left(A_i - T_c\right) \right\} \right] \tag{5.2}$$

This method requires two sets of data, the highest rejected gaps and accepted gaps. This method is different from maximum likelihood method in two aspects especially prevailing in developing countries. One, it does not make prior assumption of consistent drivers which is always untrue for real world situation and two, it does not fail even when there are very few rejected gaps. Rejected gap in such a case can be taken as zero, instead of fixing as some small value as done in MLM method.

The function given in equation (5.2) can be minimized by using optimization toolbox in MATLAB or Solver in the MS EXCEL. This is an iterative process and the first value for iteration must be logical. It would be good to use average of all the highest rejected gaps and all accepted gaps as a first value or starting point so that it could converges fast and reduces the number of iterations. Spreadsheet for estimating the critical gap by using proposed method is given in Table 5.1.

Table 5.1 Spreadsheet for estimating the critical gap by proposed method

	A	**B**	**C**
1	**Highest Rejected Gaps (R_i)**	**Accepted Gaps (A_i)**	$\left[\mathbf{Abs}\left(T_c\text{-}R_i\right)+\mathbf{Abs}\left(A_i\text{-}T_c\right)\right]$
2	0.56	2.08	1.96
3	1.78	4.58	2.80
4	0.00	2.08	2.52
5	0.88	2.24	1.48
…	…	…	…
…	…	…	…
…	…	…	…
46	1.44	3.04	1.60
47	1.96	3.76	1.80
48	0.00	2.92	2.92
49	AVERAGE (Initial value of T_c)		2.30
50	SUM		487.59

The values in the column A and column B are the highest rejected gaps (R_i) and accepted gaps (A_i) respectively. An initial value of T_c (Cell C49) is needed to start the iteration. This is taken as the average of all accepted and rejected gaps (values in cell A2 to B48). This is used to calculate the value of function as given in column C. The sum of the function is given in the cell C50. The iteration process is started using the solver function in MS EXCEL to get the minimized value of sum of differences (in absolute values). The iterative process converged the function value from 487.59 to 480.18. The convergence was achieved at T_c = 2.11 sec, which is the critical gap.

The proposed method has been checked with variation in initial value of T_c to start the iteration. The critical gap values at different initial values of T_c are given in Table 5.2. It can be seen that the variation in critical gap values is only ±0.01s and the effect of initial value of T_c on critical gap is negligible. Consequently, any initial value of T_c for iteration can be used to estimate the critical gap. However, the suggested initial value of T_c to start the iteration is the average of all accepted and rejected gaps.

Table 5.2 The critical gap values at different initial value of T_c

S. N.	**Initial Value of T_c**	**Critical Gap, T_c (seconds)**
1	0.01	2.12
2	0.05	2.11
3	0.1	2.12
4	0.5	2.10
5	1.0	2.12
6	1.5	2.10
7	2.0	2.10
8	2.5	2.11
9	3.0	2.10
10	3.5	2.10
11	4.0	2.11
12	5.0	2.12
13	6.0	2.12
14	7.0	2.12

The critical gap values could be estimated only on five roundabouts, namely R_1, R_2, R_3, R_5, and R_6. It was problematic to estimate the critical gap on other roundabouts due to their exceptionally large circulating roadway width, entry width and weaving length. The gap acceptance approach is not working on these roundabouts due to propensity of merging and diverging maneuvers of entering and circulating traffic vehicles. In this situation, entering vehicles do not use gap between the circulating vehicles. However, they merge into circulating traffic stream then diverge to their respective directions. The ease of operation has made it difficult to estimate the critical gap on these roundabouts.

The critical gap values are estimated by the proposed method for five categories of vehicles as already mentioned viz. standard cars (SC), big cars (BC), motorized two-wheelers (2W), motorized three-wheelers (3W), and heavy vehicles (HV). The estimated critical gap values are given in Table 5.3 for the five selected roundabouts.

Table 5.3 Critical gaps estimated by proposed method

Roundabout ID	**Critical Gaps (Seconds)**				
	2W	**3W**	**SC**	**BC**	**HV**
R_1	1.60	1.94	2.30	2.39	2.67
R_2	1.50	1.88	2.11	2.21	2.55
R_3	1.48	1.84	2.08	2.13	2.45
R_5	1.55	1.73	1.85	1.92	2.63
R_6	1.59	1.68	1.97	2.03	2.52

The critical gap value is found varying between 1.48-1.60 s for two-wheelers, 1.68-1.94 s for three-wheelers, 1.85-2.30 s for small cars, 1.92-2.39 s for big cars and 2.45-2.67 s for heavy vehicles. These values are lower than the critical gap values as reported in literature. It may be attributed to the behavior of the drivers on roads under mixed traffic conditions in developing countries. These drivers are ready to pick small gaps to complete their maneuver. It is because of higher pressure on roads during peak periods, may be due to increasing motor vehicle ownership and population. The data interpretation may put these drivers in aggressive driver category, as compared to the driver discipline and behavior in developed countries, which is traffic lane based. Patil and Pawar (2014) estimated the critical gap values at four-legged unsignalized intersections in Kolhapur city located in south-west part of Maharashtra, India. They found that the critical gap values are smaller than that reported in developed countries including HCM (2010) indicating drivers' aggressiveness in India. Pradeep et al. (2015) also found that the estimated critical gap and follow-up time values for Indian traffic conditions are lower than as given in HCM (2010).

5.2 ESTIMATION OF CRITICAL GAP

Data on accepted and rejected gaps are analyzed into two parts. First, the critical gaps for five categories of vehicles are estimated using five most popular methods and results are compared and discussed. In the second part of the analysis, the critical gap is estimated by the proposed method and the results obtained by this method are compared with the results of other methods.

5.2.1 Comparison of Existing Methods

The critical gap values were estimated using different methods namely Harders, Ashworth, Modified Raff, MLM and Wu on roundabout R_2. The estimated critical gap values are given in Table 5.4. These are found to be ranging between 1.32-2.18s for two-wheelers, 1.86-2.38 for three-wheelers, 1.75-2.74s for small cars, 2.14-2.40s for big cars and 2.45-3.06s for heavy vehicles. It can be seen that with an increase in the size of the vehicle the critical gap value is increasing. This may be due to the space occupied by a vehicle category and availability of lesser opportunities for their maneuverability.

Table 5.4 Critical gaps estimated by different methods

Method	Critical gap (s)				
	2W	3W	SC	BC	HV
Ashworth	1.32	1.86	1.75	2.14	3.06
Harders	2.18	2.38	2.76	2.40	2.50
MLM	1.59	1.92	2.12	2.18	2.51

Wu	1.91	2.00	2.25	2.34	2.45
Modified Raff	1.60	1.90	2.10	2.30	NA*

* Sample data not sufficient

The comparison among the methods revealed the following:

Ashworth method gave the lowest and the Harders' method gave the highest estimates of critical gap for vehicles like two-wheelers, three-wheelers, small cars and big cars. In the case of heavy vehicles, Wu method gave the lowest estimates. The critical gap of big cars and heavy vehicles was found around 4.3 percent and 18.4 percent higher respectively than the estimated critical gap of small cars. In the case of 2-wheelers and 3-wheelers, the critical gap was 25 and 9.4 percent lower respectively than that of a small car.

The maximum likelihood method is reported in literature as the best method among all methods. The suitability of this method to mixed traffic conditions is judged by the criterion of explaining the percent violators. There are two sets of data, one highest rejected gaps and the other accepted gaps. The critical gap value should be greater than all the highest rejected gaps and smaller than all accepted gaps. Some values of highest rejected gaps and accepted gaps would not satisfy this criterion because of inconsistent drivers and these are termed as violations. The percentage of violated cases for five categories of vehicles under different estimation methods are given in Table 5.5.

Table 5.5 Percent violations across methods of critical gap estimation

Vehicle	**Method**	**Violations, %**		
		Rejected Gaps	**Accepted Gaps**	**Overall**
2W	Ashworth	34.38	10.94	22.66
	Harders	1.95	51.17	26.56
	MLM	18.36	26.17	22.27
	Wu	7.42	41.41	24.41
	Modified Raff	18.36	28.13	23.24

3W	Ashworth	24.07	18.53	21.30
	Harders	14.81	42.59	28.70
	MLM	22.22	22.22	22.22
	Wu	16.67	25.93	21.30
	Modified Raff	22.22	22.22	22.22
SC	Ashworth	29.92	8.20	19.06
	Harders	5.33	47.54	26.43
	MLM	17.62	17.62	17.62
	Wu	12.30	24.18	18.24
	Modified Raff	19.26	17.62	18.44
BC	Ashworth	11.63	9.30	10.47
	Harders	4.65	20.93	12.79
	MLM	11.63	11.63	11.63
	Wu	6.98	18.60	12.79
	Modified Raff	6.98	20.93	13.95
HV	Ashworth	0.00	5.56	2.78
	Harders	0.00	0.00	0.00
	MLM	0.00	0.00	0.00
	Wu	0.00	0.00	0.00
	Modified Raff	NA*	NA*	NA*

* Sample data not sufficient

Maximum likelihood method gave minimum percentage of total violated values for two-wheelers in comparison to the other methods. Harders method gave the maximum case of violations. In the case of rejected gaps, Harders and Wu methods have least percentage violations while Ashworth method has maximum percentage of violated case. But no method provided equal violations for accepted and rejected gaps. Ashworth method gave minimum percentage of total violated values for three-wheelers and big cars in comparison to other methods but again did not provide equal violations for accepted and rejected gaps. Maximum likelihood method again resulted in the minimum percentage of total violated cases for motorized two-wheeler, small cars and heavy vehicles as compared to the other methods.

Therefore, it is inferred that Harders method gave high estimates (because of maximum cases of violations) and is not good for the estimation of the critical gap value for given traffic condition on roundabouts in developing countries. Ashworth method does not utilize data of rejected gap. Modified Raff's method is not appropriate for a small sample size as can be seen for heavy vehicles. Out of the discussed methods, maximum likelihood method is found to be the best for estimating the critical gap value with almost equal violations for rejected and accepted gaps. However, one criticism of this method is that it does not work when no gaps are rejected by the entering vehicles. Vasconcelos et al. (2012a) suggested to set the highest rejected gap to some very small value while using the maximum likelihood method in such situation. Troutbeck (2014) also suggested to set highest rejected gap as 0.1s when no gaps were rejected. However this assumption may influence the outcome when rejected gaps are zero in majority of the cases, which generally happen in mixed traffic conditions prevailing in most of the developing countries.

5.2.2 MLM versus Proposed Method

A comparison is made between the proposed method and the maximum likelihood method based on violation cases. The comparative values of percent violation cases for the two methods are given in Table 5.6.

Table 5.6 Percent violation cases for MLM and proposed method

Vehicle	**Method**	**Violations, %**		
		Rejected Gaps	**Accepted Gaps**	**Overall**
2W	MLM	18.36	26.17	22.27
	Proposed	22.27	22.27	22.27

3W	MLM	22.22	22.22	22.22
	Proposed	22.22	22.22	22.22
SC	MLM	17.62	17.62	17.62
	Proposed	17.62	17.62	17.62
BC	MLM	11.63	11.63	11.63
	Proposed	11.63	11.63	11.63
HV	MLM	0.00	0.00	0.00
	Proposed	0.00	0.00	0.00

Maximum likelihood method and the method of minimizing the sum of absolute differences have equal overall violated cases for all the five categories of vehicles except two-wheelers. In the case of two-wheelers, critical gap value using maximum likelihood method does not provide equal violations in rejected and accepted gaps while proposed method does. It implies that the definition of critical gap (the gap that has an equal probability of being accepted or rejected) is getting satisfied by the proposed method. Hence, it looks to be more satisfying as compared to maximum likelihood method.

Further, it is important to check the accuracy of the critical gap estimated by the proposed method. According to Velan and Van Aerde (1996), a small variation in the critical gap would result in significant variation in the entry capacity. Vasconcelos et al. (2012a) reported that 0.5s difference in the critical gap can result in the capacity difference of up to 15%. It is already been observed that the proposed method could estimate the critical gap with both, small and large sample of data set, as well as, in situation where no gap is rejected by a vehicle during peak hours or lean traffic flows. In order to observe the effect of sample size on results of critical gap under heterogeneous traffic flow condition, an analysis is done with varying sample size. The range of critical gaps and the maximum deviation (from the final critical gap obtained from large sample size) for two-wheelers, three-wheelers and small cars are given in Table 5.7, Table 5.8 and Table 5.9 respectively. It is found that the deviation from critical gap value decreased with an increase in the number of data set. In the case of two-wheelers, the maximum deviation from critical gap value by proposed method and maximum likelihood method was 0.11s and 0.12s respectively for the data set of 30. In the case of three-wheelers, it was 0.12s and 0.14s respectively. In the case of small cars, it was 0.18s and 0.20s respectively. If a limit of maximum deviation is set at 0.1s then the optimum data set comes out to be 45 for two-wheelers and three-wheelers each, and 60 for small cars. If a limit of maximum deviation is set at 0.05s then the optimum data set comes out to be 60 for two-wheelers and three-wheelers each, and 150 for small cars. This indicates that the behavior of small car drivers is relatively more inconsistent within the group as compared to two-wheeler and three-wheeler drivers. The optimum number of data set for big cars and heavy vehicles could not be estimated. The required number of sample size for big cars and heavy vehicles was not sufficient to compute optimum number of data set. The percentage of heavy vehicles was only 2 percent of the entry traffic flow.

Table 5.7 Range of critical gaps and maximum deviation for two-wheelers

Data set	**Proposed Method**		**Maximum likelihood Method**	
	Range of critical gaps in a set	**Maximum deviation from critical gap (1.50)**	**Range of critical gaps in a set**	**Maximum deviation from critical gap (1.59)**
30	1.40-1.61	0.11	1.49-1.71	0.12
45	1.44-1.58	0.08	1.51-1.64	0.08
60	1.45-1.53	0.05	1.52-1.63	0.07
90	1.48-1.53	0.03	1.57-1.60	0.02
120	1.50-1.50	0.00	1.56-1.61	0.03
150	1.47-1.53	0.03	1.55-1.61	0.04
180	1.50	0.00	1.58	0.01
210	1.50	0.00	1.57	0.02
240	1.50	0.00	1.58	0.01
270	1.50	0.00	1.59	0.00

Table 5.8 Range of critical gaps and maximum deviation for three-wheelers

Data set	Proposed Method		Maximum likelihood Method	
	Range of critical gaps in a set	Maximum deviation from critical gap (1.88)	Range of critical gaps in a set	Maximum deviation from critical gap (1.92)
30	1.82-2.0	0.12	1.87-2.06	0.14
45	1.84-1.96	0.08	1.85-2.01	0.09
60	1.86-1.94	0.06	1.88-1.99	0.07
75	1.92	0.04	1.98	0.06
90	1.91	0.03	1.94	0.02
105	1.89	0.01	1.94	0.02
120	1.88	0.00	1.92	0.00

Table 5.9 Range of critical gaps and maximum deviation for small cars

Data set	Proposed Method		Maximum likelihood Method	
	Range of critical gaps in a set	Maximum deviation from critical gap (2.11)	Range of critical gaps in a set	Maximum deviation from critical gap (2.12)
30	1.93-2.20	0.18	1.93-2.32	0.20
45	1.92-2.21	0.19	2.01-2.29	0.17
60	2.03-2.17	0.08	2.04-2.18	0.08
90	2.03-2.13	0.08	2.05-2.16	0.07
120	2.03-2.17	0.08	2.05-2.14	0.07
150	2.05-2.15	0.06	2.04-2.15	0.08
180	2.10	0.01	2.09	0.03

210	2.10	0.01	2.11	0.01
240	2.11	0.00	2.12	0.00

5.2.3 Relationship between Critical Gap and Circulating Traffic Flow

The relationship between critical gap and circulating traffic flow on roundabout is discussed at this juncture. For this study, only small car and motorized two-wheeler are considered, because the percentage of other vehicles is not sufficient to generate adequate gap data. Gaps (accepted and rejected by a vehicle) were extracted for every 15 minute interval. These were used to estimate critical gap using proposed method. In the same time interval of 15 minutes, the circulating traffic data were also extracted from the video. The associative critical gaps for small car and motorized two-wheeler and the circulating traffic volume are plotted. The circulating traffic volume was converted into pcu using the values as recommended before. The variation of critical gap with circulating traffic flow on five roundabouts is shown in Figure 5.2 to Figure 5.6. It has been observed that with an increase in the circulating traffic flow the estimated value of critical gap remains more or less constant. To arrive at the best fit relationship between the estimated critical gap and circulating traffic flow a large number of distributions were examined. The best fit is found to be the exponential function between the two, but with very low value of coefficient of correlation. Based on the statistical parameters it has been concluded that there is no significant effect of the circulating traffic flow on the critical gap. The estimated critical gap value for motorized two-wheeler is around 24% lower than that of small car.

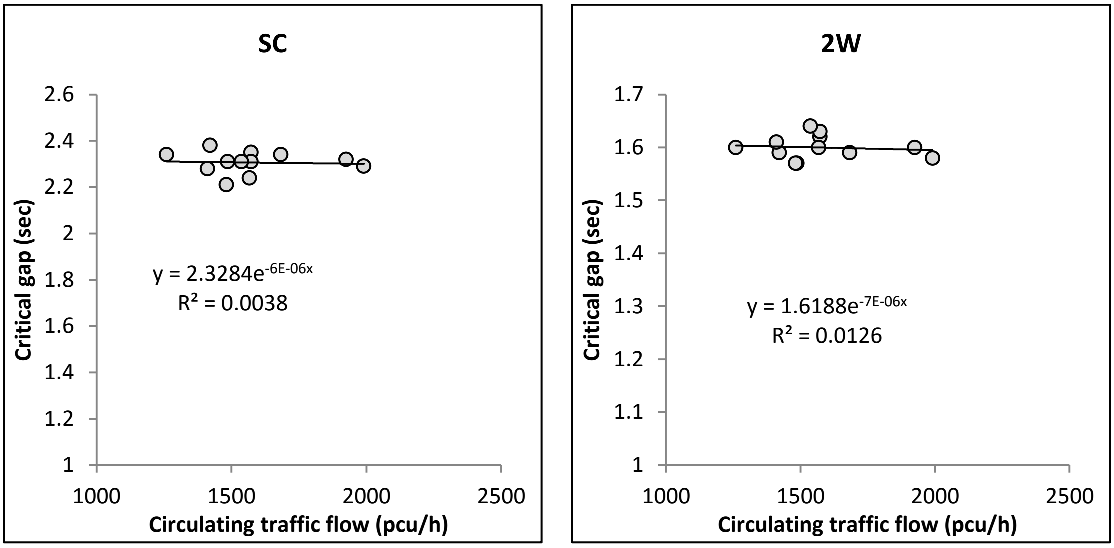


Figure 5.2 Critical gap v/s circulating traffic flow at R_1

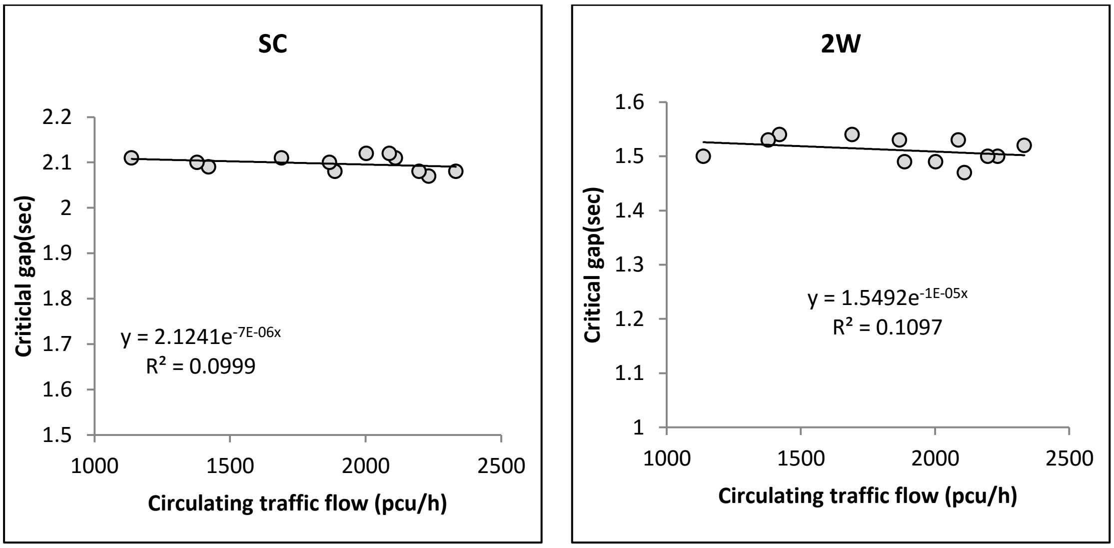


Figure 5.3 Critical gap v/s circulating traffic flow at R_2

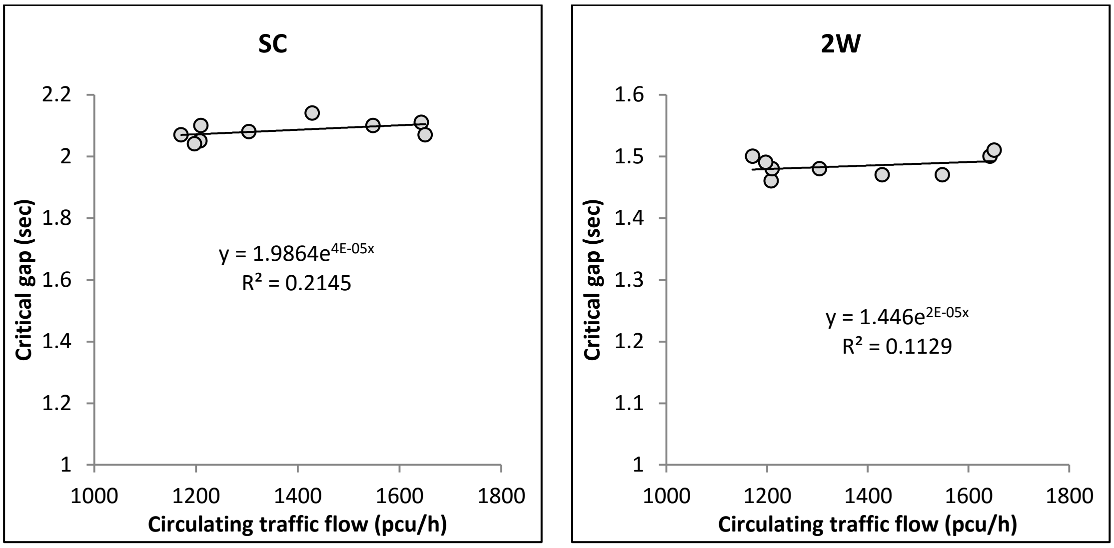


Figure 5.4 Critical gap v/s circulating traffic flow at R_3

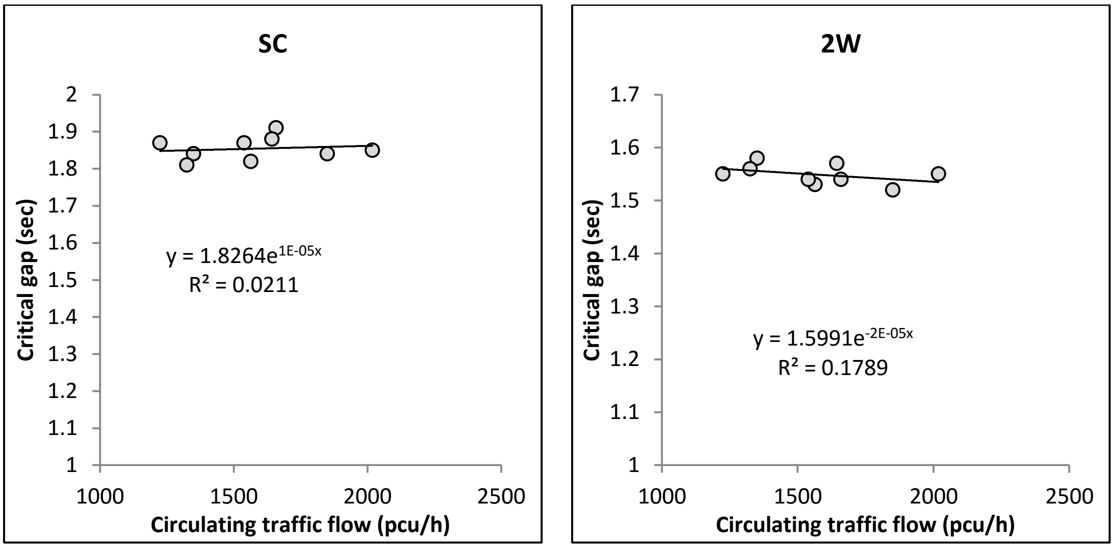


Figure 5.5 Critical gap v/s circulating traffic flow at R_5

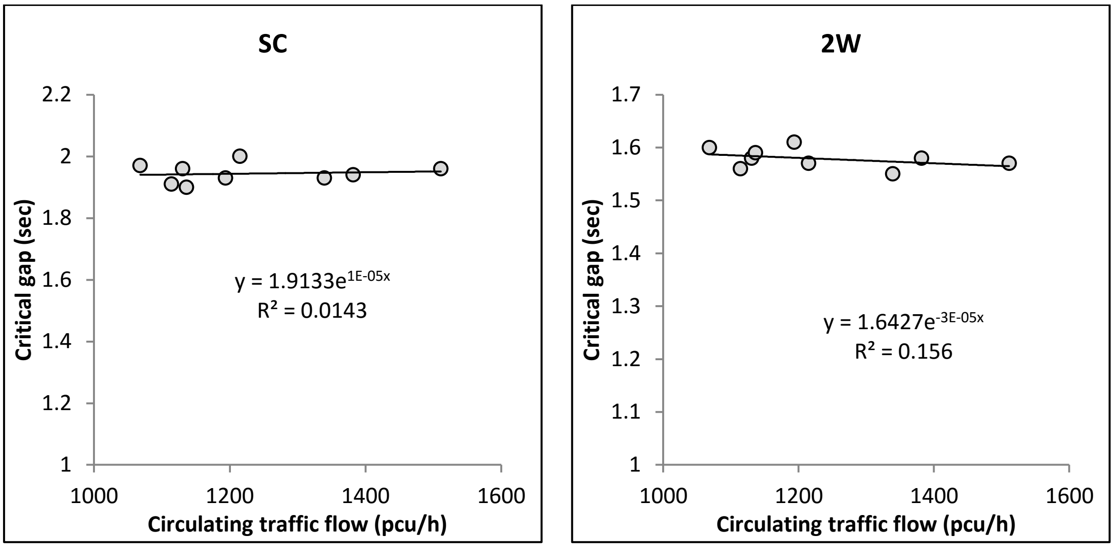


Figure 5.6 Critical gap v/s circulating traffic flow at R_6

5.3 FOLLOW-UP TIME

For gap acceptance based analytical procedures, another important flow variable is the follow-up time. This is important in the present study as one of the aims of research is to estimate the capacity of an entry at an approach of a roundabout. The accuracy of capacity estimate is mainly determined by the accuracy of estimation of the critical gap and follow-up time. If more than one vehicle from an approach of the roundabout uses a gap then the succeeding vehicles are referred to as follow-ups. The follow-up time is the headway between the vehicles entering into circulating area of the roundabout as shown in Figure 5.7. The follow-up time can only be measured when there is a queue situation.

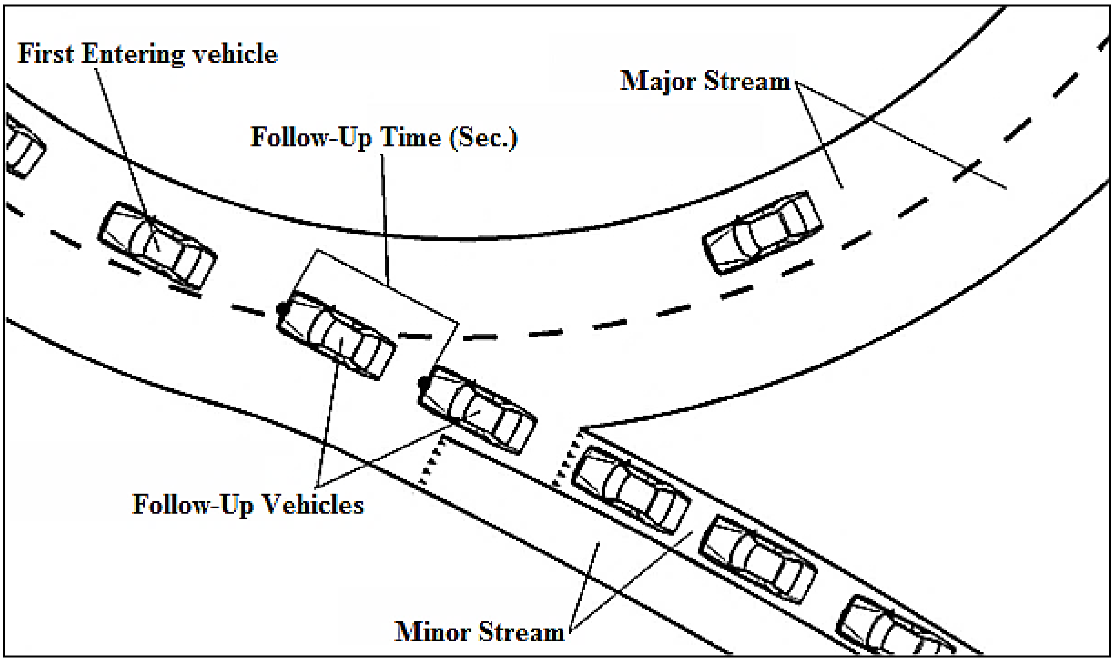


Figure 5.7 Definition of follow-up time

5.3.1 Relationship between Follow-up Time and Circulating Traffic Flow

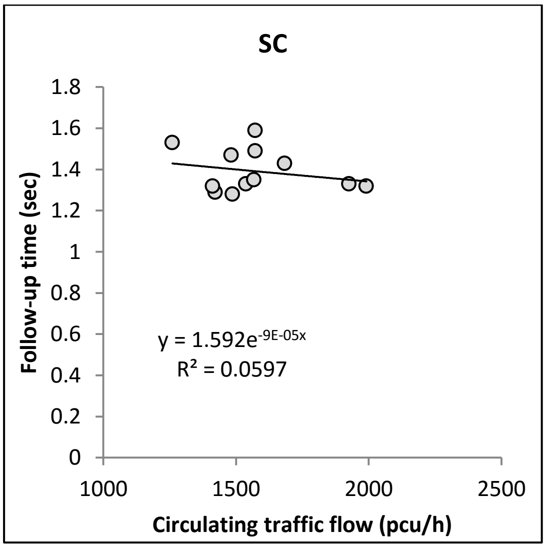
The follow-up time could be measured only for motorized two-wheeler and small car as observations related to small car followed by small car and motorized two-wheeler followed by motorized two-wheeler were available in sufficient numbers. The proportion of motorized three-wheelers and heavy vehicles are not much as compared to motorized two-wheelers and small cars. Therefore, these followed by respective vehicle category did not happen regularly on the studied roundabouts. The follow-up time of motorized two-wheeler and small car on five roundabouts are given in Table 5.10.

Table 5.10 The follow-up time for small car and motorized two-wheeler

Roundabout	Follow up time	
ID	2W	SC
R_1	0.99	1.41
R_2	0.95	1.30
R_3	0.97	1.33

R_5	1.00	1.24
R_6	0.98	1.28

The follow up-time for small car and motorized two-wheeler is plotted against the circulating traffic flow. The variation of follow-up time with circulating traffic flow on five roundabouts is shown in Figure 5.8 to Figure 5.12. It can be seen that with an increase in the circulating traffic flow the estimated value of follow-up time remains more or less constant. The best fit relationship between follow-up time and circulating traffic flow has been examined and exponential function has been found to be the best out of many distributions. Based on the statistical parameters, it has been concluded that the relationship between the follow-up time and circulating traffic flow is not significant. The follow-up time value for motorised two-wheelers is around 25% lower than that of small cars.



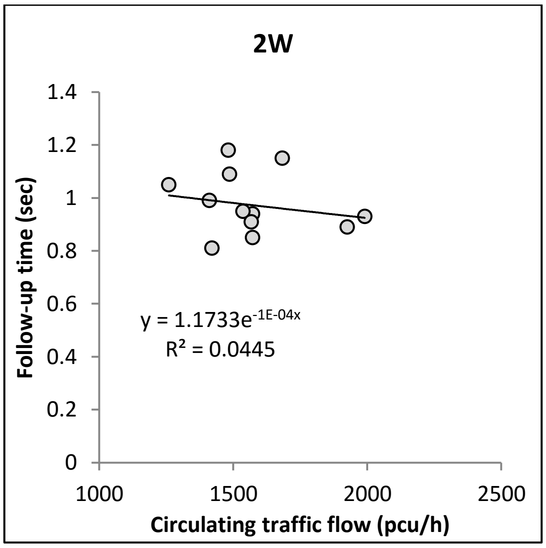


Figure 5.8 Follow up time v/s circulating traffic flow at R_1

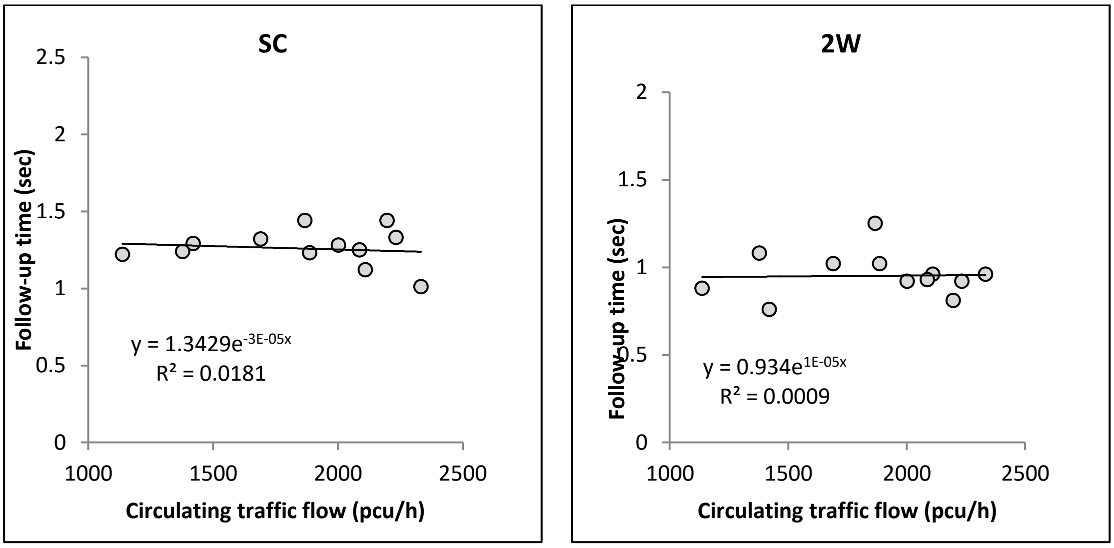


Figure 5.9 Follow up time v/s circulating traffic flow at R_2

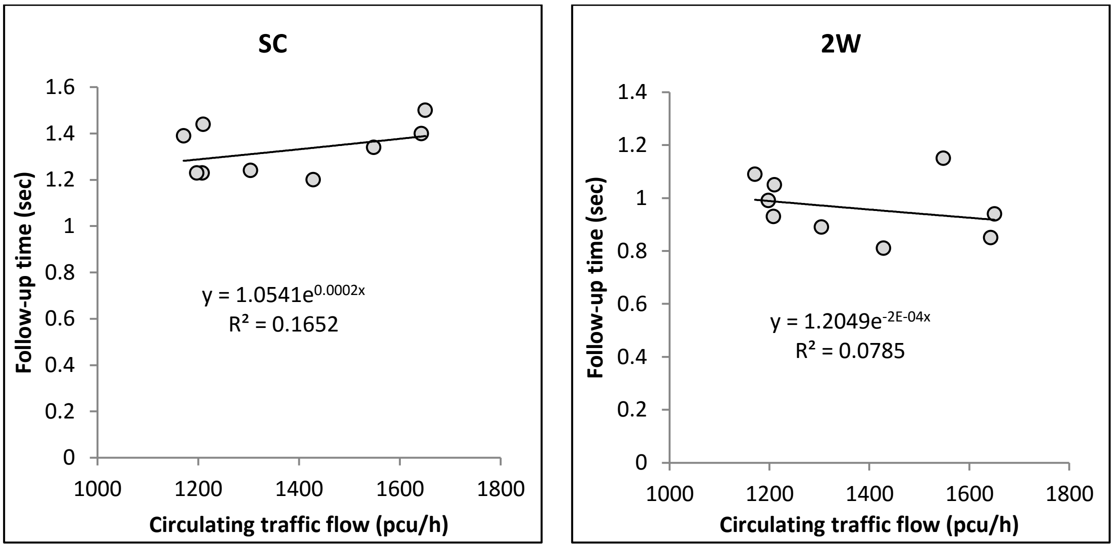


Figure 5.10 Follow up time v/s circulating traffic flow at R_3

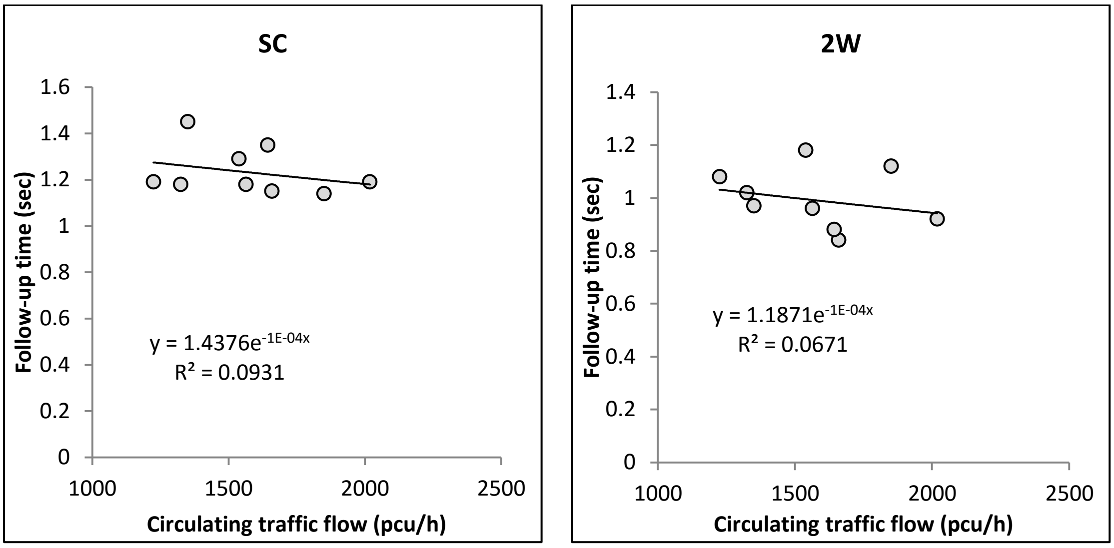


Figure 5.11 Follow up time v/s circulating traffic flow at R_5

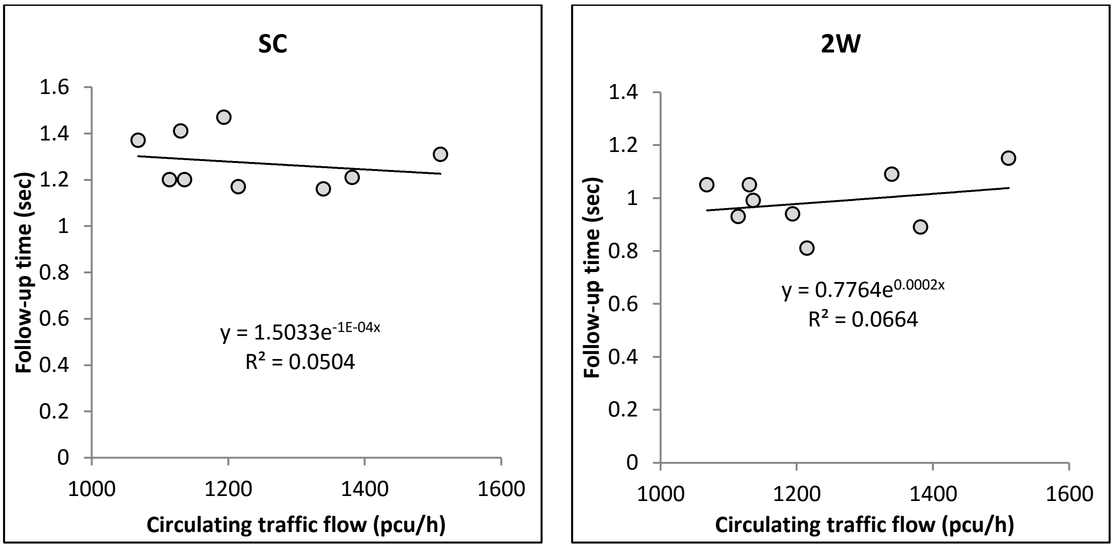


Figure 5.12 Follow up time v/s circulating traffic flow at R_6

5.4 CRITICAL GAP VERSUS FOLLOW-UP TIME

The follow-up time, critical gap and ratio of follow-up time to critical gap are given in Table 5.11 for motorized two-wheeler and small car. The average ratio of follow-up time to critical gap is 0.64. The literature suggests this to be 0.60 or more mainly for developed countries (Brilon 1988; Hagring et al. 2003; Tian et al. 2000). Therefore, the follow-up time has been suggested as ‘0.64 times the critical gap’ for all vehicles.

Table 5.11 The follow-up time, critical gap and ratio

Roundabout ID	**Follow up time**		**Critical gap**		**Ratio= t_f/t_c**	
	2W	**SC**	**2W**	**SC**	**2W**	**SC**
R_1	0.99	1.41	1.60	2.30	0.62	0.61
R_2	0.95	1.30	1.50	2.11	0.63	0.62
R_3	0.97	1.33	1.48	2.08	0.66	0.64
R_5	1.00	1.24	1.55	1.85	0.65	0.67
R_6	0.98	1.28	1.59	1.97	0.62	0.65

5.5 SUMMARY

The critical gap is usually used as a major parameter in the estimation of entry capacity at roundabouts. Any bias in the estimation of critical gap will result in biased capacity value. Several methods are in use for the determination of the critical gap of a vehicle. But all of these methods are based on strict adherence of rule of priority and homogeneous traffic flow conditions. When a traffic stream has variety of vehicles, the concept of 'follow the leader' and the rules of priority are often violated. Small size vehicles tend to use very small gaps (or lags) due to their smaller size and better maneuverability. Many of times, under very heavy flows or free flow conditions, no gaps are rejected. Maximum likelihood method, which is advocated as the most reliable method, can also lead to very trivial results in the estimation of critical gap in such situations. To deal with this situation, a new method based on minimizing the sum of absolute difference in a gap with respect to the accepted and rejected gaps is proposed in this study. This method is more suitable than any other method available in literature as it can be applied to the situations where rule of priority is not respected. It does not make prior assumptions regarding the distribution function of critical gap and the driver behaviors, and does not fail even when there are very few rejected gaps, which is very common under mixed traffic conditions of developing countries. Moreover, the proposed method can be used for estimating the critical gaps even with smaller number of observations of entering vehicles. The present study showed that the optimum number of data set for the estimation of critical gap is 45 for two-wheelers and three-wheelers, and 60 for small cars.

The critical gap and follow-up-time are found to be different than those adopted by other countries. The estimated critical gap and follow-up time values for motorized two-wheeler are around 24% and 25% lower than that of small car, respectively. Based on the statistical parameters, it has been concluded that the critical gap and follow-up time are not varying much with the change in circulating traffic flow. The average ratio of follow-up time to critical gap has been found to be 0.64.

The above discussion clearly highlights the differences between developing countries and developed countries, with regard to the estimated critical gaps and follow-up times for different vehicles on roundabouts. Therefore, the critical gap and follow-up time values as recommended in developed countries cannot be used directly as such in the traffic flow conditions prevailing in developing countries like India.

CHAPTER 6:
ENTRY CAPACITY MODEL

6.0 GENERAL

The estimation of entry capacity of a roundabout under conditions prevailing in developing countries is a tedious process. This is because of heterogeneity in vehicle types having variation in their operational performance and due to the driver behavior, which is highly adaptable towards the prevailing traffic flow on the road rather than being influenced and governed by the geometrics and controls. The entry capacity is considered as a function of circulating flow. The normal notion is that as circulating flow decreases, the entry flow should increase. This may be due to the higher opportunities being made available to the vehicles desiring to enter the circulation area. The entry traffic flow on an approach and the corresponding circulating traffic flow are extracted for a period of queue dissipation and are extrapolated to the equivalent hourly flow. The linear and non-linear relationship between the two is examined. The influence of roundabout geometrics on entry flows is also examined. This chapter also presents an approach to develop an entry capacity model for roundabouts in developing countries, especially for India. HCM (2010) model is examined and its parameters are calibrated so that the modified model suits traffic condition prevailing in India. Further, a regression model is developed to estimate entry capacity of an approach on a roundabout for Indian traffic flow conditions. The circulating traffic flow versus entry capacity charts have been made with the purpose of comparing the results of the proposed model with the existing regression models used in developing countries. Then, the sensitivity analysis has been done to examine the effect of influencing variables on the entry capacity.

6.1 ENTRY FLOW VERSUS CIRCULATING FLOW

Ten roundabouts out of eleven were considered to examine the relationship between entry flow and circulating flow. One roundabout (ID - R_7) was used for validation purpose. To estimate the field entry capacity, the maximum number of vehicles on an approach while having a stable queue and the corresponding circulating traffic flow were extracted for a period of queue dissipation and were extrapolated to the equivalent hourly flow. First of all a correlation analysis was performed to perceive the correlations of circulating traffic flow and exiting traffic flow with entry traffic flow during the queue formation in the approach. Correlation analysis is a statistical technique that can show whether and how strongly pairs of variables (independent and dependent) are related. The results of the correlation analysis for the ten roundabouts are given in Table 6.1. Based on the correlation coefficients, it has been concluded that the correlation between entry traffic flow and circulating traffic flow is high, whereas its correlation with exiting traffic flow is low. Still one observation can be made here. The increase in circulating traffic flow would increase the propensity of increase in exiting traffic flow. This looks to be obvious. But on increase in the entry traffic flow would reduce the exiting traffic flow. This could not be reasoned out. Probably more information is needed to comment on this phenomenon. Therefore, exiting traffic flow has been taken out from the analysis and the circulating traffic flow has been considered.

Table 6.1 Correlation coefficient between the variables

Intersection ID		**Entry traffic flow**	**Circulating traffic flow**	**Exiting traffic flow**
R$_1$	Entry traffic flow	1.000	-0.885	-0.294
	Circulating traffic flow	-0.885	1.000	0.348
	Exiting traffic flow	-0.294	0.348	1.000
R$_2$	Entry traffic flow	1.000	-0.829	-0.253
	Circulating traffic flow	-0.829	1.000	0.332
	Exiting traffic flow	-0.253	0.332	1.000
R$_3$	Entry traffic flow	1.000	-0.888	-0.247
	Circulating traffic flow	-0.888	1.000	0.165
	Exiting traffic flow	-0.247	0.165	1.000
R$_4$	Entry traffic flow	1.000	-0.892	-0.204
	Circulating traffic flow	-0.892	1.000	0.160

	Exiting traffic flow	-0.204	0.160	1.000
R_5	Entry traffic flow	1.000	-0.880	-0.281
	Circulating traffic flow	-0.880	1.000	0.149
	Exiting traffic flow	-0.281	0.149	1.000
R_6	Entry traffic flow	1.000	-0.862	-0.157
	Circulating traffic flow	-0.862	1.000	0.134
	Exiting traffic flow	-0.157	0.134	1.000
R_8	Entry traffic flow	1.000	-0.842	-0.201
	Circulating traffic flow	-0.842	1.000	0.115
	Exiting traffic flow	-0.201	0.115	1.000
R_9	Entry traffic flow	1.000	-0.871	-0.265
	Circulating traffic flow	-0.871	1.000	0.216
	Exiting traffic flow	-0.265	0.216	1.000
R_{10}	Entry traffic flow	1.000	-0.845	-0.298
	Circulating traffic flow	-0.845	1.000	0.347
	Exiting traffic flow	-0.298	0.347	1.000
R_{11}	Entry traffic flow	1.000	-0.882	-0.249
	Circulating traffic flow	-0.882	1.000	0.172
	Exiting traffic flow	-0.249	0.172	1.000

Further, the extracted entry traffic flow from an approach and circulating traffic flow during the period of stable queue were used to plot scatter plots between the two. To formulate the relationships, various types of curves have been examined (Refer Appendix B). The two relationships, linear and exponential, are found to have adequately represented the relation between entry capacity and circulating traffic flow, whereas, other functional forms show inadequate results because entry capacity tends to infinity at low value of circulating traffic flow. The linear and exponential relationships for the ten roundabouts are presented in Figure 6.1 to Figure 6.10. The linear and exponential functional forms for all ten roundabouts are given in Table 6.2 and Table 6.3 respectively.

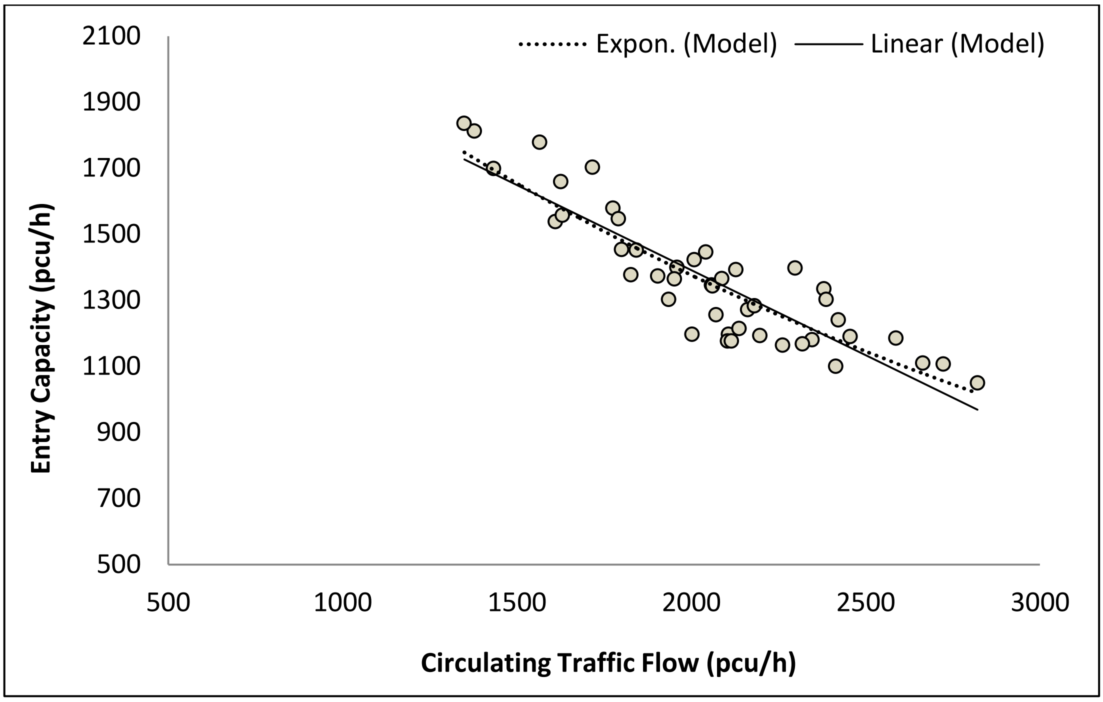


Figure 6.1 Entry flow versus circulating flow at R_1

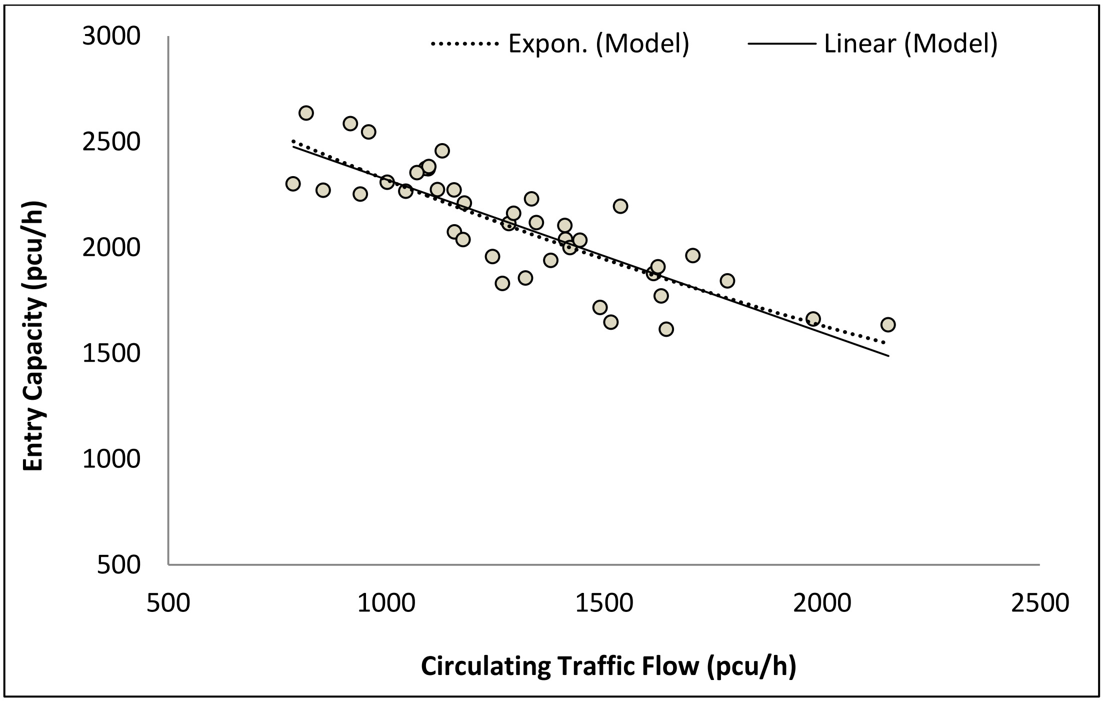


Figure 6.2 Entry flow versus circulating flow at R_2

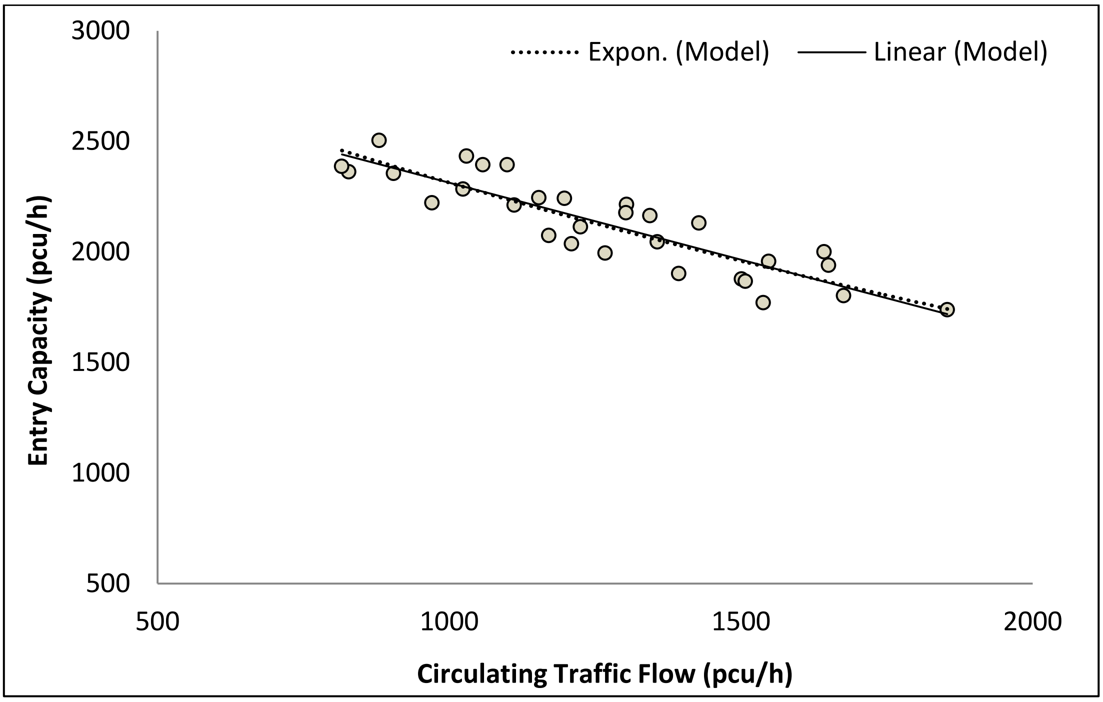


Figure 6.3 Entry flow versus circulating flow at R_3

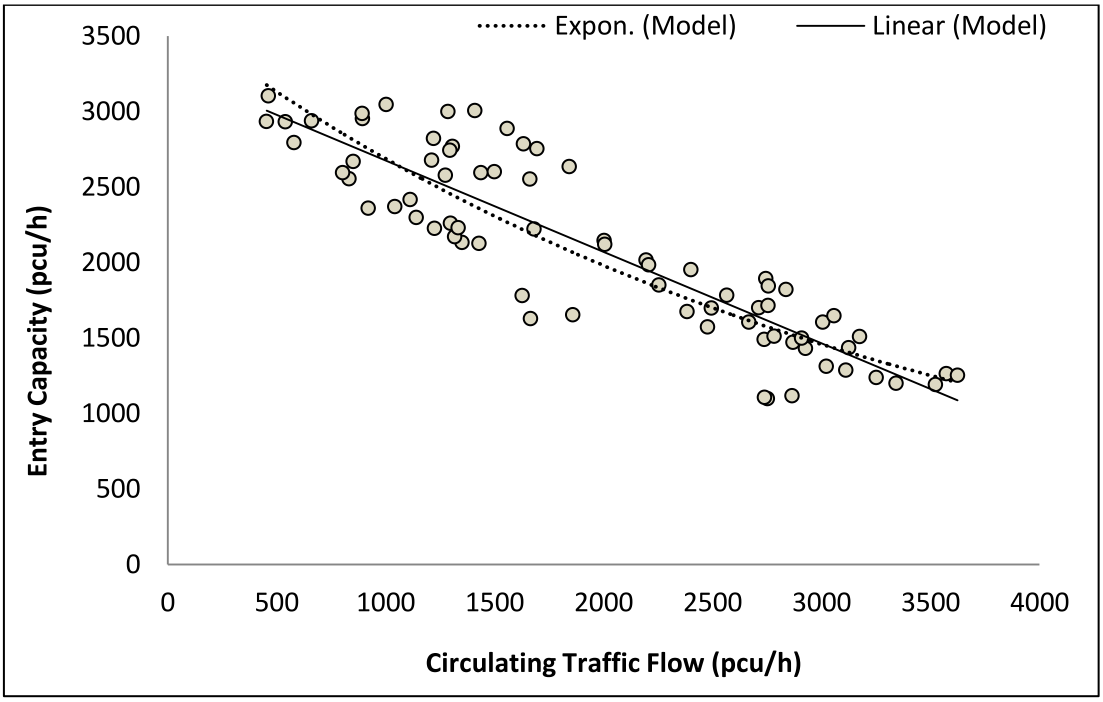


Figure 6.4 Entry flow versus circulating flow at R_4

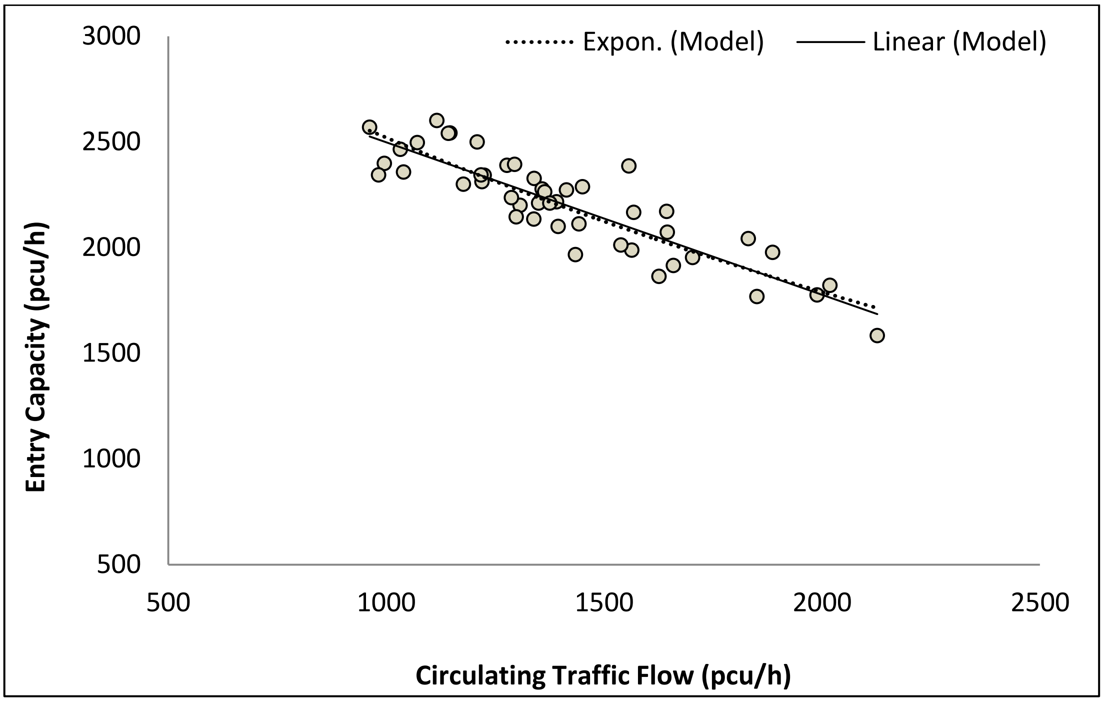


Figure 6.5 Entry flow versus circulating flow at R_5

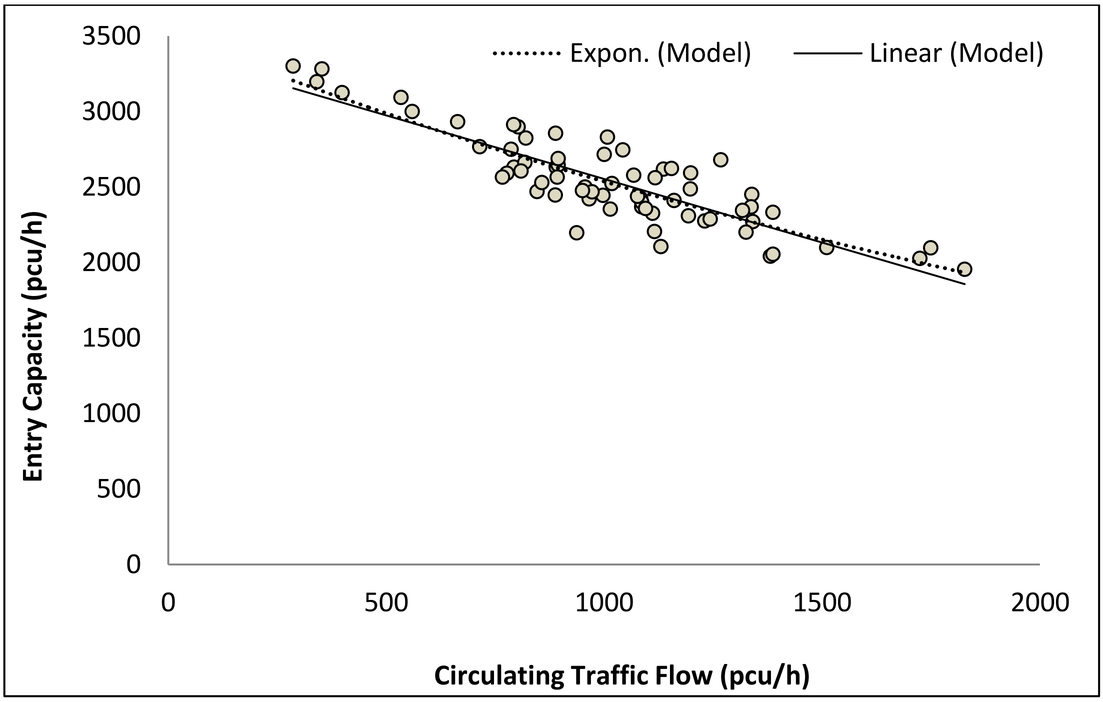


Figure 6.6 Entry flow versus circulating flow at R_6

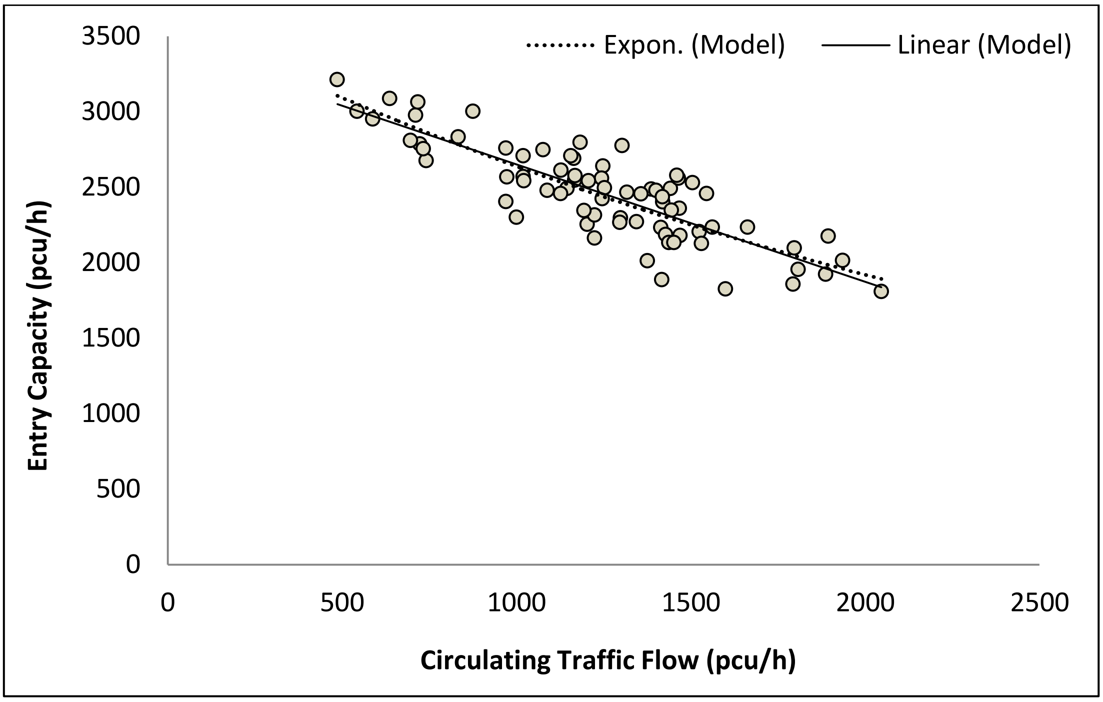


Figure 6.7 Entry flow versus circulating flow at R_8

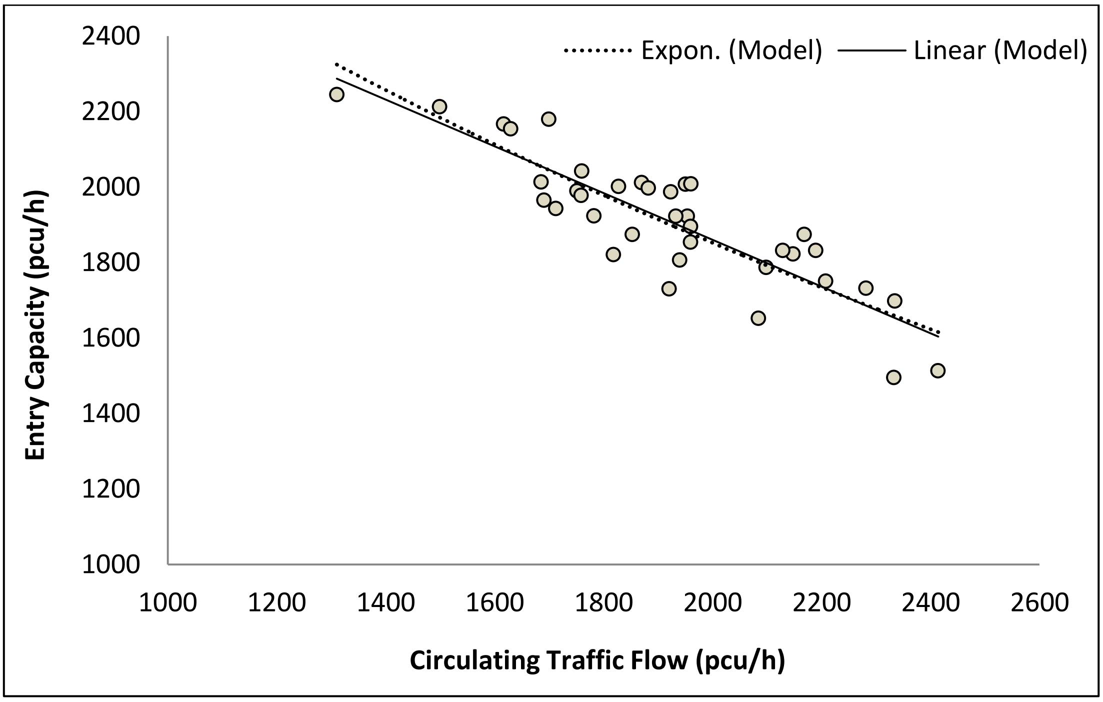


Figure 6.8 Entry flow versus circulating flow at R_9

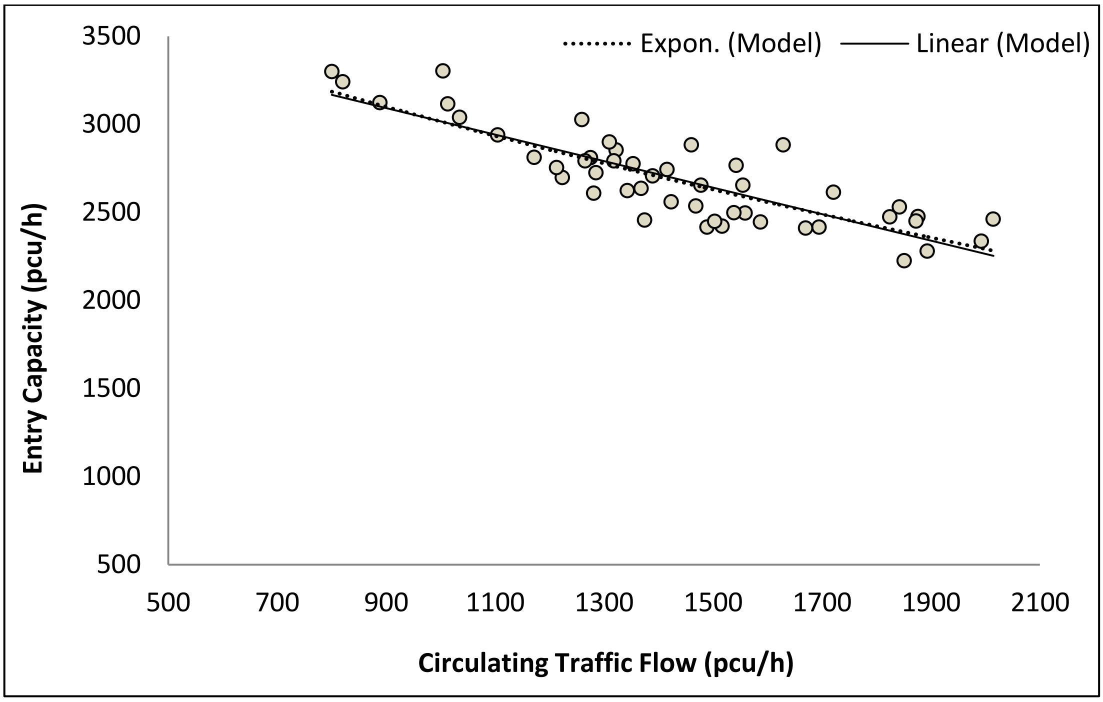


Figure 6.9 Entry flow versus circulating flow at R_{10}

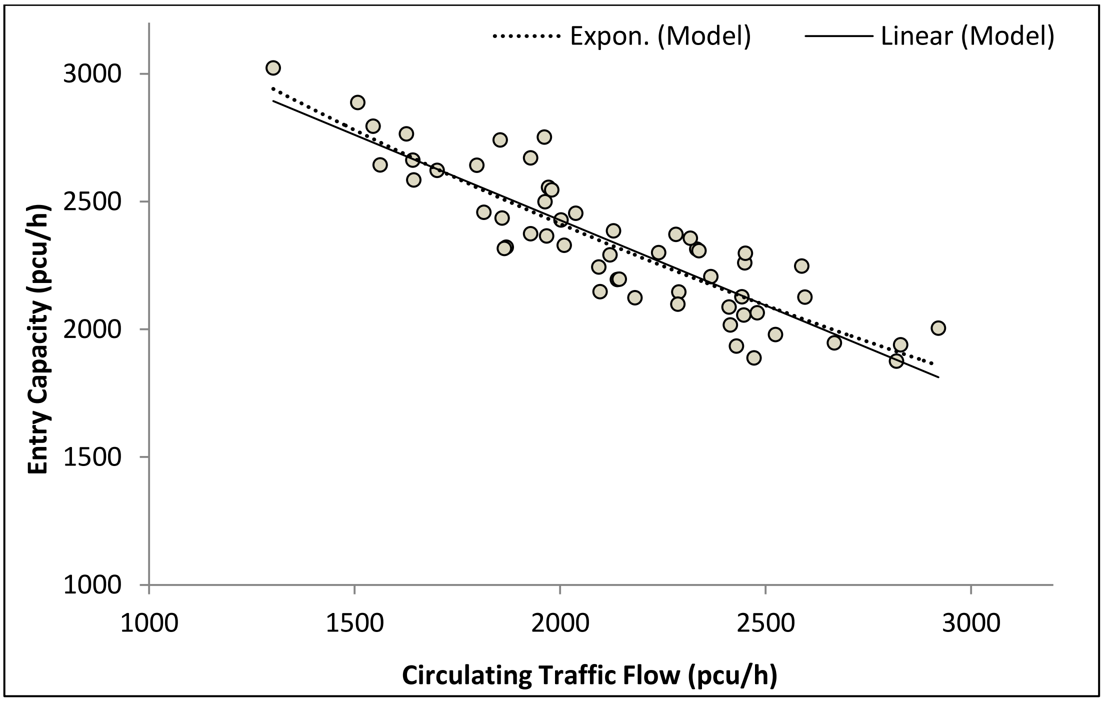


Figure 6.10 Entry flow versus circulating flow at R_{11}

Table 6.2 Linear model relating entry capacity to circulating flow

Intersection ID	Model	R^2	$DF_{regression}$	$DF_{residual}$	F Value	Significance F (p-value)
R_1	$-0.5144*Q_c + 2420.0$	0.783	1	43	154.87	7.60E-16
R_2	$-0.7244*Q_c + 3045.8$	0.687	1	39	85.47	2.25E-11
R_3	$-0.6963*Q_c + 3010.2$	0.788	1	28	104.07	6.19E-11
R_4	$-0.6053*Q_c + 3280.0$	0.796	1	72	280.46	1.54E-26
R_5	$-0.7216*Q_c + 3220.2$	0.775	1	44	151.46	7.64E-16
R_6	$-0.8428*Q_c + 3395.9$	0.742	1	65	187.27	8.23E-21
R_8	$-0.7776*Q_c + 3428.9$	0.709	1	72	175.37	5.59E-21
R_9	$-0.6195*Q_c + 3098.6$	0.757	1	35	109.12	2.67E-12
R_{10}	$-0.7523*Q_c + 3768.2$	0.715	1	46	115.25	4.06E-14
R_{11}	$-0.6676*Q_c + 3762.7$	0.779	1	53	186.29	5.52E-19

Table 6.3 Exponential model relating entry capacity to circulating flow

Intersection ID	Model	R^2	$DF_{regression}$	$DF_{residual}$	F Value	Significance F (p-value)
R_1	$3009.0*e^{-0.00039*Qc}$	0.813	1	43	186.91	2.96E-17
R_2	$3336.6*e^{-0.00036*Qc}$	0.697	1	39	89.55	1.19E-11
R_3	$3215.5*e^{-0.00033*Qc}$	0.787	1	28	103.36	6.67E-11
R_4	$3573.2*e^{-0.00029*Qc}$	0.784	1	72	261.60	1.12E-25
R_5	$3514.3*e^{-0.00033*Qc}$	0.770	1	44	147.43	1.21E-15
R_6	$3561.8*e^{-0.00034*Qc}$	0.760	1	65	205.38	8.55E-22
R_8	$3617.8*e^{-0.00031*Qc}$	0.711	1	72	176.98	4.42E-21
R_9	$3523.2*e^{-0.00032*Qc}$	0.751	1	35	105.77	4.05E-12
R_{10}	$4033.6*e^{-0.00029*Qc}$	0.732	1	46	125.66	9.52E-15
R_{11}	$4303.7*e^{-0.00029*Qc}$	0.789	1	53	198.68	1.44E-19

The coefficients of determination for all roundabouts are found varying between 0.697 and 0.813 for exponential models, and between 0.687 and 0.796 for linear models. The coefficients of determination for the two types of model can be termed as satisfactory and above. A linear relationship may provide a reasonable estimate and easier prediction. Based on the *Significance F* (p-value) and coefficients of determination, the performance of the exponential form is found to be the best for maximum number of roundabouts. Apart from this, if the roundabout is not locked then a minimum amount of flow will always enter the circulating traffic stream. This norm also supports the adoption of exponential relationship. Earlier models too indicated that a non-linear relationship between the circulating traffic flow and the entry capacity is the best (Al-Masaeid and Faddah 1997; HCM 2000, 2010; Pollatschek et al. 2002).

6.1.1 Entry Flow Model

Once the relationship has been ascertained between entry flow and circulating traffic flow, it was decided to come up with minimal number of relationships so that implementation agencies do not find any problem. Keeping under consideration that all the roundabouts have varying physical features, especially the central-island diameter, it was decided to have these relationships for roundabouts with central-island diameter as 25 m to 40 m, 41 m to 60 m and 61 m to 90 m and these are termed as small, medium and large sized roundabouts, respectively. These are now given below:

a) For small size roundabouts

$$Q_e = 3252 * e^{-0.00037 * Q_c} \tag{6.1}$$

b) For medium size roundabouts

$$Q_e = 3483 * e^{-0.00030 * Q_c} \tag{6.2}$$

c) For large size roundabouts

$$Q_e = 3843 * e^{-0.00024 * Q_c} \tag{6.3}$$

The statistical characteristics of the above models are given in Table 6.4. 't' statistics for all of the coefficients in the models show that all coefficients are statistically significant at 95 percent level of confidence (being outside the critical value range of ± 1.98). The coefficient of determination (R^2) for equations (6.1), (6.2) and (6.3) are 0.848, 0.800 and 0.811 respectively, which also indicate towards good predictive strength of the model. *Significance F* (p-value) of the models is much less than 0.05, which signifies that the regression output for the models is not merely a chance occurrence.

Table 6.4 Regression parameter estimates

Round-about	**Parameter**	**Estimate**	**Std. Error**	**t Stat**	**P-value**	**Lower Bound**	**Upper Bound**
Small	a	3252	27.36	118.8	2.34E-121	3198	3306
	b	0.00037	0.00002	21.8	1.55E-42	0.00034	0.00040
Medium	a	3482	45.34	76.8	1.60E-197	3393	3571
	b	0.00030	0.00001	31.7	2.78E-97	0.00028	0.00032
Large	a	3843	79.31	48.5	9.66E-72	3686	4000
	b	0.00024	0.00001	20.6	5.42E-38	0.00022	0.00026

Analysis of Variance

Round-about		**Sum of Squares**	**DF**	**Mean Squares**	**F**	***Significance F* (p-value)**
Small	**Regression**	1.92E+07	1	1.92E+07	921.27	1.92E-56
	Residual	2.376E+06	114	2.08E+04		
	Total	2.157E+07	115			
Medium	**Regression**	4.65E+07	1	4.65E+07	1187.706	1.27E-105
	Residual	1.158E+07	296	3.91E+04		
	Total	5.804E+07	297			
Large	**Regression**	8.525E+06	1	8.53E+06	432.14	2.86E-38
	Residual	1.99E+06	101	1.97E+04		
	Total	1.052E+07	102			

6.1.2 Validation of model

The proposed model has been validated at roundabout R_7 (central-island diameter 50 m, circulating roadway width 10 m and entry width 14.7 m). This roundabout was not considered while developing the entry capacity model for an approach. Roundabout R_7 comes under medium size roundabout category. Therefore, equation (6.2) for medium size roundabouts has been selected for validation purpose. The field capacity values of the selected roundabout have been compared with those predicted by the model proposed in this chapter. These values have been plotted against each other, as shown in Figure 6.11. Considering the linear model between field and predicted entry flow values and keeping the constant as zero (which should be the case), it is found that the R^2 value for the plot based on proposed entry capacity model is 0.804. This indicates its adequacy and good agreement between the two values.

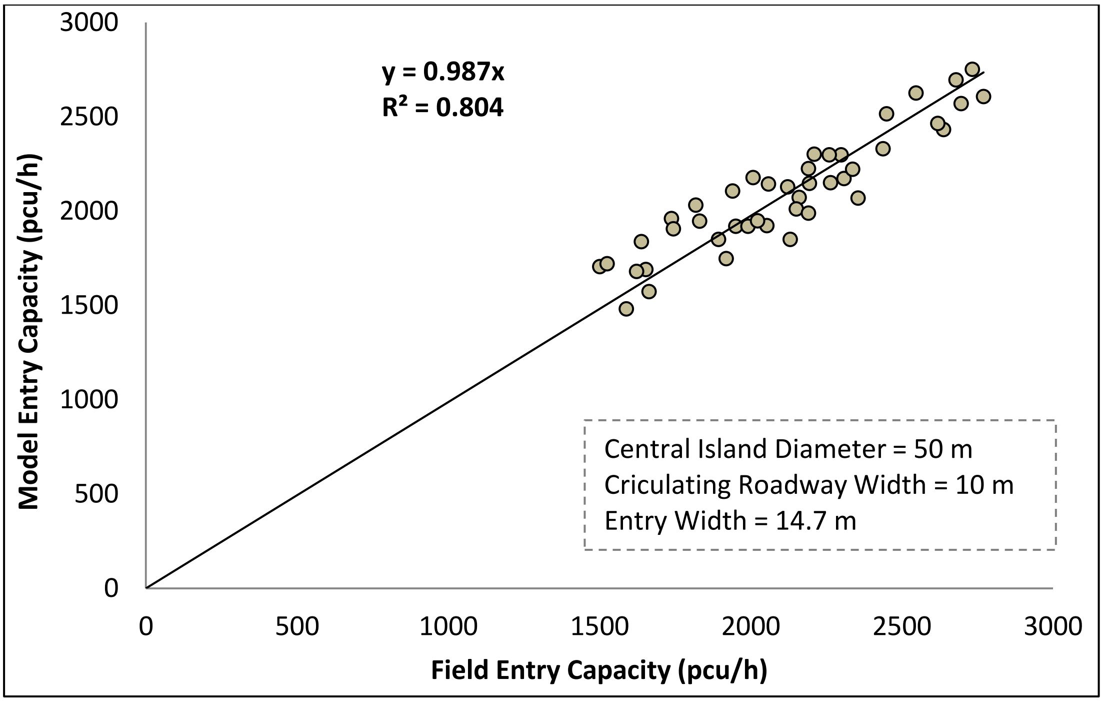


Figure 6.11 Field entry capacity versus model entry capacity

Z-test analysis has been carried out to see whether there is any similarity between the mean of field capacity values and those predicted by the proposed model. The null hypothesis (H_0) was: *There is no significant difference between the mean of field capacity values and those predicted by the proposed model*. The field capacity values and those predicted by the proposed model are normally distributed (Refer Appendix C). The results of z-test analysis are given in Table 6.5. It shows that the p-value > 0.01, and z < 2.58. Therefore, the null hypothesis is not rejected at the 99% level of confidence and there is no significant difference between the means of both capacity values.

Table 6.5 Z-test: comparison of field capacity and model capacity

	Field Capacity	**Model Capacity**
Mean	2115	2097
Variance	123151	95642
Observations	43	
Hypothesized Mean Difference	0	
z	0.25	
P(Z<=z) two-tail	0.80	
z Critical two-tail	2.58	

6.2 ENTRY CAPACITY VERSUS PHYSICAL FEATURES

To find relation between entry capacity and geometric parameters, entry capacities must be extracted at the same circulating traffic flow for all roundabouts. It has been found that the entry capacities have been estimated at different circulating traffic flow for all roundabouts. For the comparison purpose, the entry capacity tried to estimate at zero circulating traffic flow in the field condition. But, field entry capacity could not be estimated since there was no queue formation in the approach leg of a roundabout at zero circulating traffic flow for all roundabouts. Therefore, entry capacities for ten roundabouts have been estimated at zero circulating traffic flow (indicating the possibility of maximum traffic flow that can enter the circulating road) using the developed exponential relationship and plotted against central-island diameter, circulating roadway width and entry width at approach leg. These are shown in Figure 6.12, Figure 6.13 and Figure 6.14 respectively. To formulate the relationships, various types of curves have been examined and the one which gives best results based on the statistical parameters is reported. Power function is found to provide the best fit between entry capacity of approach leg and central-island diameter, circulating roadway width and entry width at approach leg. The variation or dispersion of data with respect to the estimated one is found to be quite low for entry capacity and central-island diameter and is a bit high for the other two relationships. In all of the cases, it can be noted that with an increase in the independent variable (central-island diameter, circulating roadway width and entry width at approach leg) the estimate of entry capacity tries to stabilize near a value of 3700 pcu/h.

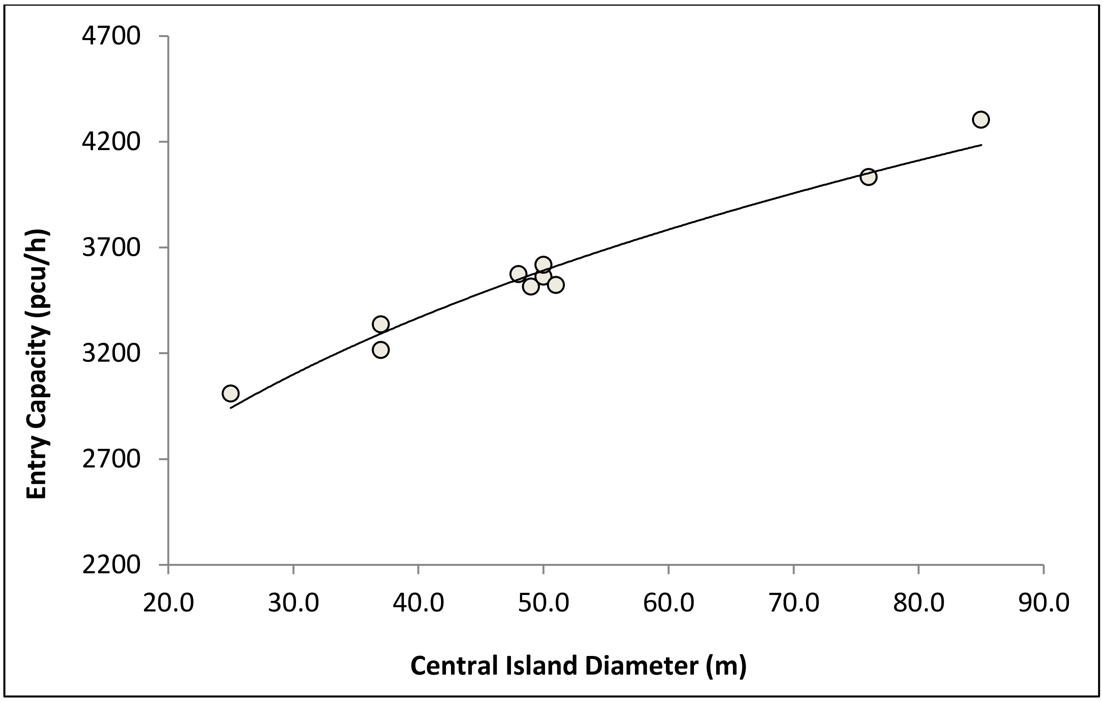


Figure 6.12 Relationship between entry capacity and central island diameter

Analysis of the individual and independent impact of the variables also indicates towards some important points. The entry capacity of the roundabout is highly correlated with the diameter of the central-island. In the case of small sized roundabouts the entry capacity may vary between 2800 pcu/h and 3300 pcu/h. For medium sized roundabouts, the entry capacity may be around 3600 pcu/h and 3700 pcu/h. If the size of the roundabout is large then the values of entry capacity would be in a range of 3900 pcu/h and 4200 pcu/h. The variation in entry capacity value is around 600 pcu/h for a change in central-island diameter from 25 m to 50 m and is 400 pcu /h for change in central-island diameter from 50 m to 75 m.

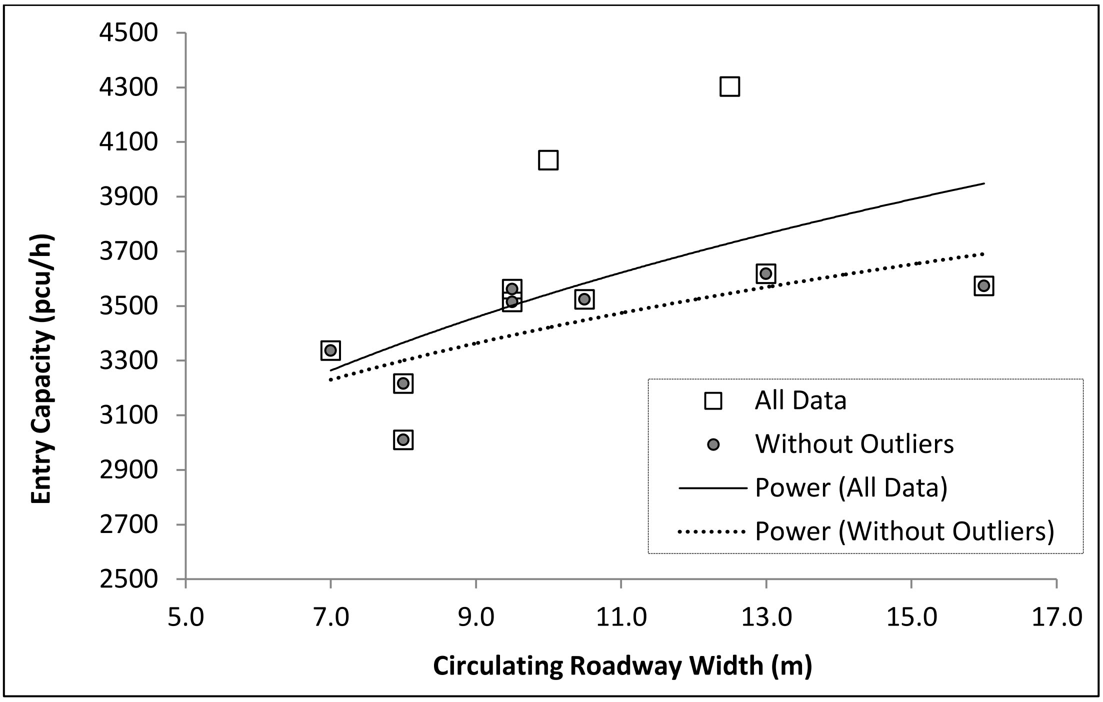


Figure 6.13 Relationship between entry capacity and circulating roadway width

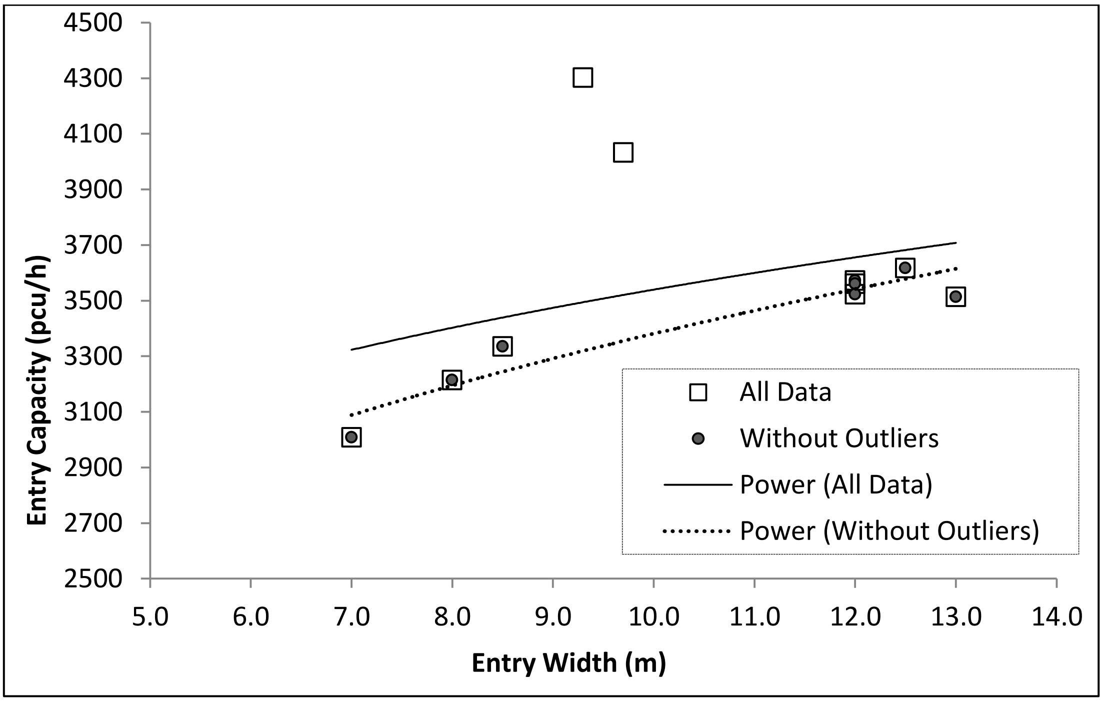


Figure 6.14 Relationship between entry capacity and entry width

Analysis with respect to the circulating roadway width can be made more meaningful if two values (looks like outliers) are not considered. In that case, it probably indicates that when the circulating roadway width increases beyond 9.0 m the entry capacity of the approach leg does not change much and may remain in a narrow range of 3500 pcu/h and 3600 pcu/h. Similar examination of the variation in entry capacity of an approach leg with respect to its width indicates that beyond 11.0 m width, the entry capacity of the approach leg on a roundabout would fluctuate between 3500 pcu/h and 3700 pcu/h.

6.3 CALIBRATION OF HCM (2010) MODEL

HCM (2010) presented gap acceptance model of entry capacity for roundabouts in U.S. However, the direct transferability of the HCM (2010) entry capacity model to Indian traffic flow conditions is doubtful as the manual do not consider mixed traffic flow behavior. Therefore, HCM model has been calibrated based on estimated values of critical gaps under heterogeneous traffic conditions. The entry capacity is estimated from modified HCM model and is compared with the field data taken during formation of a continuous and stable queue at the entry of the roundabout.

The critical gap values could be estimated only on five roundabouts, namely R_1, R_2, R_3, R_5, and R_6. Therefore, these roundabouts have been selected to calibrate the HCM (2010) model for the estimation of entry capacity under heterogeneous traffic condition. The analysis is presented for calibration of HCM model to suit the heterogeneous traffic conditions as prevailing in developing countries like India.

6.3.1 Critical Gap and Follow-up Time

For a multilane roundabout, the critical gap can be calculated using two techniques. One technique considers each entering lane and circulating lane separately. Second technique estimates the critical gap for the entire approach, combining the entering lanes and circulating lanes into single entering and circulating streams, respectively (NCHRP Report-572 2007). In HCM (2010), the critical gap has been given for right lane entry and left lane entry separately based on the first technique since the lane markings are properly positioned in the U.S. roundabouts and vehicles follow the lane discipline. In this study, the second technique is used to estimate the critical gap since the lane markings on these roundabouts are completely absent and vehicles try to find gaps to enter the circulating area, without following the concept of lane.

In a mixed traffic situation, the critical gap (t_c) and follow-up time are different for different types of vehicles. To address this problem, Dahl and Lee (2012) suggested the use of volume weighted average of critical gap values. The same approach is used in this study also and t_c for a mixed traffic stream is calculated using equation (6.4).

$$t_{c,m} = \sum t_{ci} * P_i \tag{6.4}$$

Where, t_{ci} is the critical gap for vehicle type i and P_i is the proportional share (fraction) of vehicle type in the traffic stream on subject approach. $t_{c,m}$ is the critical gap value for mixed traffic stream.

The stream critical gap values are computed using weighted average approach. The traffic composition is used as weight for the respective value of critical gaps to attain the traffic stream value. The estimated critical gap values, percent composition and stream values are given in Table 6.6 for the selected roundabouts. The follow-up time is taken as '0.64 times the critical gap' for all vehicles as discussed in the previous chapter.

Table 6.6 Critical gaps, percent composition and stream values

Roundabout ID	**Critical Gaps, s**					**% Composition**					**Stream Critical Gap, s**
	2W	**3W**	**SC**	**BC**	**HV**	**2W**	**3W**	**SC**	**BC**	**HV**	
R_1	1.60	1.94	2.30	2.39	2.67	42	4	41	12	1	2.00
R_2	1.50	1.88	2.11	2.21	2.55	53	7	36	2	2	1.78
R_3	1.48	1.84	2.08	2.13	2.45	45	4	41	8	2	1.81
R_5	1.55	1.73	1.85	1.92	2.63	40	8	37	10	5	1.77
R_6	1.59	1.68	1.97	2.03	2.52	41	17	33	6	3	1.79

Once stream critical gap values and follow-up time values were estimated, these are used as input parameters in the HCM (2010) equation to estimates entry capacity of a roundabout. This is now discussed in the following sub-section.

6.3.2 Calibration of HCM (2010)

The HCM (2010) suggests the use of equation (3.3) to estimate entry capacity (Q_e) at roundabouts. This is replicated here for better readability as equation (6.5).

$$Q_e = A * e^{-B*Q_c} \tag{6.5}$$

Where,

Q_c = circulating traffic flow (pcu/h)

$$A = \frac{3600}{t_f} \tag{6.6}$$

$$B = \frac{t_c - 0.5 * t_f}{3600} \tag{6.7}$$

Where,

t_f = follow-up time (s)

t_c = critical gap (s)

The HCM (2010) equation for estimating the entry capacity was modified for its adaptation to heterogeneous traffic conditions. Parameters A and B of the HCM equation were calculated based on critical gap values obtained from the field data. The values of the HCM parameters under heterogeneous traffic conditions for five roundabouts are given in Table 6.7.

Table 6.7 HCM parameters under heterogeneous traffic

Roundabout	**$t_{c,m}$, s**	**t_f, s**	**Parameter**	
			A	**B**
R_1	2.00	1.28	2812	0.00038
R_2	1.78	1.14	3160	0.00034
R_3	1.81	1.16	3108	0.00034

R_5	1.77	1.13	3178	0.00033
R_6	1.79	1.15	3142	0.00034

The modified HCM (2010) equations to estimate entry capacity of a roundabout under heterogeneous traffic are given below.

For Roundabout R_1:

$$Q_e = 2812 * e^{-0.00038 * Q_c} \tag{6.8}$$

For Roundabout R_2:

$$Q_e = 3160 * e^{-0.00034 * Q_c} \tag{6.9}$$

For Roundabout R_3:

$$Q_e = 3108 * e^{-0.00034 * Q_c} \tag{6.10}$$

For Roundabout R_5:

$$Q_e = 3178 * e^{-0.00033 * Q_c} \tag{6.11}$$

For Roundabout R_6:

$$Q_e = 3142 * e^{-0.00034 * Q_c} \tag{6.12}$$

Entry capacity was determined in the field also to see the difference between the modified HCM models and field models. The circulating traffic flow and entry traffic flow data were collected during periods of continuous and stable queuing at the entry leg of the roundabout. Circulating traffic flow and entry capacity were expressed in passenger car units (pcu) by multiplying the volume of individual vehicle types by their estimated pcu values in this research work. The scatter plots of entry capacity and circulating traffic flow at selected roundabouts are shown in Figure 6.15 to Figure 6.19 respectively. Field data yielded the following relations for entry capacity by given equations (6.13) to (6.17) for respective roundabouts.

For Roundabout R_1:

$$Q_e = 3009.0 * e^{-0.00039 * Q_c} \tag{6.13}$$

For Roundabout R_2:

$$Q_e = 3336.6 * e^{-0.00036 * Q_c} \tag{6.14}$$

For Roundabout R_3:

$$Q_e = 3215.5 * e^{-0.00033*Q_c} \tag{6.15}$$

For Roundabout R_5:

$$Q_e = 3514.3 * e^{-0.00033*Q_c} \tag{6.16}$$

For Roundabout R_6:

$$Q_e = 3561.8 * e^{-0.00034*Q_c} \tag{6.17}$$

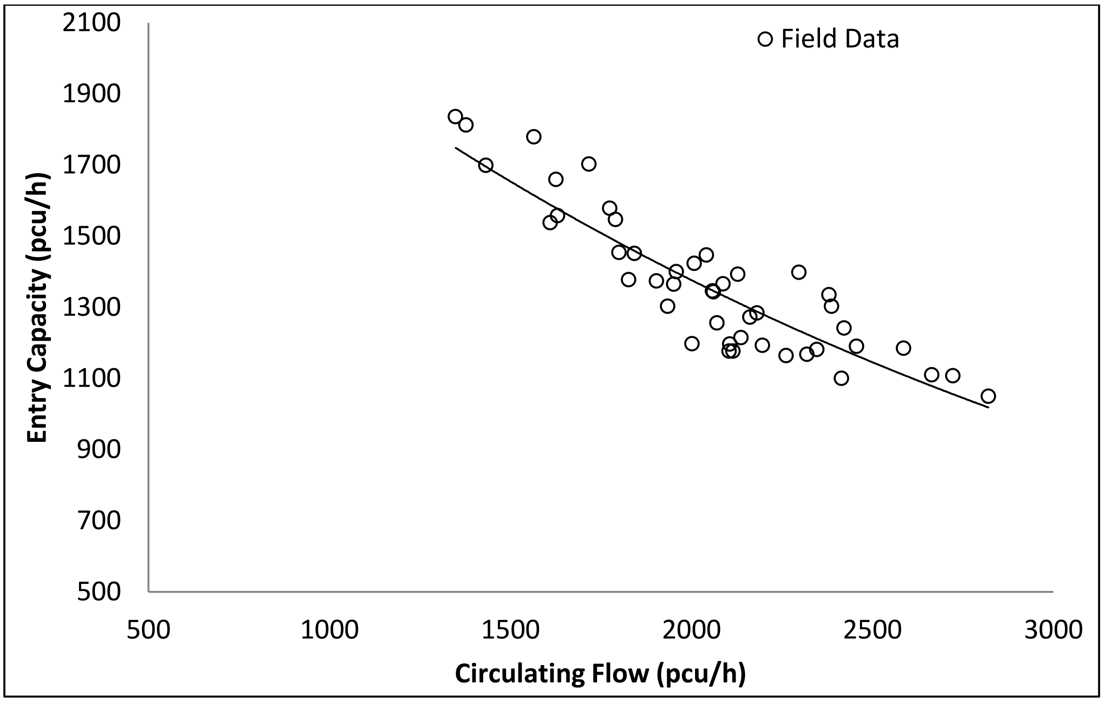


Figure 6.15 Entry capacity versus circulating flow at R_1

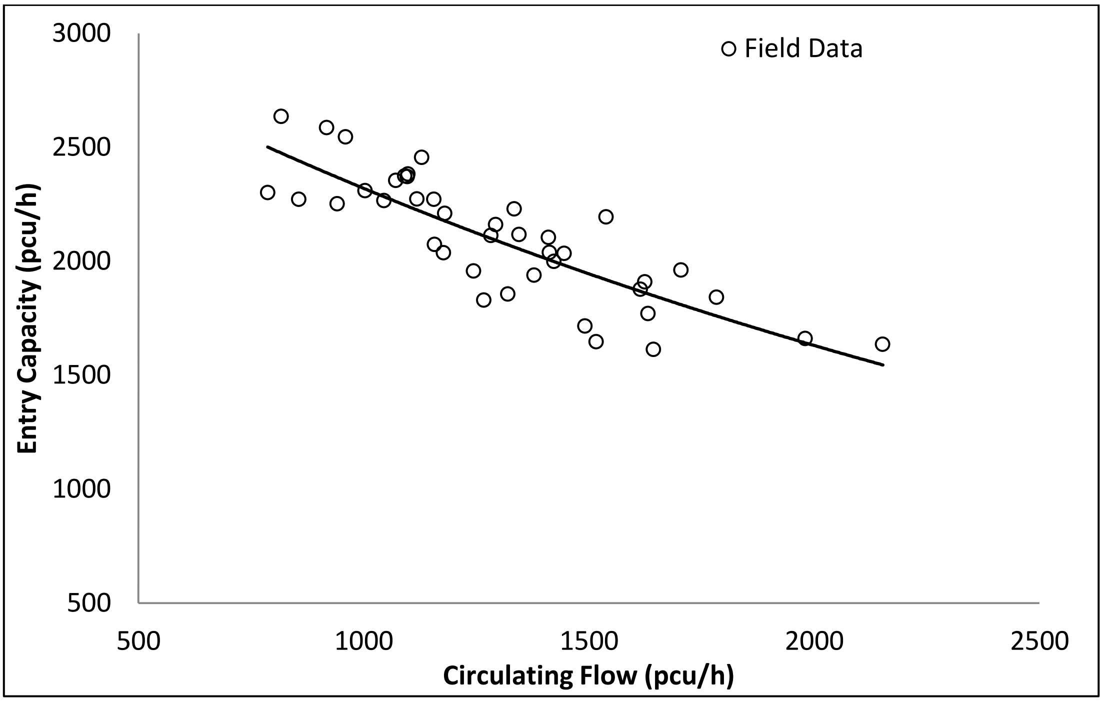


Figure 6.16 Entry capacity versus circulating flow at R_2

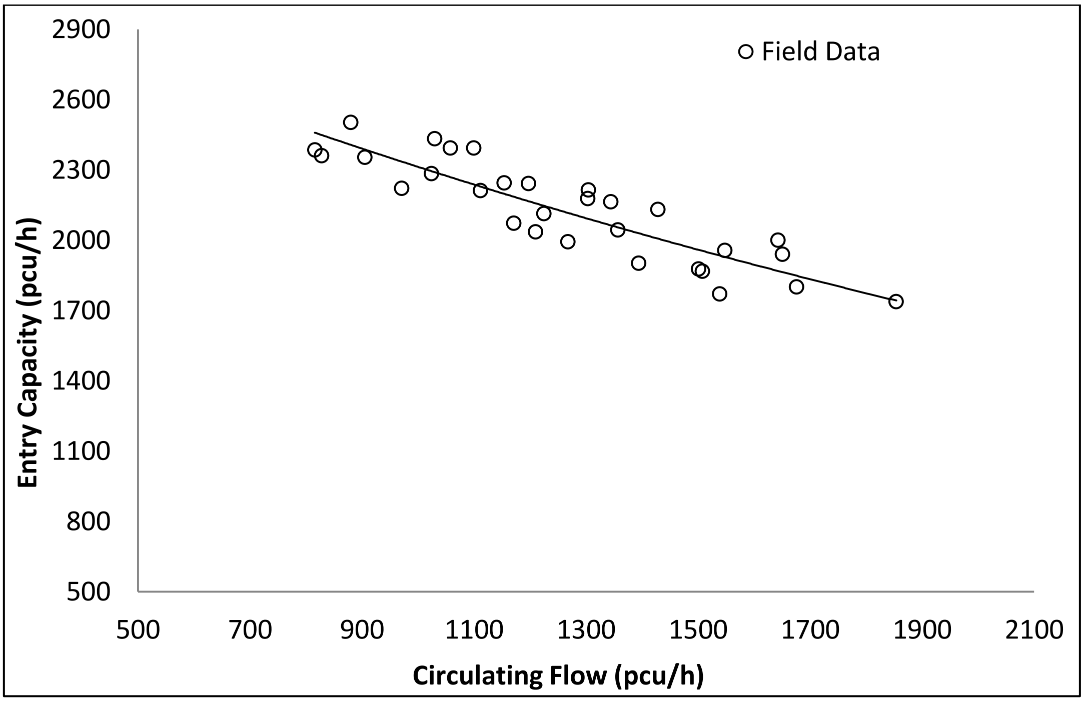


Figure 6.17 Entry capacity versus circulating flow at R_3

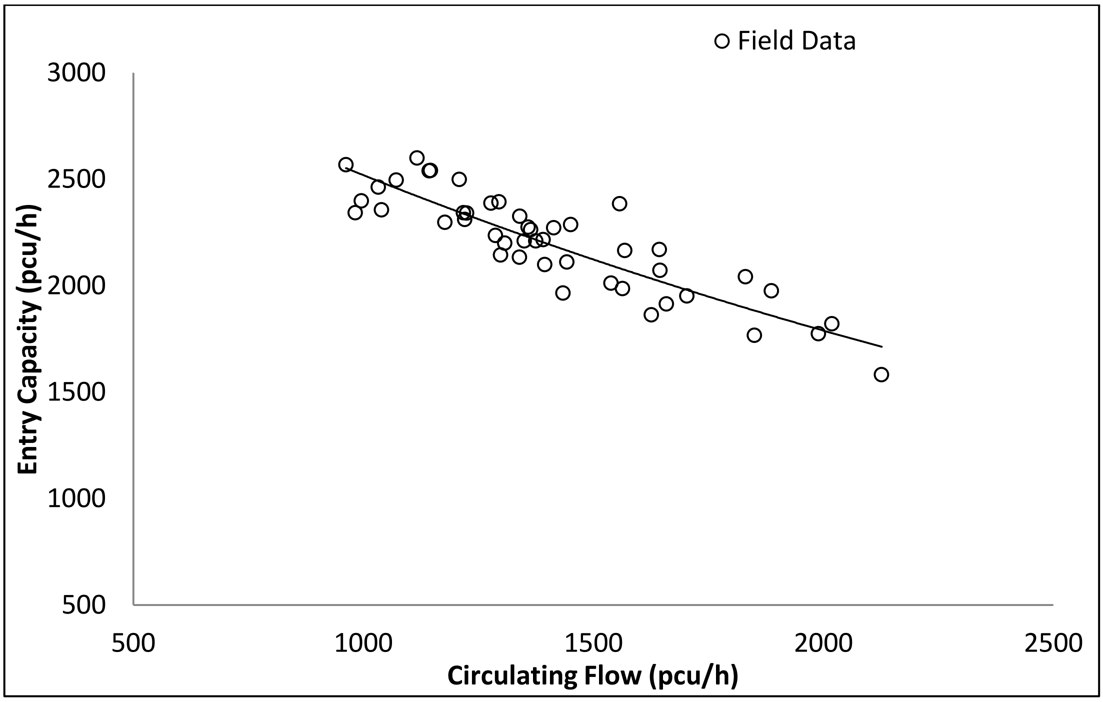


Figure 6.18 Entry capacity versus circulating flow at R_5

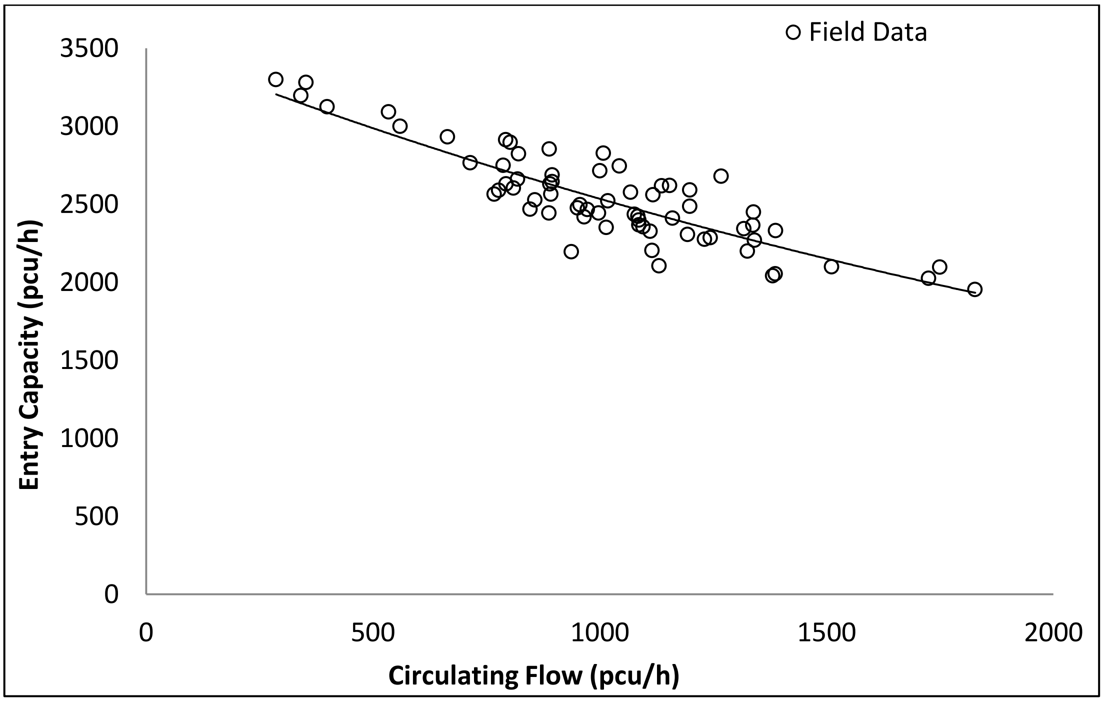


Figure 6.19 Entry capacity versus circulating flow at R_6

The modified HCM (2010) equations (6.8) to (6.12) and field model equations (6.13) to (6.17) are plotted in Figure 6.20 to Figure 6.24 for respective sites. As may be seen, the field model closely follows the modified HCM model at roundabouts R_1, R_2 and R_3. The gap between entry capacity estimated from modified HCM model and the field model for roundabouts R_5 and R_6 are widened. The average gap in two plots between field model and modified HCM model is 60, 50, 100, 225 and 295 pcu/h for roundabouts R_1, R_2, R_3, R_5 and R_6 respectively. The gap between modified HCM model and the field model is increasing with the size of roundabout. This is attributed to the estimated stream critical gap at roundabouts R_2, R_3, R_5 and R_6. The stream critical gap values at these roundabouts are nearly same. It can be inferred that as the stream critical gap value becomes constant, the entry capacity using modified HCM equations does not increase with the increase in the size of the roundabout. However, the field entry capacity increases with the increase in the size of the roundabout. This result is consistent with the findings of previous study, which reported that the entry capacity increases as its diameter increases (Al-Masaeid 1999).

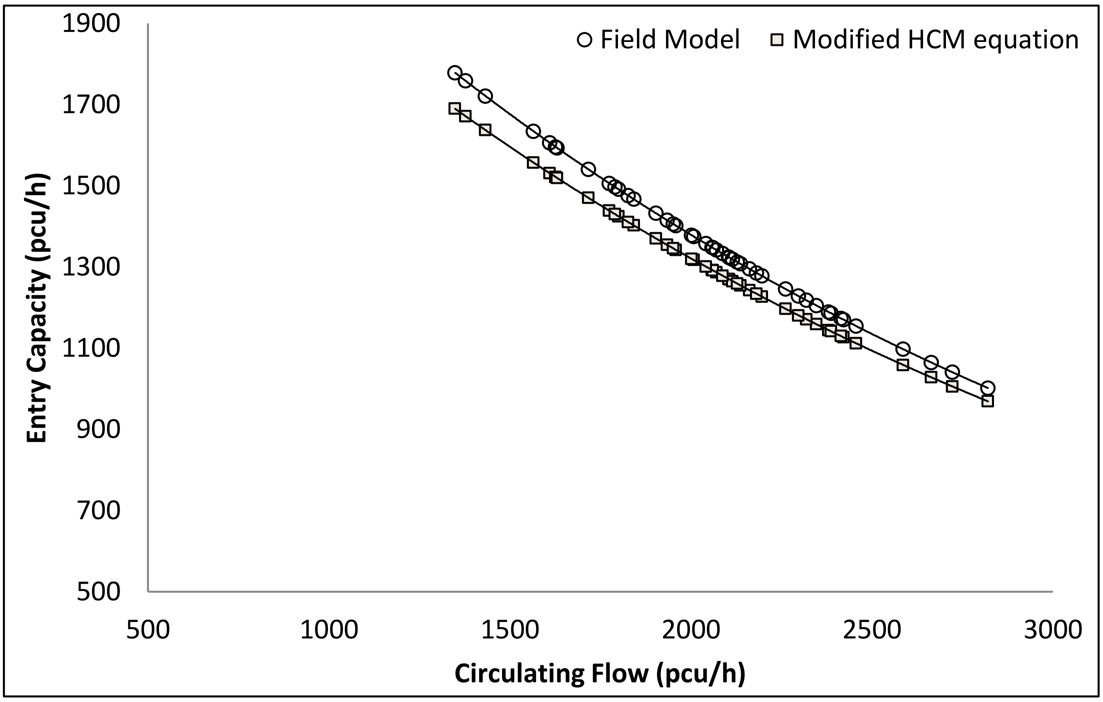


Figure 6.20 Comparison between field and modified HCM model at R_1

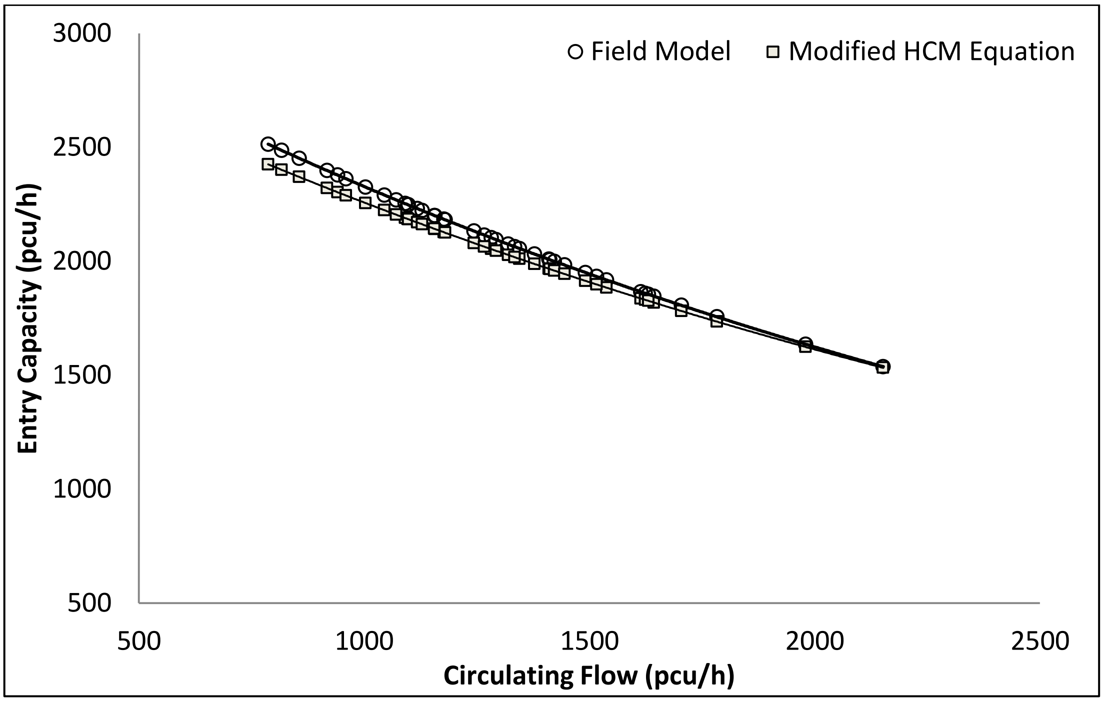


Figure 6.21 Comparison between field and modified HCM model at R_2

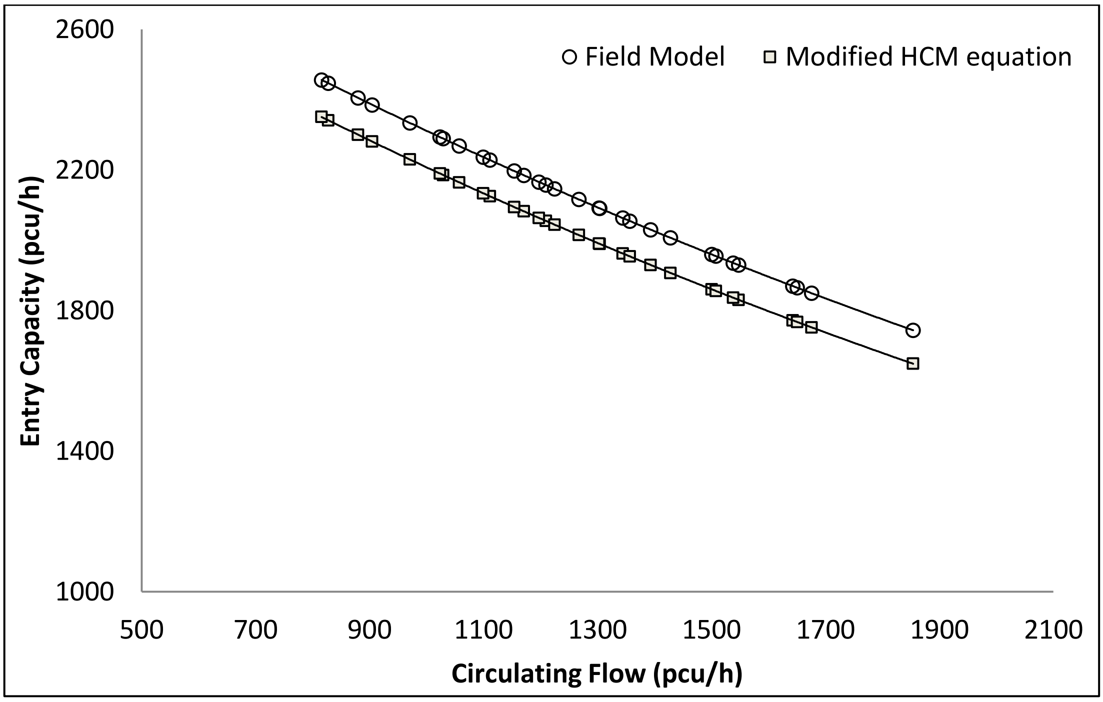


Figure 6.22 Comparison between field and modified HCM model at R_3

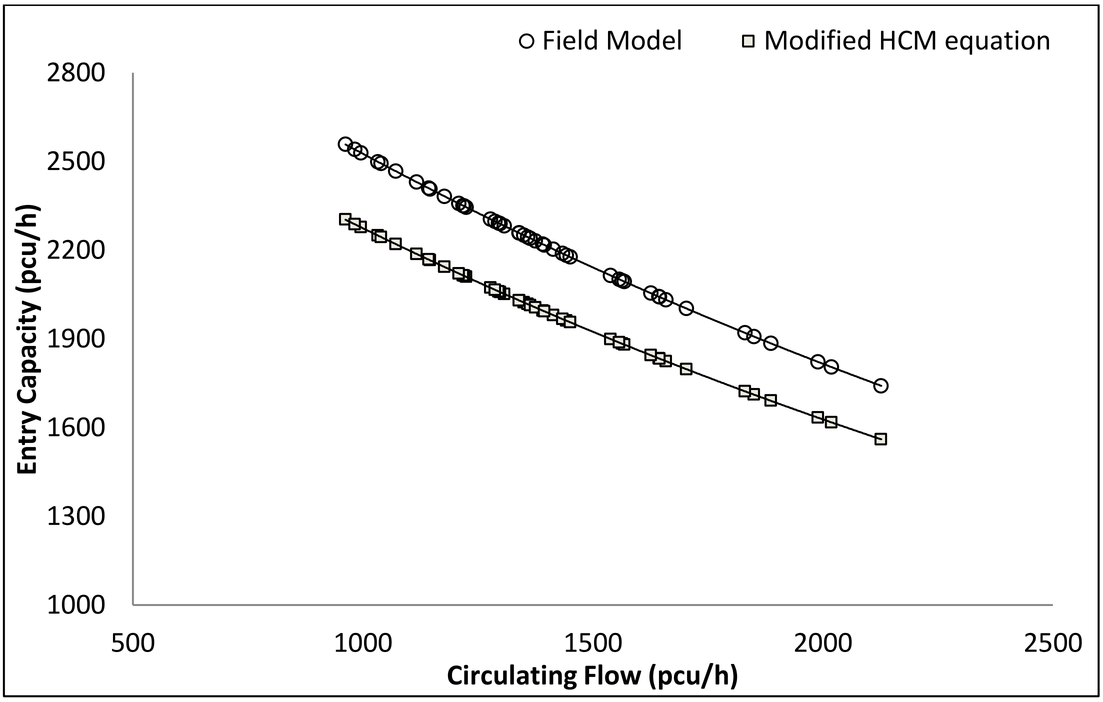


Figure 6.23 Comparison between field and modified HCM model at R_5

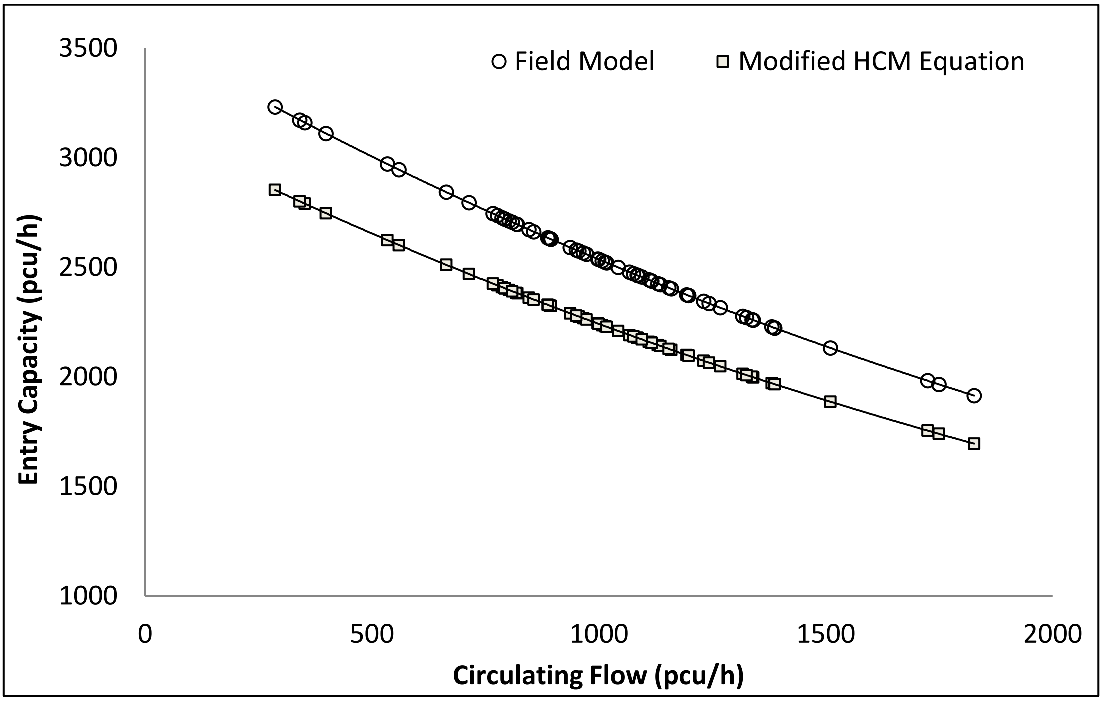


Figure 6.24 Comparison between field and modified HCM model at R_6

6.3.3 Final Calibrated HCM (2010) Model

It may be noted here that the modified HCM models are found differing from the field models. Therefore, there is a need to use a multiplicative adjustment factor in modified HCM models to satisfy the traffic flow condition prevailing in developing countries like India. The stream critical gap values at roundabouts R_5 and R_6 are nearly same as estimated at roundabouts R_2 and R_3. Therefore, roundabouts R_2, R_3, R_5 and R_6 have been grouped together for further analysis. The adjustment factor (f_a) is defined as the ratio between the field entry flow value and that given by the modified HCM model. The final calibrated HCM model and adjustment factors for different size of roundabouts are given in Table 6.8.

Table 6.8 Final calibrated HCM model and adjustment factors for different size of roundabouts

Roundabout ID	**Central island diameter**	**Adjustment factor (f_a)**	**Parameter**		**Entry capacity (Q_e)**
			A	*B*	
R_1	25.0	1.054	2812	0.00038	$Q_e = f_a * A * e^{-B*Q_C}$
R_2	37.0	1.033	3147	0.00034	
R_3	37.0				
R_5	49.0	1.133			
R_6	50.0				

The calibrated HCM models are compared with the field models to see the difference between these models. These are plotted in Figure 6.25, Figure 6.26 and Figure 6.27 for roundabouts of central-island diameter 25 m, 37 m and 50 m respectively. The field entry capacity model for roundabout R_1 is differing by only ±1 percent from the calibrated HCM model. This is ±2 and ±1 percent for roundabouts R_2 and R_3 respectively. The field entry capacity models for roundabouts R_5 and R_6 follows the calibrated HCM model. The difference between field and calibrated HCM model is less than ±1 percent for these roundabouts.

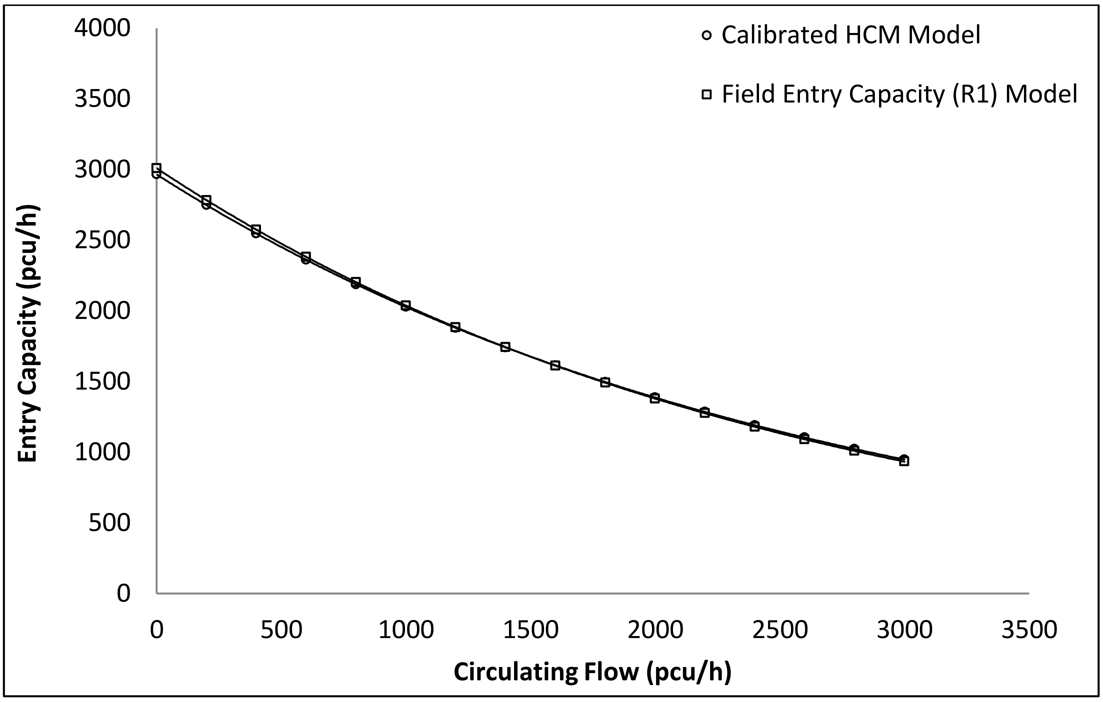


Figure 6.25 Comparison between field and calibrated HCM model at R_1

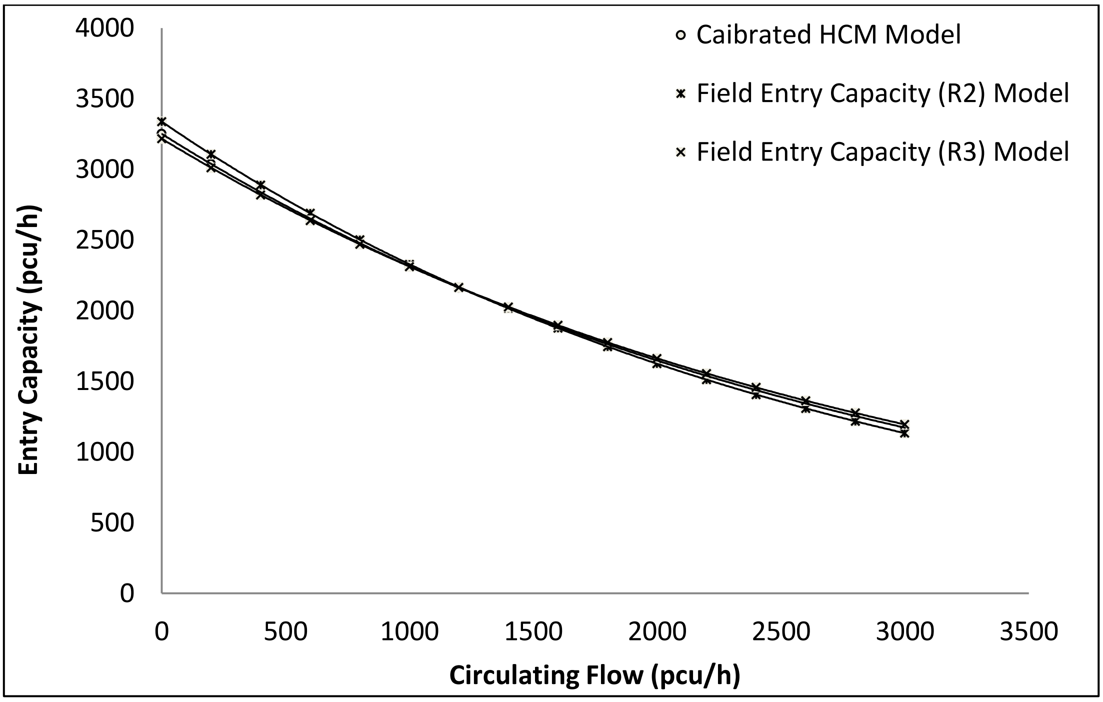


Figure 6.26 Comparison between field and calibrated HCM model at R_2 and R_3

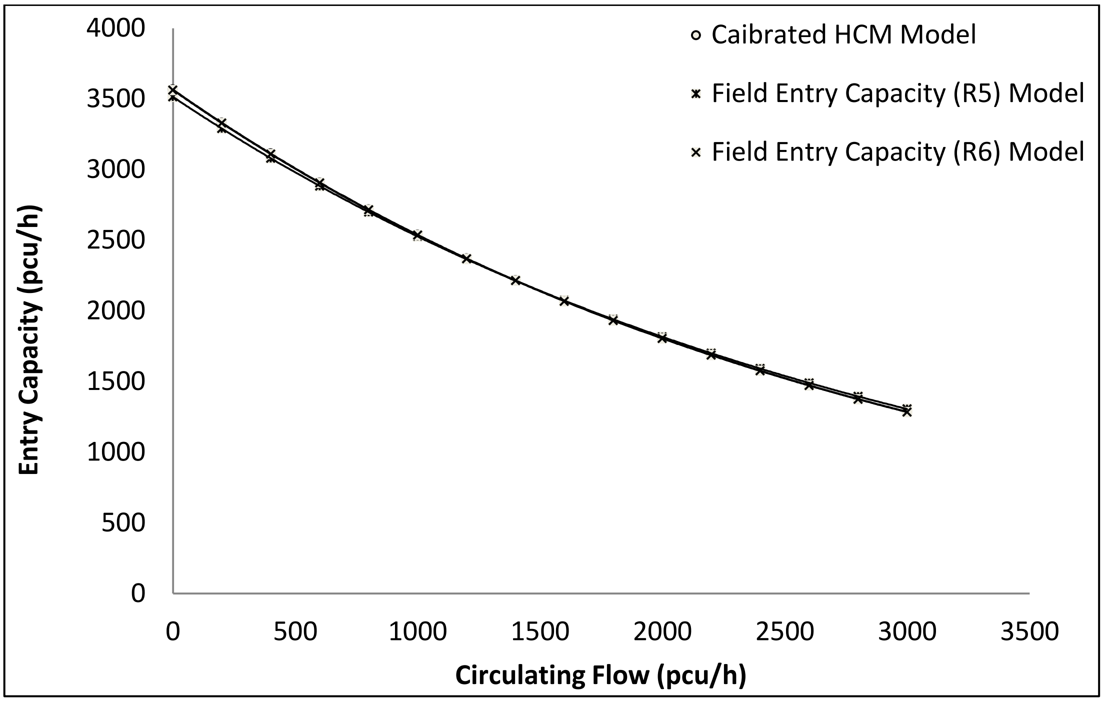


Figure 6.27 Comparison between field and calibrated HCM model at R_5 and R_6

6.4 REGRESSION BASED ENTRY CAPACITY MODEL

Over the years, researchers have developed different capacity estimation methods for roundabouts to evaluate their functionality. The entry capacity is defined as the maximum number of vehicles that can enter the roundabout in a given amount of time, with a given amount of circulating traffic volume. The gap acceptance method and the regression based method describe the entry capacity as a function of circulating traffic flow. In general, as circulating traffic flow decreases, the capacity increases due to higher chances of entry. The regression based method estimates the capacity based on the observed capacity of the existing roundabouts which have been built in the past. Although the regression based method can best reflect local traffic conditions, they can be applied to other areas after applying corrections or adjustment factors. Linear and Non-linear regression analysis has been usually adopted to develop an entry capacity model for the roundabouts.

Before considering the independent variables to arrive at the relationship or a model for entry capacity of an approach leg on a roundabout, a correlation analysis was performed in this study. The results of the correlation analysis are given in Table 6.9. Based on the correlation coefficients, it has been concluded that the entry capacity is highly correlated with the central-island diameter. The correlation between the entry capacity and the circulating roadway width is low, whereas its correlation with the entry width is very low. Therefore, entry width has been taken out and the model has been estimated by incorporating central-island diameter, circulating roadway width and circulating traffic flow as independent variables.

Table 6.9 Correlation coefficient between geometric variables

	Entry capacity	**Central island diameter**	**Circulating roadway width**	**Entry width**
Entry capacity	1.00	0.99	0.50	0.27
Central island diameter	0.99	1.00	0.43	0.23
Circulating roadway width	0.50	0.43	1.00	0.55
Entry width	0.27	0.23	0.55	1.00

Using regression based approach, the mathematical linear and non-linear forms have been fitted to the data. These forms are given in equation (6.18) and (6.19) respectively.

Linear Model:

$$Q_e = a + a_1 * Q_c + a_2 * D + a_3 * CW \tag{6.18}$$

Non-linear Model:

$$Q_e = b * e^{-b_1 * Q_c} * D^{b_2} * CW^{b_3} \tag{6.19}$$

Where,

Q_e = Entry capacity (pcu/h)

Q_c = Circulating flow (pcu/h)

D = Central island diameter (m),

CW = Circulating roadway width (m)

The statistical characteristics of the developed linear and non-linear regression models are given in Table 6.10. ‘t’ statistics for all of the coefficients in both the models show that all coefficients are statistically significant at 95 percent level of confidence (being outside the critical value range of ± 1.96). Signs of coefficients are also logical. The value of entry capacity would decrease with an increase in the circulating traffic flow, and reduction in the diameter of central-island, as well as, circulating roadway width. The coefficient of determination (R^2) for equation (6.18) and equation (6.19) are 0.873 and 0.876 respectively, which also indicate towards good predictive strength of the model. The information regarding analysis of variances, for both models, too indicates that the circulating traffic flow, and geometric design elements including central-island diameter and circulating roadway width had a strong effect on the entry capacity (*Significance F* or p-value of the models is much less than 0.05). This also signifies that the regression output for both models is not merely a chance occurrence. Therefore, linear and non-linear models are significant and can be used to estimate the entry capacity for roundabouts under heterogeneous condition in developing countries like India.

When using linear regression model, the entry capacity would be zero at high levels of circulating traffic, whereas, it would not be zero at high levels of circulating traffic if a non-linear regression model is used. However, the field observations showed that the entry capacity does not fall to zero at high levels of circulating traffic because some entry drivers would not wait for a considerable period of time and would make a forced entry into the roundabout. The field observation supports the non-linear regression model as entry capacity would not be zero at high circulating traffic flow. *Consequently, the exponential form is recommended as the most ideal approach to estimate entry capacity for Indian traffic flow conditions on roundabouts and is used for further examination.*

Table 6.10 Regression parameter estimates for linear and non-linear regression model

Model	Parameter	Estimate	Std. Error	t Stat	P-value	Lower Bound	Upper Bound
Linear	**a**	2161.00	34.97	61.79	1.57E-239	2092.31	2229.70
	a_1	-0.6772	0.0125	-54.15	4.47E-214	-0.7018	-0.6526
	a_2	14.70	0.4756	30.91	3.87E-119	13.77	15.64

	a_3	32.69	2.91	11.24	2.38E-26	26.98	38.41
Non-linear	**b**	589.90	29.47	20.02	2.85E-66	532.02	647.79
	b_1	0.00030	0.00001	49.54	1.90E-197	0.00029	0.00032
	b_2	0.39515	0.01264	31.26	8.78E-121	0.37032	0.41998
	b_3	0.09940	0.01562	6.36	4.40E-10	0.06872	0.13009

Analysis of Variance

Model		**Sum of Squares**	**DF**	**Mean Squares**	**F**	***Significance F* (p-value)**
Linear	**Regression**	1.06E+08	3	3.55E+07	1178.20	2.43E-229
	Residual	1.54E+07	512	3.01E+04		
	Total	1.22E+08	515			
Non-linear	**Regression**	1.07E+08	3	3.55E+07	1184.23	7.76E-230
	Residual	1.53E+07	512	3.00E+04		
	Total	1.22E+08	515			

6.5 VALIDATION OF MODEL

It is important to validate the proposed model before putting it to use. In order to check the validity of the proposed model, field capacity values were extracted at roundabout R_7. This roundabout was not considered while developing the entry capacity model for an approach. The field capacity values of the selected roundabout have been compared with those predicted by the model proposed in this chapter. These values have been plotted against each other, as shown in Figure 6.28. The field capacity values differ by around ±6 percent from those predicted by the proposed model. This indicates that the proposed model is capable to predict entry capacity values quite accurately and, therefore, can be used for different sized roundabouts in Indian traffic flow condition.

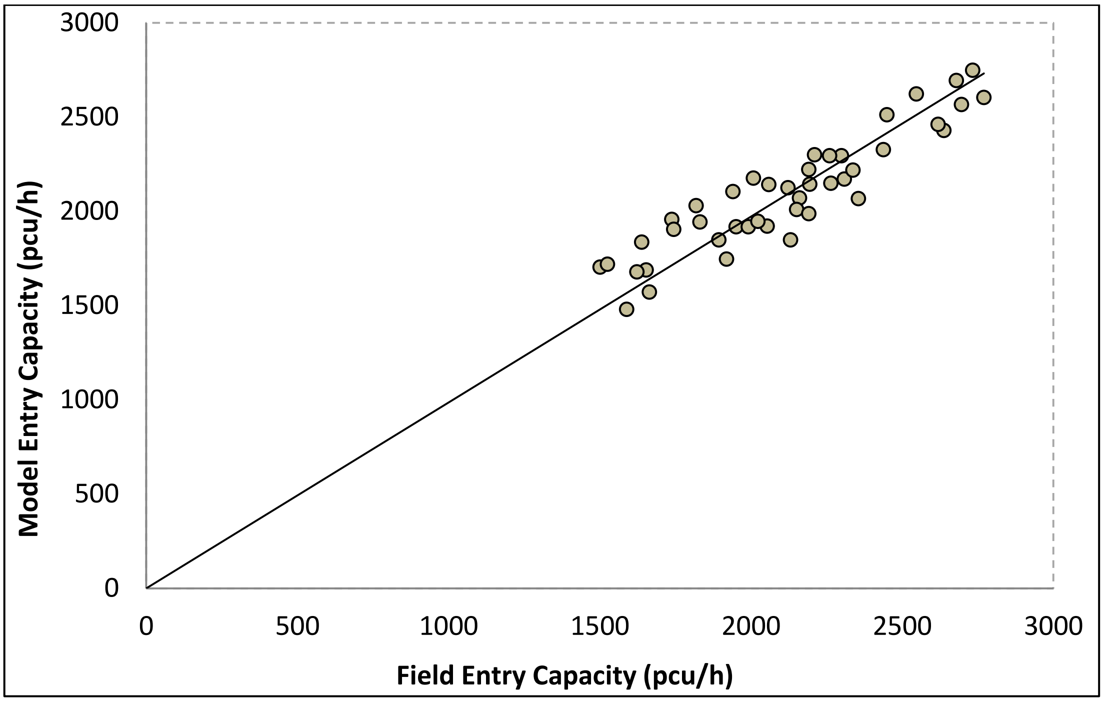


Figure 6.28 Field entry capacity versus model entry capacity

Z-test analysis has been carried out to see whether there is any similarity between the mean of field capacity values and those predicted by the proposed model. The null hypothesis (H_0) was: *There is no significant difference between the mean of field capacity values and those predicted by the proposed model*. The field capacity values and those predicted by the proposed model are normally distributed (Refer Appendix D). The results of z-test analysis are given in Table 6.11. It shows that the p-value > 0.01 and z < 2.58. Therefore, the null hypothesis is not rejected at the 99% level of confidence and there is no significant difference between the means of both capacity values.

Table 6.11 Z-test: comparison of field capacity and model capacity

	Field Capacity	**Model Capacity**
Mean	2114.61	2094.92
Variance	123151.40	95454.42
Observations	43	
Hypothesized Mean Difference	0	

z	0.28
P(Z<=z) two-tail	0.78
z Critical two-tail	2.58

The developed model was used to predict the entry capacity values for different roundabouts to see if the range of error is within acceptable values or not. It was observed from Table 6.12 that on the whole error varies between ±5% which is highly acceptable. For lower size roundabouts it is -5.3% and 5%; for medium sized roundabouts it is -6.2% and 5.8%; and for bigger sized roundabouts it is -4% and 4.5%.

Table 6.12 Error values in entry capacity on different roundabouts

Intersection ID	**Average differences in capacity (pcu/h)**		**Error in capacity (%)**	
	Range		**Range**	
R_1	-88	70	-7	4
R_2	-114	143	-6	6
R_3	-64	106	-3	5
R_4	-144	223	-7	10
R_5	-117	74	-6	3
R_6	-129	123	-6	4
R_8	-151	130	-7	5
R_9	-102	30	-5	2
R_{10}	-107	116	-4	4
R_{11}	-92	119	-4	5

6.6 SENSITIVITY ANALYSIS

Once the model is validated, then the analysis was done to examine the effect of included variables on the entry capacity. These are presented in successive sub-sections.

6.6.1 Effect of Central Island Diameter on Entry Capacity

The effect of physical parameters on the entry capacity is of considerable interest. Based on the results presented in this study, the increase in the central-island diameter, as well as, circulating roadway width provide a considerable improvement in the entry capacity values. Effects of central-island diameter on roundabout entry capacity at circulating roadway width of 7 m, 12 m and 17 m have been shown in Figure 6.29, Figure 6.30 and Figure 6.31 respectively. The plots indicate that an increase in the diameter of the central-island improves the estimated entry capacity. An increase in central-island diameter by 20 m (from 20 to 40 m) results in an increase of about 32 percent in the estimated capacity. Lower percentages are obtained if the central-island diameter is relatively large. For instance, increase in central-island diameter from 40 to 60 m results in an increase in entry capacity by 17 percent and increase from 60 to 80 m results in an increase in the entry capacity by 12 percent. These results are consistent with findings of earlier studies, which reported that a 20-m increase in central-island diameter results in a 10 to 30 percent increase in the entry capacity (Al-Masaeid and Faddah 1997).

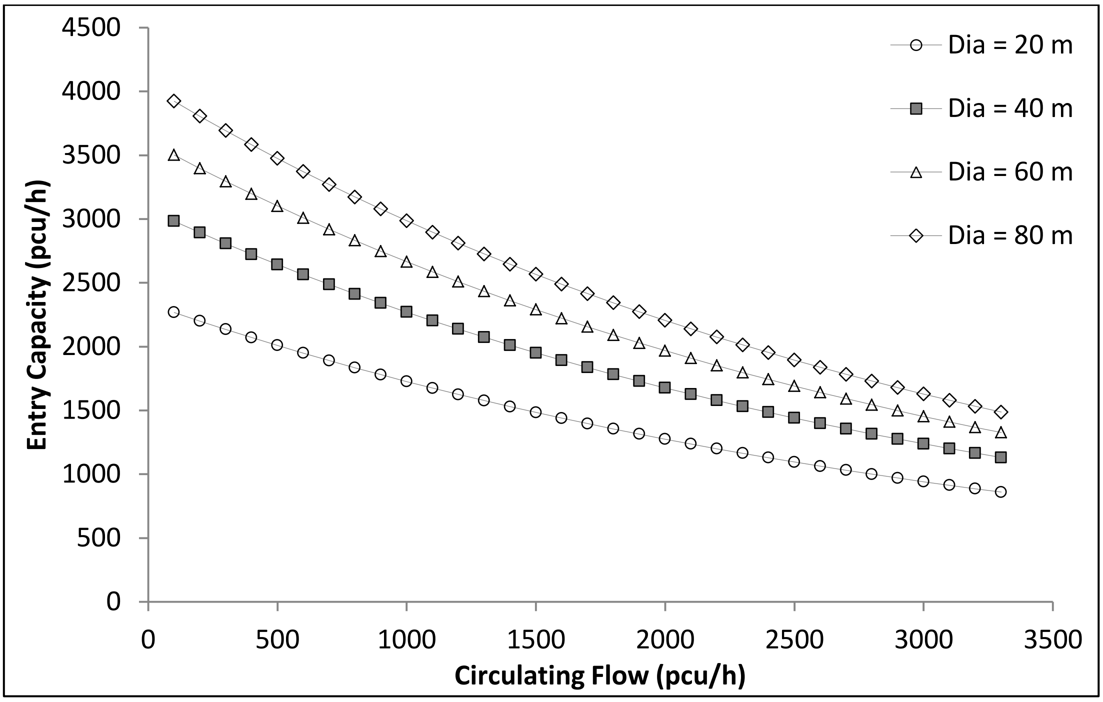


Figure 6.29 Effect of central-island diameter for roundabout CW = 7m

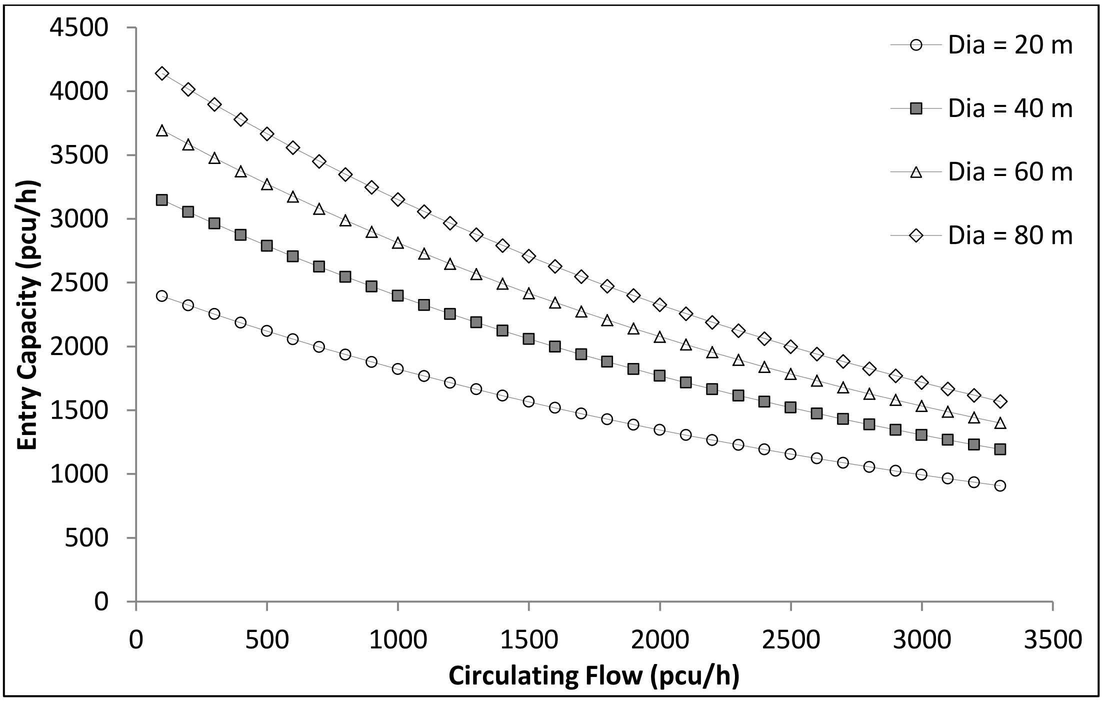


Figure 6.30 Effect of central-island diameter for roundabout CW = 12m

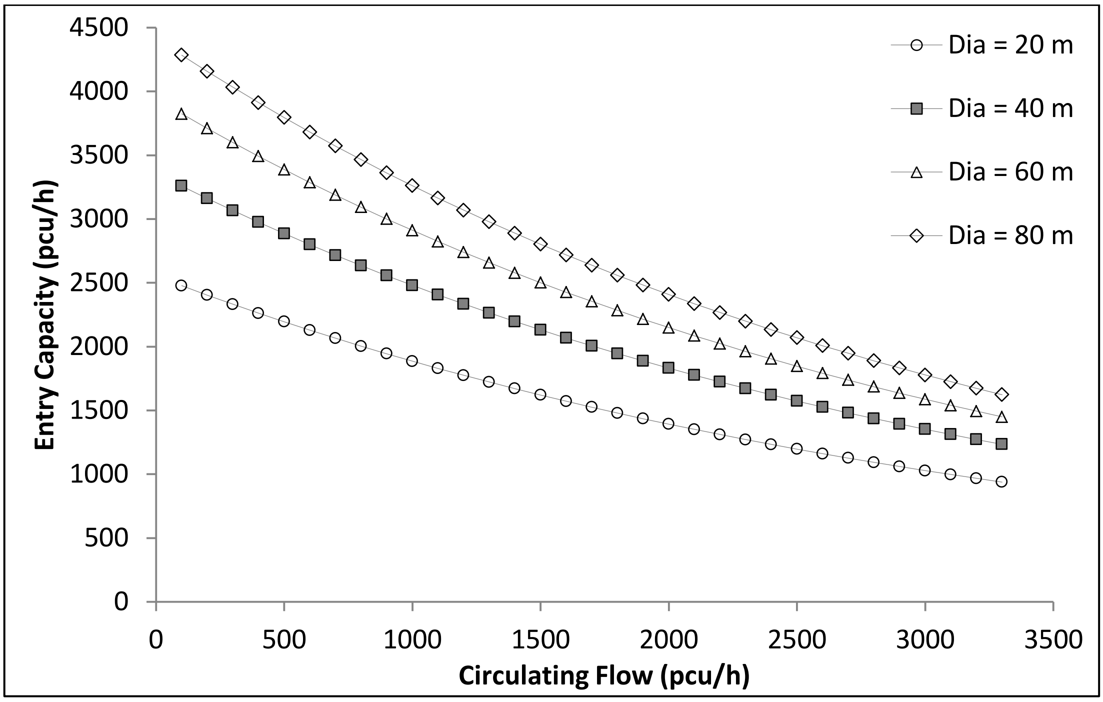


Figure 6.31 Effect of central-island diameter for roundabout CW = 17m

6.6.2 Effect of Circulating Roadway Width on Entry Capacity

The results of this study also indicate that the circulating roadway width has a significant influence on estimated capacity. Effects of circulating roadway width on roundabout entry capacity at central island diameter 25 m, 50 m and 75 m have been shown in Figure 6.32, Figure 6.33 and Figure 6.34 respectively. An increase of 5 m (from 7 to 12 m) in the circulating roadway width increases the entry capacity by 6 percent. Lower percentages are obtained if the circulating roadway width is relatively large. For instance, a 5 m increase in circulating roadway width (from 12 to 17 m) results in an increase in entry capacity of about 4 percent. The entry capacity increases by very small value with an increase in circulating roadway width. This small increase might be attributed to the behavior of circulating drivers. Field observation shows that the drivers on circulating roadway use the inner two lanes near central-island and avoids the outer lane, consequently reducing the effectiveness of the circulating roadway width.

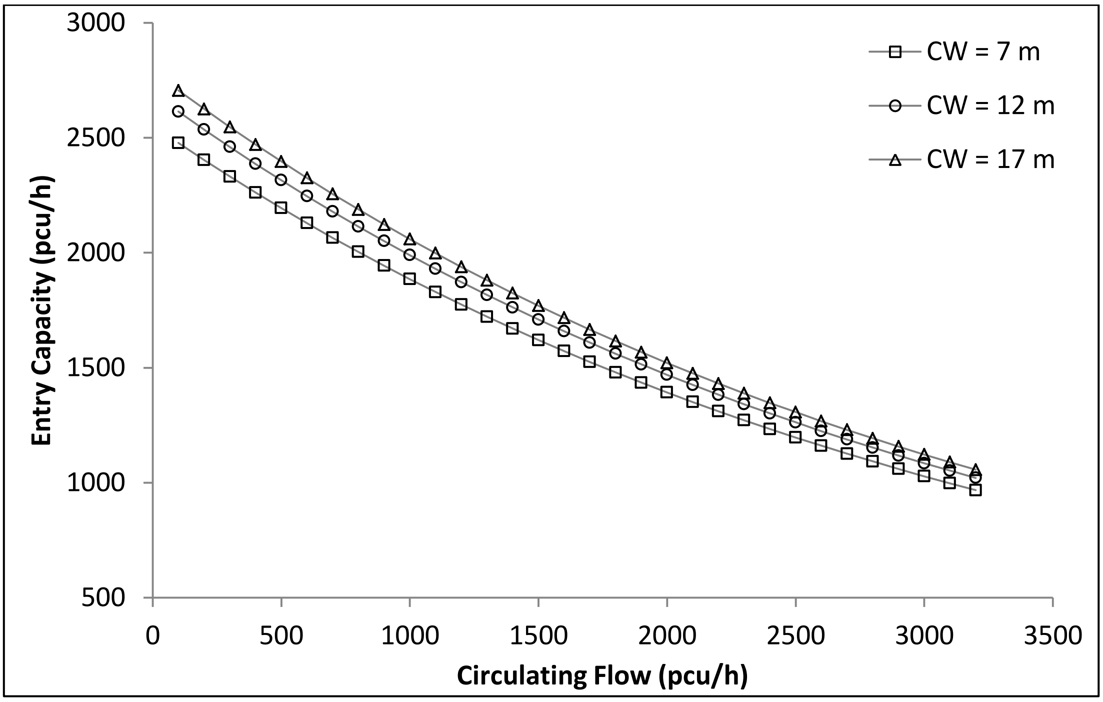


Figure 6.32 Effect of circulating roadway width on roundabout size 25 m

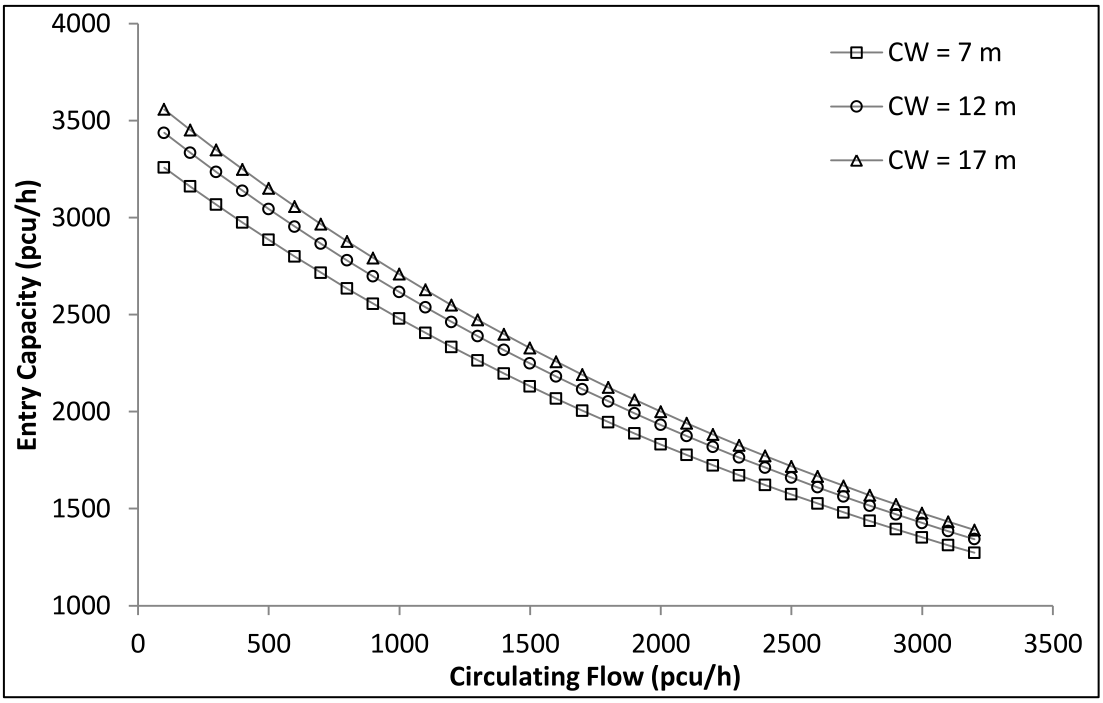


Figure 6.33 Effect of circulating roadway width on roundabout size 50 m

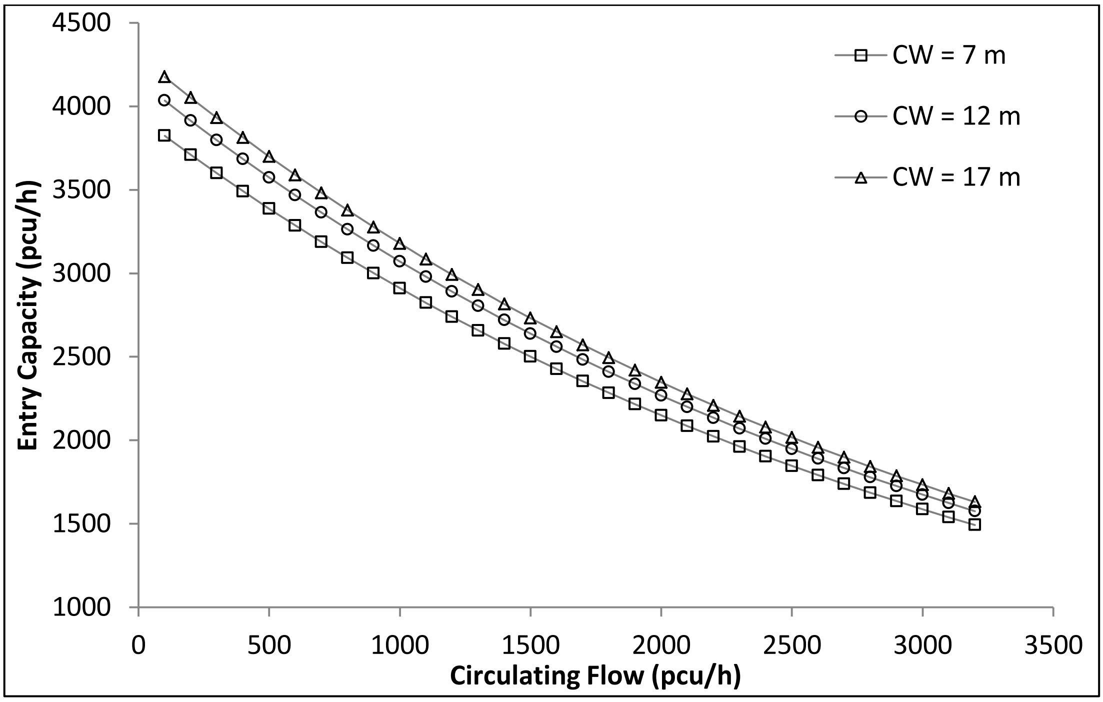


Figure 6.34 Effect of circulating roadway width on roundabout size 75 m

The range of efficient traffic flows as per IRC:65-1976 is 500 veh/h to 3000 veh/h or 410 pcu/h to 2458 pcu/h after converting into homogenous traffic based on the average traffic composition on the roundabouts as found in this study. The comparative values of weaving capacity and entry capacity at 410 pcu/h and 2458 pcu/h for roundabouts having different geometrics are presented in Table 6.13.

Table 6.13 Comparative capacity values in weaving section and at entry to a roundabout, pcu/h

Circulating Roadway width, m	**Weaving capacity for roundabout having central-island diameter / Weaving length, (IRC:65-1976)**			**Entry capacity for roundabout having central-island diameter, (Proposed entry capacity model)**		
	25 / 28 m	*50 / 45 m*	*75 / 55 m*	*25 m*	*50 m*	*75 m*
For circulating flow 410 pcu/h						
7	2970	3311	3429	2255	2965	3481
12	4130	4773	5006	2380	3129	3673

17	5052	6011	6374	2463	3239	3802
For circulating flow 2458 pcu/h						
7	2970	3311	3429	1212	1594	1870
12	4130	4773	5006	1278	1681	1973
17	5052	6011	6374	1323	1740	2043

It can be seen from the table that weaving capacity of a roundabout increases by 39 percent if the circulating roadway width gets increased by almost double of that value. But above this, if the width is increased from 7 m to 17 m (2.43 times increase) even then the weaving capacity increases merely by 70 percent. Though the increase in the circulating roadway width indicates a possibility of accommodating more vehicles but it is finally controlled by the width of the entry which defines the number of vehicles which can get in or go out of the system at a time. This is clearly depicted by the entry capacity model proposed in this study, which indicates that for the same increase (7 to 12 m and 7 to 17 m) in circulating roadway width the entry capacity only improves by around 6 percent and 9 percent respectively.

With an increase in the weaving length (due to increase in diameter of central-island) by say 60 percent, the weaving capacity is found to have increased by only 11 to 19 percent with simultaneous increase in the circulating roadway width. Even if the weaving length is almost doubled, this rise varies between 15 and 26 percent. As compared to this, the entry capacity of a roundabout is found increasing by 31 percent and 54 percent for an increase in weaving length by 60 percent and almost 100 percent respectively. This looks logical too as more space is being made available to the entering vehicles, following each other, to get accommodated in the circulating flow. It is also observed that at circulating traffic flow equal to 3500 PCU/h, irrespective of the circulating roadway width for a specific diameter of central-island, the entry capacity becomes more or less constant (varying within ±100 PCU/h). With respect to the increase in central-island diameter, it is found to be increasing by 250 PCU/h for subsequent increase in diameter by 25 m. Based on the combinations of diameter of central-island and circulating roadway width, the rate of increase in entry capacity has been estimated with respect to the base value corresponding to no circulating traffic flow condition. This is given in Table 6.14 and Table 6.15.

Table 6.14 Rate of increase in entry capacity with respect to central-island diameter (In Percent)

Central-island Diameter, m	Circulating Roadway Width, m		
	7	**12**	**17**
20	Base value = 2338	Base value = 2467	Base value = 2554
40	32	32	32
60	54	54	54
80	73	73	73

Table 6.15 Rate of increase in entry capacity with respect to circulating roadway width (In Percent)

Circulating Roadway Width, m	Central-island Diameter, m		
	25	**50**	**75**
7	Base value = 2554	Base value = 3358	Base value = 3942
12	6	6	6
17	9	9	9

It can be seen that the entry capacity increases by big value when diameter of the central-island is increased even keeping the circulating roadway width as constant. This is because of larger area being made available for the vehicles to interact with each other. In other words it can be said that when length of weaving section increases it allows more vehicles to line up in the circulating roadway.

6.7 COMPARISON WITH EXISTING REGRESSION MODELS

The proposed entry capacity model for roundabouts has been compared with Jordanian (Al-Masaeid and Faddah 1997), Malaysian (Chik et al. 2004) and Indian

(Prakash 2010) entry capacity models as given in equations (2.95), (2.111) and (2.116) respectively. For the comparison purpose, roundabouts R_2 (Diameter 37 m), R_6 (Diameter 50 m) and R_{10} (Diameter 76 m) have been selected and these are termed as small, medium and large sized roundabouts, respectively. The comparison between the proposed entry capacity model, Jordanian, Malaysian and Indian (Prakash 2010) entry capacity models for roundabouts R_2, R_6 and R_{10} are shown in Figure 6.35, Figure 6.36 and Figure 6.37 respectively.

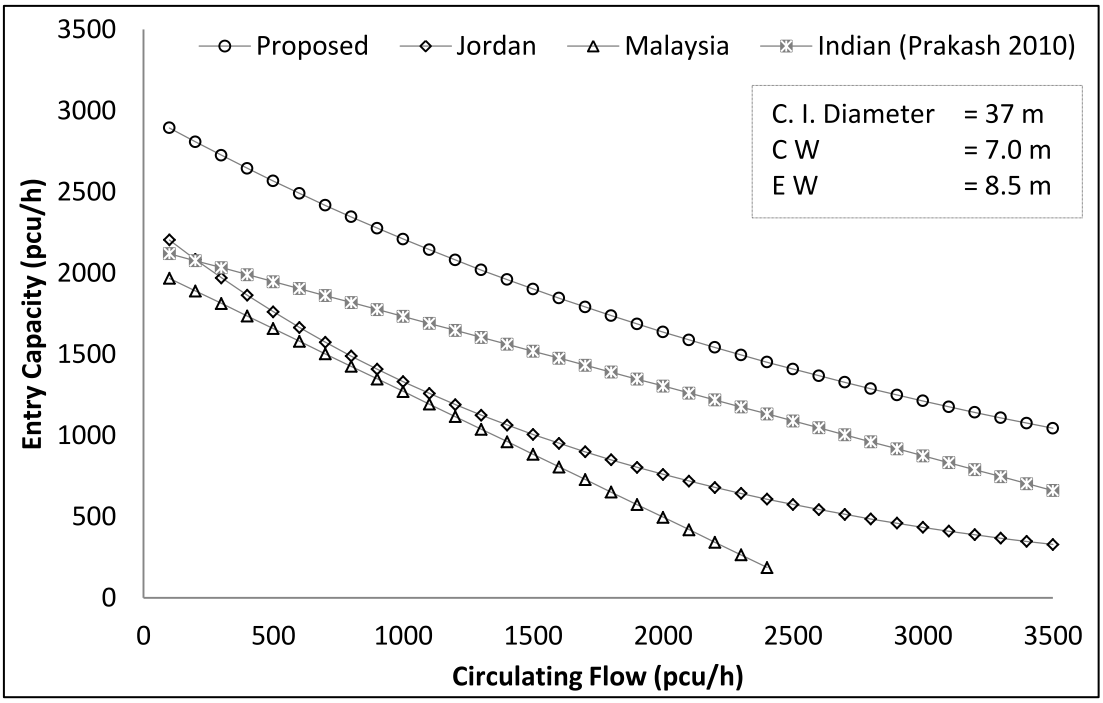


Figure 6.35 Comparison of entry capacity models for roundabout R_2

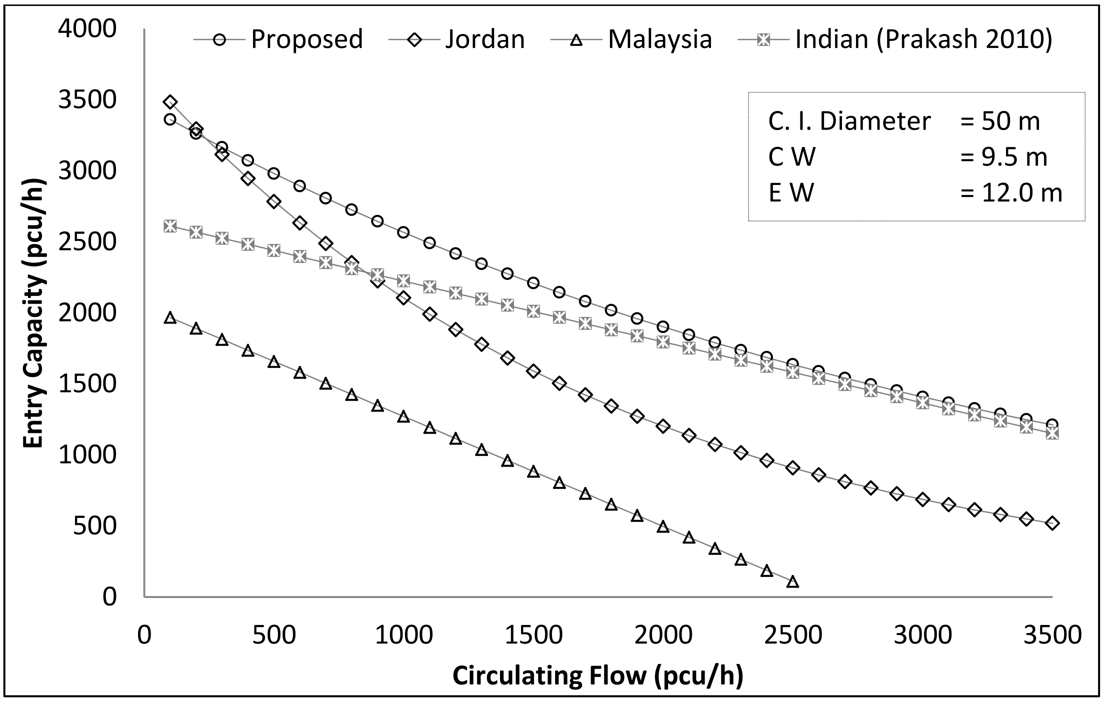


Figure 6.36 Comparison of entry capacity models for roundabout R_6

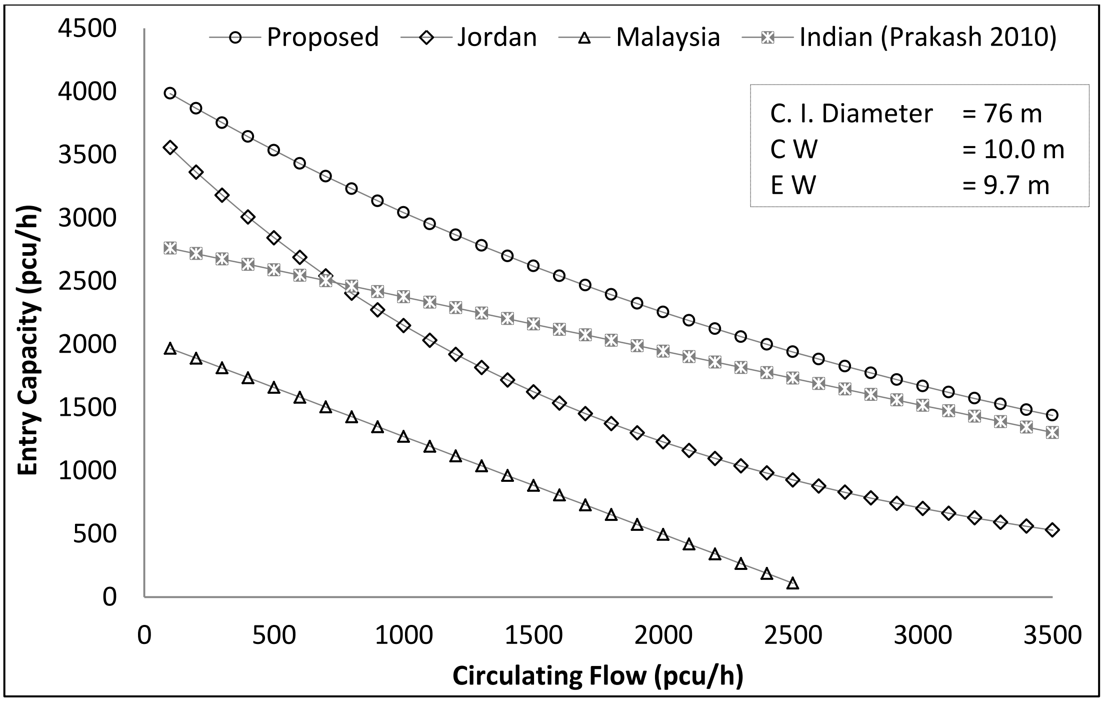


Figure 6.37 Comparison of entry capacity models for roundabout R_{10}

Jordanian entry capacity model had exponential form somewhat synonymous to the proposed entry capacity model, whereas, Malaysian and Indian (Prakash 2010) model had linear form. The approach presented in this chapter is based on a premise that there will be some entry flow from an approach in the circulating area even at high circulating traffic flows. This is similar to the Jordanian model. But Malaysian and Indian (Prakash 2010) model indicates that the entry flow from an approach will become zero at high circulating traffic flows, thus indicating the locking of the roundabout. In that sense, the Malaysian model gives locking circulating traffic flow as around 2500 pcu/h, whereas, Indian (Prakash 2010) model gives a very high value of around 4700 pcu/h. This locking circulating traffic flow is not getting affected by the diameter of the central-island of a roundabout in the case of Malaysian model. But, Indian (Prakash 2010) model shows that this locking circulating traffic flow is in the range of 5500 to 6000 pcu/h for the diameter of the central-island varying between medium to large size. This looks to be quite high for the given condition.

In general, the proposed entry capacity model gives highest entry capacity from an approach, whereas, Malaysian model gives the lowest entry capacity amongst the four models, irrespective of the size of the roundabout and width of entry or circulating road. In the case of roundabout R_2 (small size roundabout), Jordanian entry capacity model and Malaysian entry capacity model gave almost the same entry capacity values (difference within 100 pcu/h) upto 1500 pcu/h of the circulating traffic flow. Thereafter, Jordanian model diverts away from the Malaysian model for high values of circulating traffic flow. The difference between proposed entry capacity model and Jordanian entry capacity model is approximately constant, and is around 800 pcu/h higher in Indian conditions for all the ranges of the circulating traffic flow. The difference between entry flow by proposed model and Indian (Prakash 2010) model is approximately 600 pcu/h upto the circulating traffic flow of 1000 pcu/h. Thereafter, the difference between these two models is reducing and becomes roughly 350 pcu/h at high levels of circulating traffic flow. The other three models provide entry capacity in the range of 1900-2200 pcu/h, whereas proposed model gave it as 3000 pcu/h.

In the case of roundabout R_6 (medium sized roundabout), Jordanian entry capacity model gave higher entry capacity values at very low level of circulating traffic flow (upto 750 pcu/h). The difference between entry capacity by proposed model and Jordanian model is low (100 pcu/h) upto 500 pcu/h of the circulating traffic flow and thereafter it stabilizes at a difference of 1000 pcu/h beyond the circulating traffic flow of 2500 pcu/h. The difference between proposed entry capacity model and Indian (Prakash 2010) entry capacity model is 1200 pcu/h at low level of circulating traffic flow. The difference between these two models narrows down with the increase in the circulating traffic flow. They behave in close proximity of 55 pcu/h beyond 2500 pcu/h of the circulating traffic flow.

The behavior of these models for roundabout R_{10} (large sized roundabout) is found to be the same as on roundabout R_6. It can be noted that Jordanian model is giving similar results of entry capacity if central-island diameter is 50 m or more. This indicates that this model may work up to middle sized roundabouts only. But Indian (Prakash 2010) model and proposed model are responsive to the variation in central-island diameter, as well as in circulating traffic flow. Proposed model is giving higher capacity values than Indian (Prakash 2010) model, which looks to be quite high while considering the geometric attributes of the roundabout.

A plot has been made between the field maximum entry traffic flows and the outcomes of different models under consideration as shown in Figure 6.38. It is observed that the predicted entry capacity values by different models are lower than the field maximum entry traffic flow values. Considering the linear model between field and predicted entry flow values and keeping the constant as zero (which should be the case), it is found that the R^2 value for the plot based on proposed entry capacity model is the highest among all (at 0.804) as given in Table 6.16. This indicates its adequacy and good agreement between the two values.

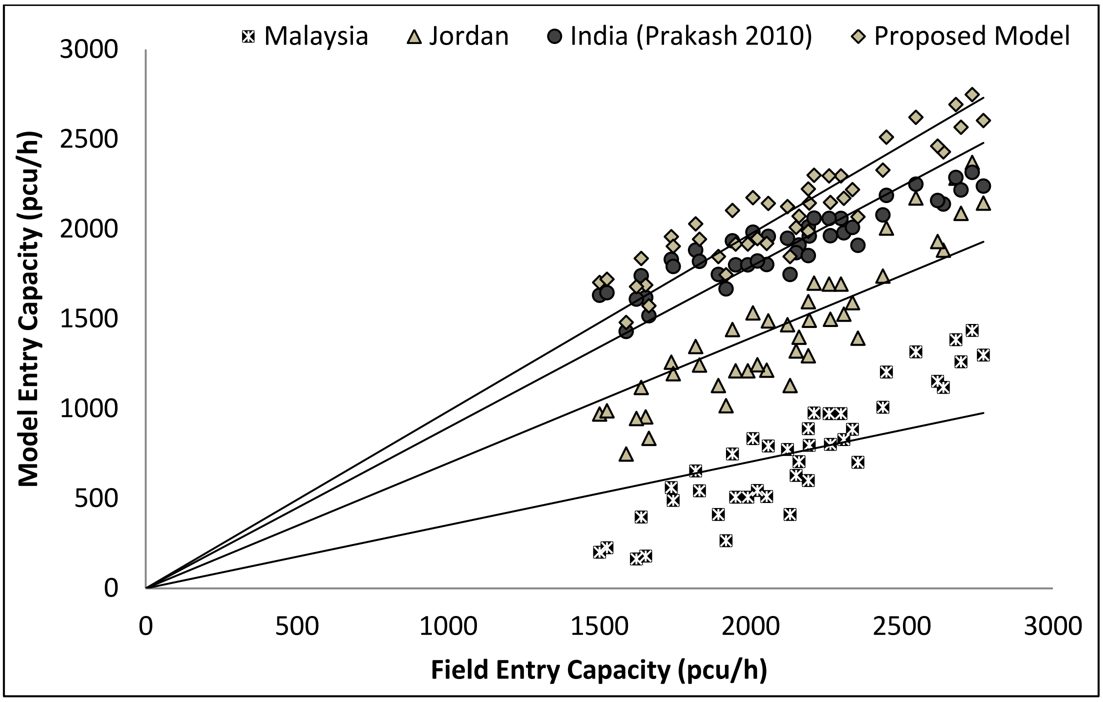


Figure 6.38 Field entry capacity versus model entry capacity

Table 6.16 The relationships between field and predicted entry capacity values

Entry Capacity Model	**Standardized Regression Relationship: CP = a*CF**	**R^2 value**
Malaysian	CP = 0.35*CF	0.478
Jordanian	CP = 0.70*CF	0.744
Prakash (India)	CP = 0.89*CF	0.501
Proposed Model	CP = 0.99*CF	0.804

CP = Predicted capacity by model; CF = Field maximum entry traffic flow

6.8 SUMMARY

The entry capacity of an approach is found to be dependent on the circulating traffic flow and size of a roundabout. Queue formation in the approach is considered as an indicator of approach being operating at capacity. The relationship between entry capacity and circulating traffic flow is found to be negative exponential i.e. the entry capacity decreased exponentially with an increase in the circulating flow. The exiting traffic flow is not found influencing the entry capacity. Three models are presented to estimate the entry capacity of a roundabout for small, medium and large size roundabouts.

The HCM (2010) equation for estimating the entry capacity was modified for its adaptation to heterogeneous traffic conditions. Parameters A and B of the HCM equation were calculated based on critical gap values obtained from the field data. The entry capacity is estimated from modified HCM model and is compared with the field model developed during formation of a continuous and stable queue at the entry of the roundabout. The modified HCM model for entry capacity is found differing from the field model under heterogeneous traffic conditions. Therefore, multiplicative adjustment factors for different size of roundabouts have been developed to satisfy the traffic flow condition prevailing in developing countries like India.

A regression based model for estimating entry capacity of an approach leg on a roundabout is developed in this chapter. Data from ten roundabouts was used for the estimation of parameters and separate roundabout was used to validate the results. The proposed method works on the premise that certain traffic will keep entering the circulating roadway even under very high circulating traffic flow condition. That's way an exponential functional form is adopted. With this form, the proposed method indicates a minimum entry traffic flow between 1000 pcu/h and 1500 pcu/h even at a circulating traffic flow of 3500 pcu/h (for variation in circulating roadway width from 7 m to 17 m).

The analysis was also carried out to examine the effect of variability in the influencing variables, which were found significant in the estimation of the regression model. The entry capacity of an approach is found to be depending on the circulating traffic flow, central-island diameter and circulating roadway width. The central-island diameter was found highly influencing for the entry capacity of an approach. Around 32% increase in the entry capacity was observed with doubling of the diameter of central island diameter from 25 m. After that the increase in diameter by 50% caused an increase in the entry capacity by 17%. The effect of circulating roadway width was to increase the entry capacity with an increase in this variable. But above width of 11.0 m, it became more of less constant, somewhere in a range of 3500 pcu/h and 3700 pcu/h.

The existing developing countries models (Jordan, Malaysian and Indian) were also examined to see if any of those can be used directly in Indian traffic flow condition on roundabouts. It was found that none of those can be applied directly to estimate or predict the entry capacity of an approach. One model which was developed in 2010 for Indian traffic flow conditions was also examined for its usefulness. However, it was observed that it was predicting entry capacity values lower than the observed field maximum entry flow values. The Malaysian and previous Indian model could give the value of circulating traffic flow at which the roundabout may get locked i.e. no traffic can enter from an approach. Interestingly Malaysian model provided this value as constant (at 2500 pcu/h) irrespective of the diameter of the central-island, whereas, previous Indian model gave values ranging between 4700 pcu/h to 6000 pcu/h depending upon the increase in the size of the roundabout.

Going by the above discussion, it may be inferred that the proposed entry capacity model for an approach on a roundabout is workable and usable under varied geometric and traffic conditions that may prevail in any developing country like India.

CHAPTER 7:
CONCLUSIONS AND RECOMMENDATIONS

7.0 GENERAL

The present research was taken up with the objective of determining entry capacity of roundabouts under heterogeneous condition. Field data were collected using video camera at roundabouts without any side friction factors. Gaps, headway and flow data were extracted from the video by running the film on a large screen monitor. The complexity of mixed nature of traffic is simplified by evaluating PCU for different categories of vehicles. Headway and width of the vehicles are taken as the prime variable to determine PCU. A new concept of heterogeneity equivalency factor (H-factor) is also introduced in this research which can avoid use of PCU factors for individual vehicles. A new method for estimating the critical gap based on minimizing the sum of absolute difference in the accepted and rejected gaps is proposed in this study. This method can be applied to the situations where rule of priority is not respected. This method can also be used for estimating the critical gaps even with smaller number of observations of entering vehicles. It has been observed that entry capacity values are different at different size of roundabouts. A mathematical non-linear model is developed based on the geometric parameters of the roundabout to determine the entry capacity. The major conclusions drawn from the above analysis are summarized in this chapter and contribution of the study and recommendations for future work are also presented.

7.1 OBSERVATIONS AND CONCLUSIONS

The significant observations and conclusions from analysis are explained in four sections as passenger car unit, critical gap and follow-up time, analysis of entry flow, and entry capacity model. These are explained below.

7.1.1 Passenger Car Unit

i. The Passenger Car Unit (PCU) is a standardized value that is used to convert the varied composition of traffic flow to a common unit, usually taken as a standard car. This makes it universal and allows comparison. This is estimated for different category of vehicles. Out of the different flow variables only lagging headway was considered. This is already discussed in chapter-4. Static characteristics i.e. the width of the vehicle was also considered for estimating the PCU factor. The equation used to determine PCU for a vehicle type *'i'* is given as (7.1)

$$PCU_i = f_i * \frac{H_i}{H_c} \tag{7.1}$$

Where,

$$f_i = \text{vehicle width factor} = \frac{w_i}{w_c} \tag{7.2}$$

The units have the meanings as defined before.

ii. The lagging headways are found varying between 1.53 s and 6.26 s across the vehicles plying on roundabouts, which vary in size. It has been observed that with an increase in the size of the roundabout, the lagging headway for the same category of vehicles, in general, decreases. Similarly as the size of the vehicle increases, the lagging headway on the same roundabout also increases. This follows the normal traffic norms. Lagging headway is found to be the least for motorized two-wheelers and highest for the heavy vehicles. The range is between 1.53 s and 3.10 s, and between 3.15 s and 6.26 s respectively for the two vehicle categories. For standard car it varies between 1.96 s and 3.78 s.

iii. New static PCU values based on this study are suggested for the analysis of any traffic study related to roundabouts in a developing country with traffic similar to India. PCU values for five-categories of vehicles, being considered in the present study, are suggested as 0.34 for motorized two-wheeler, 1 for motorized three-wheeler and small car, 1.36 for big car and 2.91 for heavy vehicle. The estimated values of PCU for different vehicles are found to be quite similar to those given in literature (Al-Masaeid and Faddah 1997; IHCM 1993; IRC-65 1976; Kumarage 1996; Pakshir et al. 2012). The comparison of the suggested values with those given in IRC-65 (1976) indicates that there is no change in the PCU value for 3W. The PCU value for 2W is found to be lower (almost half) than that given in the IRC-65 (1976) code. In the case of HV, the estimated PCU value is not too different than that given in the IRC-65 (1976) code. PCU value for HV got marginally increased.

iv. Another important finding has been the division of car category into two categories based on size and engine power. These were termed as ‘small car’ and ‘big car’. The estimated PCU values of the two have been found statistically different. Hence, PCU values for both categories are suggested for incorporating into the roundabouts like IRC:65 (1976).

v. The effect of traffic flow and geometric parameters of the roundabout is examined on the estimated PCU values for different category of vehicles. It has been found that there is a weak or no relationship between PCU values of different vehicle categories, and geometric and traffic flow parameters of the roundabouts. However, the PCU values of heavy vehicles were found getting positively influenced with the circulating roadway width, entry width, circulating traffic volume and circulating traffic volume per circulating roadway width. But it got negatively influenced with central-island diameter. Such influences were quite low for vehicles having size equivalent to standard car, and negligible for small size vehicles.

vi. A new concept of Heterogeneity Equivalency Factor (H-factor) is introduced in the present research which can be used to convert mixed traffic flow into homogeneous traffic stream without actually making use of PCU factors. To arrive at this factor, it is defined as ratio of traffic flow in pcu/h to traffic flow in veh/h. For roundabouts having quite low composition of heavy vehicles, it was found to be 0.8166. For those cases where the proportion of heavy vehicle is more than 4-5 %, a new value needs to be estimated. This shall be more than 1.0.

vii. H-factor also correlated with the proportion of vehicles and traffic volume per circulating roadway width. This is reproduced here as (7.3):

$$H = 1 - 0.676 * P_{2W} + 0.508 * P_{BC} + 2.718 * P_{HV} - 6.081 / V_{CW} \quad (R^2 = 0.853) \quad (7.3)$$

(-31.85) (3.83) (16.23) (-2.50)

The t-statistics given in ( ) are found to be significant at 95 % confidence level.

viii. H-factor is found increasing with an increase in the circulating traffic flow per circulating roadway width and tends to become constant at higher circulating traffic flow, irrespective of the composition of vehicles. H-factor also increases with an increase in the percentage of HVs in the traffic stream, whereas, it increases with a decrease in the percentage of 2Ws in the traffic stream.

7.1.2 Critical Gap and Follow-Up Time

i. A new method is proposed for the estimation of critical gap for different categories of vehicles. This is based on minimizing the sum of absolute difference between accepted and rejected gap data. Only two sets of data (highest rejected gap and accepted gap) are required. It allows to take the rejected gap as zero in those cases where the first gap is accepted by a driver. The functional form used for estimating the critical gap is a minimization problem which is given by equation (7.4).

$$\mathrm{Min}\left[\sum_{i=1}^{n}\left\{\mathrm{Abs}\left(T_c - R_i\right) + \mathrm{Abs}\left(A_i - T_c\right)\right\}\right] \tag{7.4}$$

All terms are already being defined in chapter-5.

ii. The advantage of using this method is that it is free of making prior assumptions regarding the distribution function of critical gap and the driver behaviors and does not fail even when there are very few rejected gaps. The values estimated by the proposed method are found to be similar to those estimated using maximum likelihood method which is reported to be the most accurate one (Miller 1972, Brilon et al. 1999, Troutbeck 2014). The comparison based on percent violations indicate that the proposed method followed the concept irrespective of the vehicle category whereas, maximum likelihood method deviated in the case of motorized two-wheelers.

iii. It is mention here that the iterations are also free of the seed value to be given for critical gap T_c. Based on the analysis, it was decided to adopt the average of all accepted and rejected gaps as a starting seed value. It gave the convergence faster. Another issue was the number of data set needed for iteration. It was found that the proposed method can be used for estimating the critical gaps even with smaller number of observations of entering vehicles. The present study showed that the optimum number of data set for the estimation of critical gap comes out to be 45 for two-wheelers and three-wheelers, and 60 for small cars if a deviation of 0.1 s is allowed from estimated critical gap value. If this is reduced to 0.05 s then the sample size needed will be 60 and 150 respectively.

iv. The critical gap value is found varying between 1.48-1.60 s for two-wheelers, 1.68-1.94 s for three-wheelers, 1.85-2.30 s for small cars, 1.92-2.39 s for big cars and 2.45-2.67 s for heavy vehicles. These are increased with an increase in the size of the vehicle. The critical gap values obtained under heterogeneous traffic conditions are very low when compared with those given in literature (Flannery and Datta 1997; Hagring et al. 2003; HCM 2010; Polus et al. 2005; Tolazzi 2004; Tracz et al. 2011) for homogeneous traffic condition. It may be attributed to the behavior of the drivers on roads under mixed traffic conditions in developing countries. These drivers are ready to pick small gaps to complete their maneuver. It is because of higher pressure on roads during peak periods, may be due to increasing motor vehicle ownership and population. The data interpretation may put these drivers in aggressive driver category, as compared to the driver discipline and behavior in developed countries.

v. Based on the statistical analysis, it has been found that there is no significant effect of the circulating traffic flow on critical gap and follow-up time in the case of roundabouts. The average ratio of follow-up time to critical gap has been found to be 0.64 (0.62 – 0.66 for motorized two-wheelers and 0.61 – 0.67 for small cars). The literature suggests this to be 0.60 or more (Brilon 1988; Hagring et al. 2003; Tian et al. 2000).

7.1.3 Analysis of Entry Flow

i. Statistical analysis indicated that the correlation between entry capacity (dependent variable) and circulating traffic flow (independent variable) is high, whereas its correlation with exiting traffic flow in found to be low. Therefore, the flow parameter considered to develop a model for entry capacity is taken as only circulating traffic flow. Functional relationship, linear as well as negative exponential, is found to be statistically significant. Negative exponential functional relationship is being suggested for use as it represents the traffic and driver behavior more closely at a roundabout, under constrained traffic conditions.

ii. Three models are presented to estimate the entry capacity of a roundabout for small, medium and large size roundabouts. These models are purely based on circulating traffic flow. These are given below:

a) *For small size roundabouts (Central-island diameter 25 m to 40 m)*

$$Q_e = 3252 * e^{-0.00037*Q_c} \tag{7.5}$$

b) *For medium size roundabouts (Central-island diameter 41 m to 60 m)*

$$Q_e = 3483 * e^{-0.00030*Q_c} \tag{7.6}$$

c) *For large size roundabouts (Central-island diameter 61 m to 90 m)*

$$Q_e = 3843 * e^{-0.00024*Q_c} \tag{7.7}$$

iii. The developed model (equation (7.6)) has been validated on a separate medium size roundabout. It has been found that the developed model is capable to predict entry capacity values quite accurately when compared to field capacity values.

iv. To find relation between entry capacity and geometric parameters, entry capacities have been plotted against central-island diameter, circulating roadway width and entry width. Power function is found to provide the best fit between entry capacity and central-island diameter, circulating roadway width and entry width. The variation or dispersion of data is found to be quite low for entry capacity and central-island diameter and is a bit high for the other two relationships.

7.1.4 Entry Capacity Model

i. The HCM (2010) equation for estimating the entry capacity was modified for its adaptation to heterogeneous traffic conditions. Parameters A and B of the HCM equation were calculated based on critical gap values obtained from the field data. The entry capacity estimated from modified HCM equations were compared with the field entry capacity and found that that the modified HCM models are found differing from the field models. Therefore, an attempt is also made to use an adjustment factor in modified HCM models to satisfy the traffic flow condition prevailing in developing countries like India. The final calibrated HCM model and adjustment factors for different size of roundabouts are given in Table 7.1. The approach presented here is different than that used in HCM (2010) which is based on number of lanes at entry and circulating roadway. As already mentioned the drivers are not following the lane discipline in developing countries, it is not wise to replicate the HCM (2010) approach here. Rather, the data suggests that traffic behavior on roundabouts is carriageway based, and differs with respect to the size of the roundabout. Hence final models are mainly governed by size, with tentative difference due to number of lanes at entry and circulating roadway taken care off by factor 'f_a'.

Table 7.1 Final calibrated HCM model and adjustment factors for different size of roundabouts

Central island diameter	**Adjustment factor (f_a)**	**Parameter**		**Entry capacity (Q_e)**
		A	**B**	
25.0	1.054*	2812	0.00038	$Q_e = f_a * A * e^{-B*Q_c}$
37.0	1.033*	3147	0.00034	
37.0				

49.0	$1.133^{\#}$			
50.0				

*Two-lane entry and circulating roadway #Three-lane entry and circulating roadway

ii. A regression based model for estimating entry capacity of an approach leg on a roundabout also developed which includes geometric parameters like diameter of central-island, and width of circulating roadway along with the traffic flow parameter circulating flow. Entry width was kept out of the model as its correlation with entry capacity (dependent variable) was very low. Both linear and non-linear models were developed and found statistically good. Again, exponential form was suggested for use, in line with previously mentioned points.

iii. The developed regression model is also validated on another roundabout and found that there is about ±6 percent difference between the field entry capacity and those predicted by the proposed model. The developed model was further examined on different size of roundabouts. The results were encouraging as the percent variation found in the estimated entry capacity with respect to field entry capacity is ±5 percent for small size roundabouts; ±6 percent for medium size roundabouts and ±4.5 percent for big size roundabouts. Regression model for entry capacity is given as:

$$Q_e = 589.90 * e^{-0.00030Q_c} * D^{0.39515} * CW^{0.09940} \qquad R^2 = 0.876 \qquad (7.8)$$

(20.02) (49.54) (31.26) (6.36)

The t-statistics given in ( ) are found to be significant at 95 % confidence level.

iv. Sensitivity analysis is used to see the effect of physical parameters of the roundabout on entry capacity and found that the central island diameter has the greatest effect on entry capacity while the circulating roadway width has the smallest effect on entry capacity. Around 32 % increase in the entry capacity was observed with doubling of the diameter of central island diameter from 25 m. After that the increase in diameter by 50% caused an increase in the entry capacity by 17 %. The effect of circulating roadway width was to increase the entry capacity with an increase in this variable. The variation is shown in

Figure 7.1 and Figure 7.2 with respect to central-island diameter and circulating roadway width respectively.

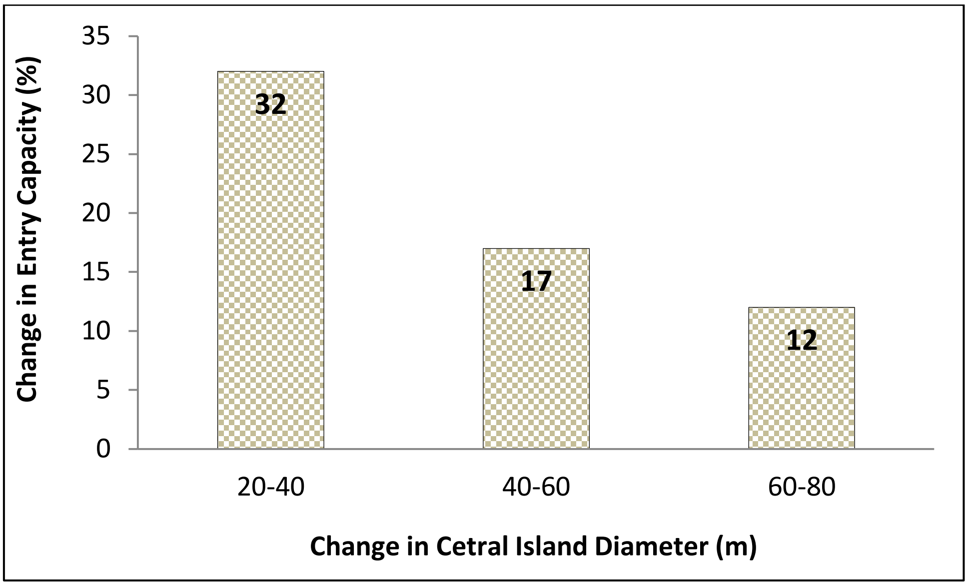


Figure 7.1 Variation in entry capacity with central-island diameter

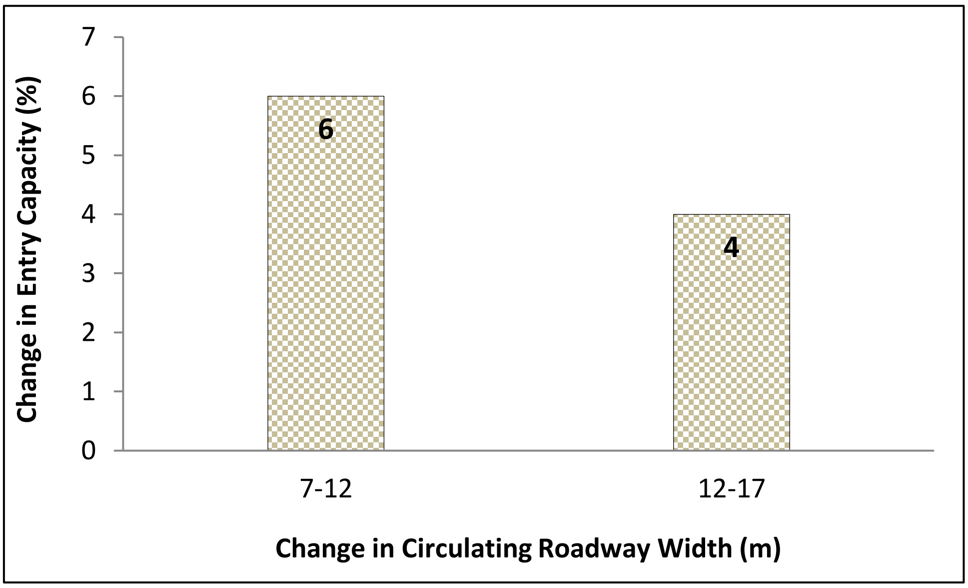


Figure 7.2 Variation in entry capacity with circulating roadway width

v. A comparison has also been made with respect to the variations in weaving length. It was found that for a certain percent increase in weaving length, the increase in the entry capacity is almost half of it. But similar analysis with respect to the simultaneous change in the circulating roadway width causes one-fourth change in the entry capacity.

vi. The existing developing countries models (Jordan, Malaysian and Indian) were also examined to see if any of those can be used directly in Indian traffic flow condition on the roundabouts. It was found that none of those can be applied directly to estimate or predict the entry capacity of an approach. The previous Indian model and Malaysian model are linear in nature. Jordan model follows the developed model for medium to high value of circulating traffic flow. Malaysian model works independent of size of the roundabout and indicates that it will lock at a circulating traffic flow of 2500 pcu/h. Previous Indian model gives these values in a range of 4700-6000 pcu/h. Both of these do not hold in field conditions. Same is the case for entry capacity. Jordan model somewhat follows the developed regression model. The proposed model gives entry capacity in the range of 3000-4000 pcu/h for small to big size roundabouts, having step increases of 500 pcu/h. Lower limit for entry capacity is expected to be ranging between 800 to 1200 pcu/h.

vii. The proposed entry capacity model for an approach on a roundabout is workable and usable under varied geometric and traffic conditions that may prevail in any developing country like India. It has been found applicable for 7 to 17 m width of circulating roadway, as well as, for 25 m to 80 m of diameter of central-island. Similar conditions of traffic in other developing countries make it universal in application.

7.2 CONTRIBUTIONS OF THE STUDY

The present research is one of the most comprehensive studies carried out so far on roundabouts in India. The major contributions of the present study are summarized below.

a) New static PCU values are suggested for the analysis of traffic on roundabouts in India. The comparison with literature indicates that these can be used in other developing countries also. The approach is extended to propose H-factor which can convert heterogeneous traffic into homogeneous one without use of PCUs.

b) A new method is suggested for the estimation of critical gap for different categories of vehicles. This is based on minimization of the sum of absolute differences of accepted and rejected gaps with respect to critical gap. This method is more suitable than any other method available in the literature as it can be applied to even such situations where rule of priority in traffic flow is not respected. The method is accurate like MLM and uses small size of data sets.

c) The development of a model for estimating entry capacity of an approach leg on a roundabout under heterogeneous traffic conditions is another mile stone. Three sets of models, one related to circulating traffic flow, second based on traffic flow parameters (i.e. modified HCM model) and third a regression model, are suggested for use. An example to estimate entry capacity for a given characteristics of a roundabout is given in Appendix E. Method related to circulating traffic flow is the simplest of all and can be readily used by the implementing agencies. If impact of geometric elements need to be incorporated then method 3 shall be used. Method based on critical gap is cumbersome and is not advised for use in present setup of implementing agencies.

7.3 CONTRIBUTION TO IRC: 65-1976

a) New static PCU values are recommended as given in Table 7.2.

Table 7.2 PCU values for different categories of vehicles on roundabouts

PCU Values	2W	3W	SC	BC	HV
New	0.34	1	1	1.36	2.91
Old (IRC: 65-1976)	0.75	1	1	-	2.80

b) New models for estimating the entry capacity of roundabouts are recommended as given in Table 7.3.

Table 7.3 Roundabout capacity models

Based on circulating traffic flow	Small size roundabouts	$Q_e = 3252 * e^{-0.00037*Q_c}$
	Medium size roundabouts	$Q_e = 3483 * e^{-0.00030*Q_c}$
	Large size roundabouts	$Q_e = 3843 * e^{-0.00024*Q_c}$
Based on circulating traffic flow and physical parameters	$Q_e = 589.90 * e^{-0.00030Q_c} * D^{0.39515} * CW^{0.09940}$	
Old capacity model based on weaving section (IRC: 65-1976)	$Q_p = \dfrac{280 * w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}}$	

7.4 LIMITATIONS OF THE STUDY

The limitations of the present study are given below.

a) The PCU factors are restricted only for five different categories of vehicles. Other categories of vehicles (Tractor, Cycle Rickshaw, Bicycles etc.) have not been considered due to limited size of the data.

b) The lags and gaps could be extracted only on five roundabouts whereas other roundabouts are having exceptionally large circulating roadway width, entry width and weaving length.

c) Present entry capacity model is restricted only to four-legged roundabouts.

d) The study has been accomplished in northern region of India. There is a need of regional validation before applying the developed entry capacity model to other regions of the country since there may be variations in the behavioral characteristics of road users.

7.5 RECOMMENDATIONS FOR FUTURE WORK

Following are the recommendations for future work:

a) The present study evaluates the PCU factors and entry capacity of roundabout under mixed traffic flow in different cities of India. The PCU factors on roundabouts are developed for five different categories of vehicles, whereas, other cities have the significant proportion of many other categories of vehicles like LCV, Hand Rikshaw, Bicycles etc. Development of PCU factors for these vehicles may be taken as a part of future study.

b) A technique should be developed to extract the lags and gaps for the large physical dimensions of the roundabouts under the heterogeneous traffic conditions.

c) This study considers only four-legged intersections with different diameters of central-island and variation in circulating roadway width. The proposed method of estimating entry capacity may be extended to three legged and multiple legged roundabouts in different parts of the country.

d) The developed models can be validated on roundabouts in use in other regions (South, East and West) of the country. The difference, if any, can be accommodated as a modification factor.

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LIST OF PUBLICATIONS

1) Abdullah Ahmad and Rajat Rastogi (2016) "*Regression Model for Entry Capacity of a Roundabout under Mixed Traffic Condition – An Indian Case Study*" Transportation Letters: the International Journal of Transportation Research, DOI: 10.1080/19427867.2016.1203603.

2) Abdullah Ahmad, Rajat Rastogi and Satish Chandra (2015) "*Estimation of Critical Gap on a Roundabout by Minimizing the Sum of Absolute Difference in Accepted Gap Data*" Canadian Journal of Civil Engineering, Vol.42, No. 12, pp. 1011–1018, DOI: 10.1139/cjce-2014-0450.

3) Abdullah Ahmad, Srinath Mahesh and Rajat Rastogi (2014). "*Selection of Roundabout Entry Capacity Model for Indian Condition*" Urban Transport Journal, Vol.13, No. 1, pp. 79-87.

4) Srinath Mahesh, Abdullah Ahmad and Rajat Rastogi (2014). "*An Approach for the Estimation of Entry Flows on Roundabouts*" Conference on Transportation Planning and Implementation Methodologies for Developing Countries, IIT Bombay, December, 10-12.

5) Abdullah Ahmad and Rajat Rastogi (2016) "*An Approach to Deal with Heterogeneity on Roundabouts*" International Journal of Civil Engineering, IJCE (Communicated).

6) Abdullah Ahmad and Rajat Rastogi (2016) "*Calibrating HCM Model for Roundabout Capacity under Heterogeneous Traffic* " Journal of Urban Planning and Development, ASCE (Communicated).

7) Mohammed Wasim Akhtar, Abdullah Ahmad and Rajat Rastogi (2015) "*Gap Acceptance Behavior and Vehicle Speed on Roundabouts: Case Study in India*" Transportation Letters: the International Journal of Transportation Research (Communicated).

8) Abdullah Ahmad and Rajat Rastogi (2015) "*Estimation of PCU and Entry Capacity on Roundabouts – A relook at IRC-65*" Journal of the Indian Roads Congress (Communicated).

9) Abdullah Ahmad and Rajat Rastogi (2015) "*Influence of Roundabout Size and Circulating Traffic Flow on Critical Gaps, Follow-Up Times and Speeds*" Journal of the Indian Roads Congress (Communicated).

APPENDIX A:

Tests of Normality for Lagging Headway of 3W, SC and BC

Table A_1: Descriptives for lagging headway of 3W

	Statistic	Std. Error	Z-value
Mean	2.723	0.047	
Lower Bound	2.630		
Upper Bound	2.815		
5% Trimmed Mean	2.711		
Median	2.725		
Variance	0.316		
Std. Deviation	0.562		
Minimum	1.590		
Maximum	4.050		
Range	2.460		
Interquartile Range	0.750		
Skewness	0.324	0.202	**1.60**
Kurtosis	-0.343	0.401	**-0.86**

Table A_2: Descriptives for lagging headway of SC

	Statistic	Std. Error	Z-value
Mean	2.644	0.048	
Lower Bound	2.549		
Upper Bound	2.740		
5% Trimmed Mean	2.629		
Median	2.580		
Variance	0.337		
Std. Deviation	0.580		
Minimum	1.560		
Maximum	4.160		
Range	2.600		
Interquartile Range	0.820		

Skewness	0.297	0.202	**1.47**
Kurtosis	-0.304	0.401	**-0.76**

Table A_3: Descriptives for lagging headway of BC

	Statistic	**Std. Error**	**Z-value**
Mean	2.919	0.057	
Lower Bound	2.806		
Upper Bound	3.032		
5% Trimmed Mean	2.904		
Median	2.880		
Variance	0.473		
Std. Deviation	0.688		
Minimum	1.620		
Maximum	4.630		
Range	3.010		
Interquartile Range	0.790		
Skewness	0.294	0.202	1.46
Kurtosis	-0.416	0.401	-1.04

Table A_4: Tests of Normality

	Kolmogorov-Smirnov			**Shapiro-Wilk**		
	Statistic	**DF**	**Sig.**	**Statistic**	**DF**	**Sig.**
Lagging_Headway_3W	0.065	144	**0.20**	0.983	144	**0.069**
Lagging_Headway_SC	0.065	144	**0.20**	0.983	144	**0.063**
Lagging_Headway_BC	0.057	144	**0.20**	0.982	144	**0.053**

Z-values (Skewness and Kurtosis) lie within the range of critical value (-1.96 to 1.96) for lagging headway of 3W, SC and BC as given in Table A_1, A_2 and A_3 respectively. Sig. values for lagging headway of 3W, SC and BC are greater than 0.05 as given in Table A_4. This signifies that the lagging headway of 3W, SC and BC are normally distributed.

APPENDIX B:

Relationships between Entry Capacity and Circulating Traffic Flow

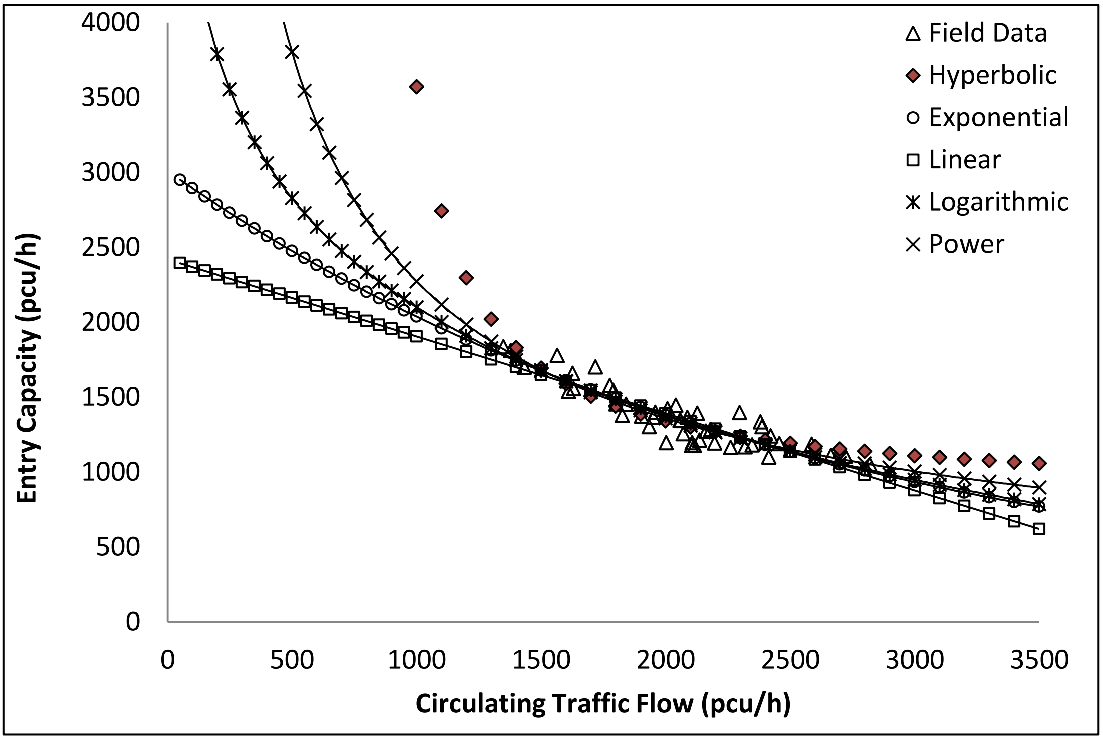

Figure B_1: Entry flow versus circulating flow at R_1

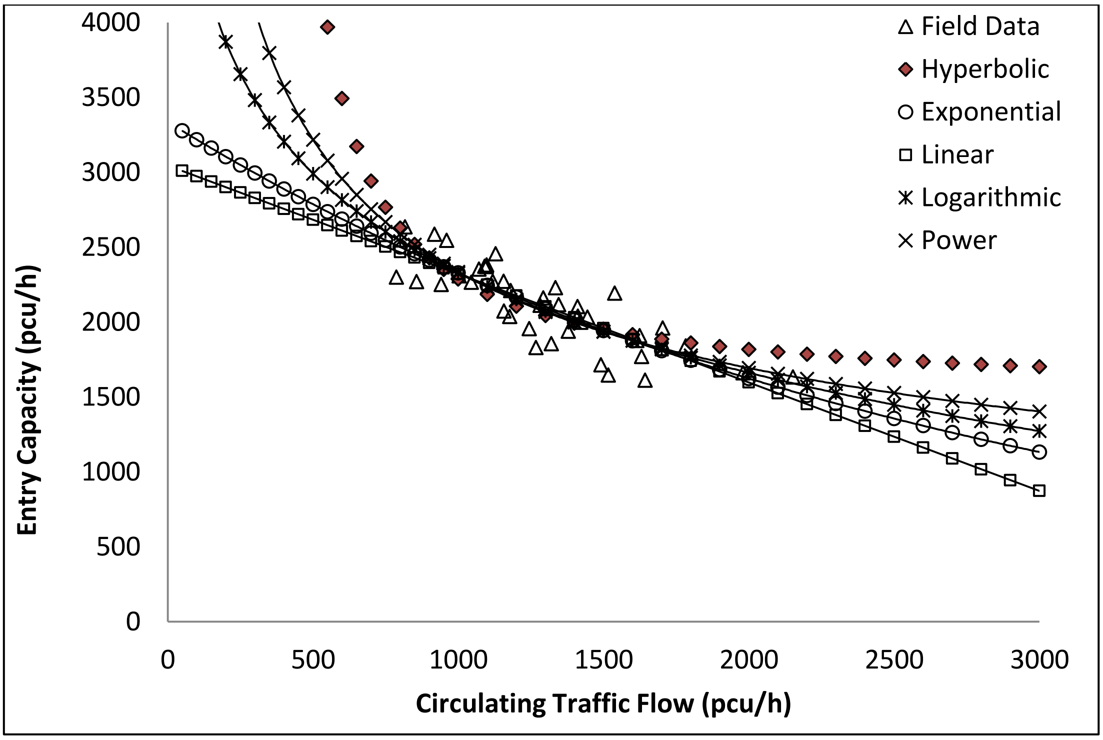

Figure B_2: Entry flow versus circulating flow at R_2

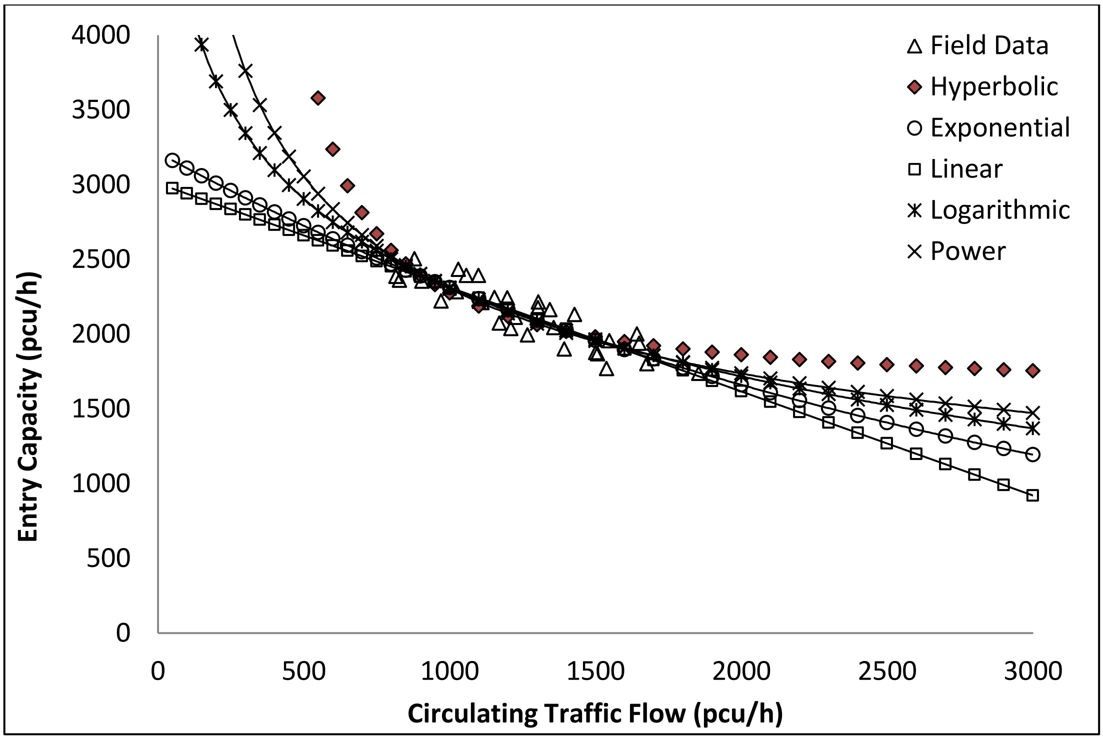

Figure B$_3$: Entry flow versus circulating flow at R$_3$

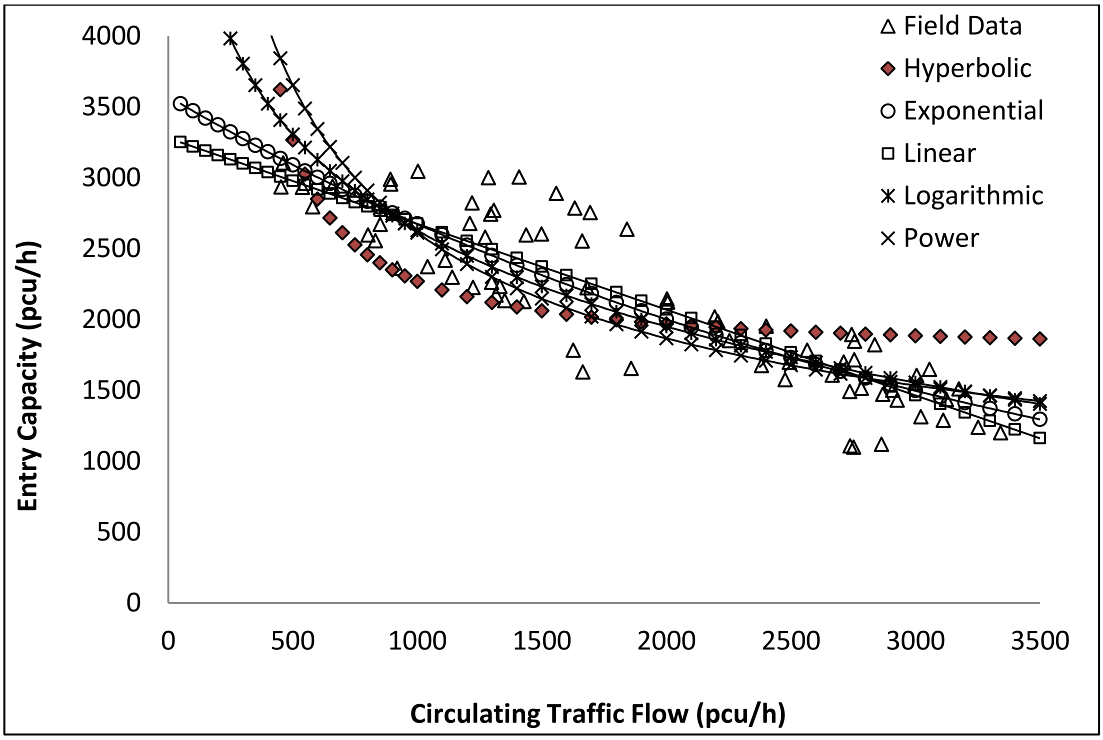

Figure B$_4$: Entry flow versus circulating flow at R$_4$

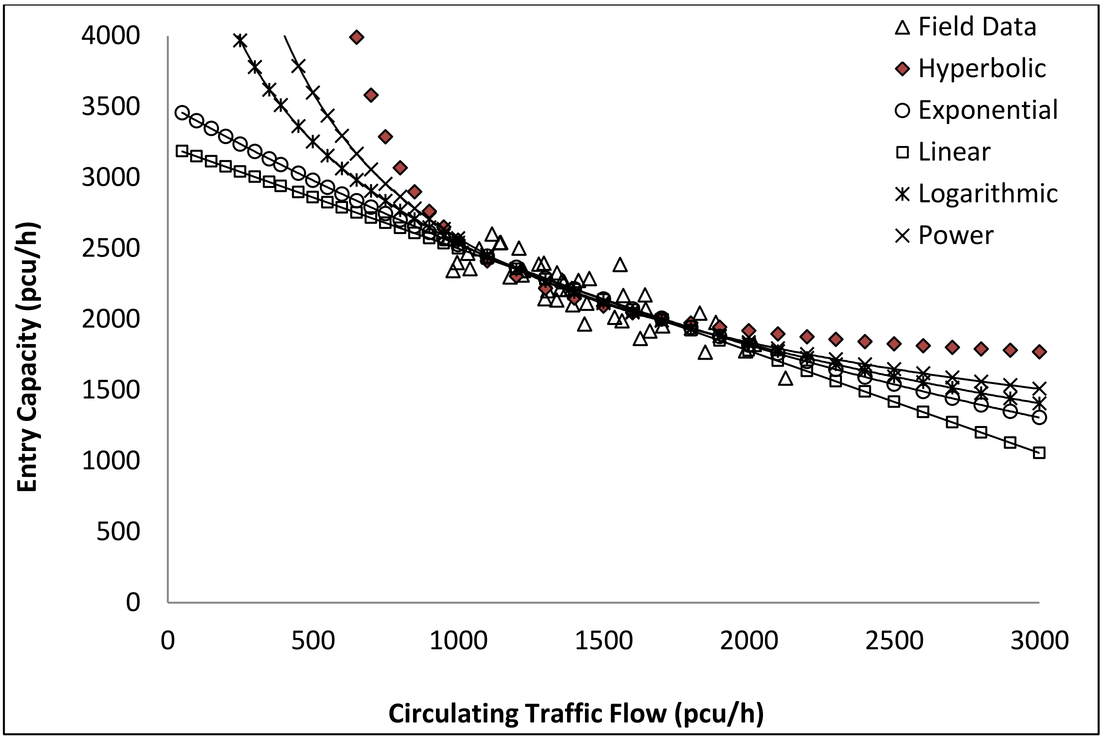

Figure B_5: Entry flow versus circulating flow at R_5

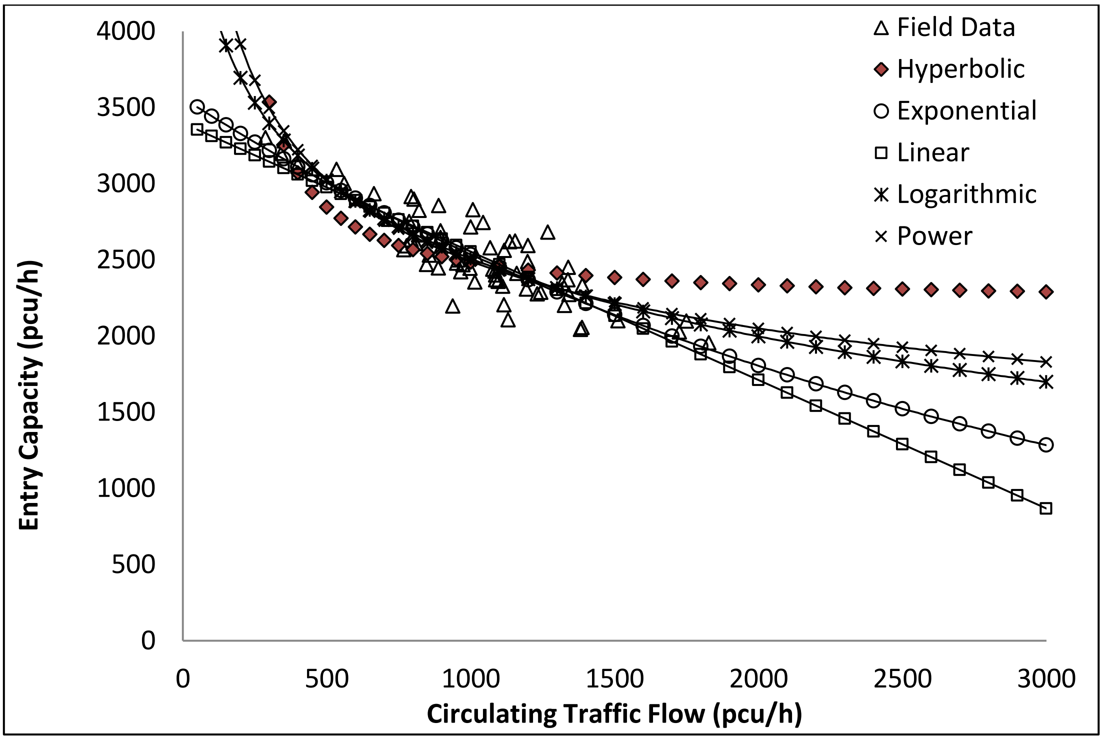

Figure B_6: Entry flow versus circulating flow at R_6

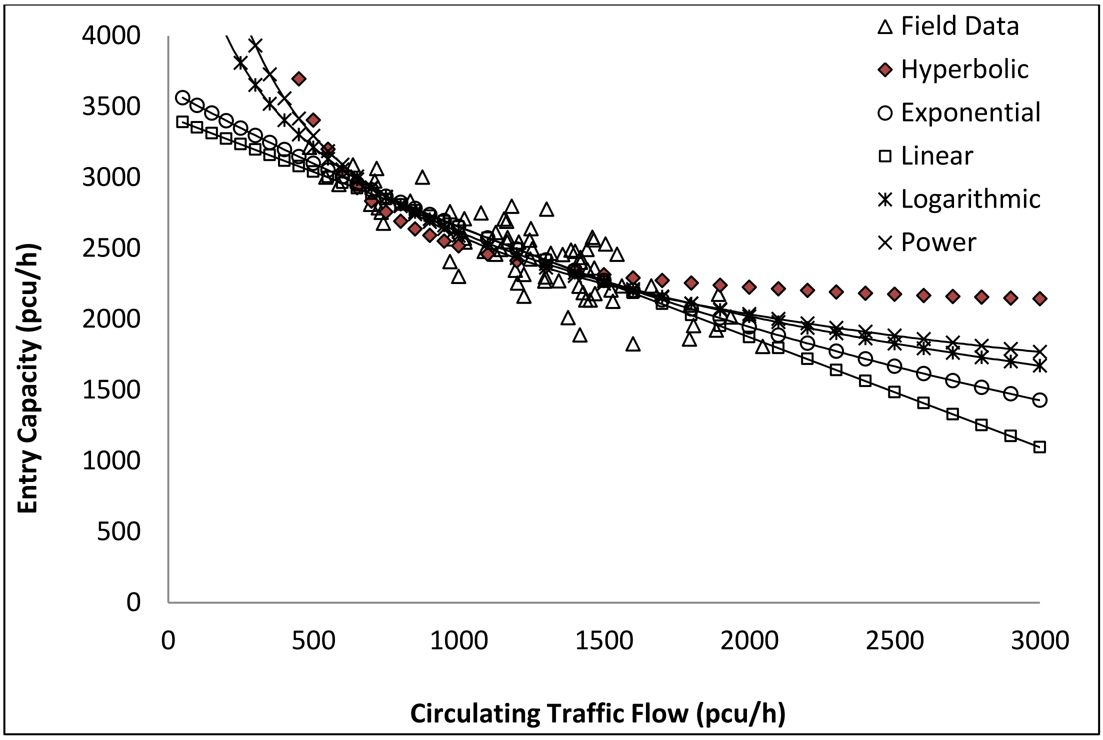

Figure B$_7$: Entry flow versus circulating flow at R$_8$

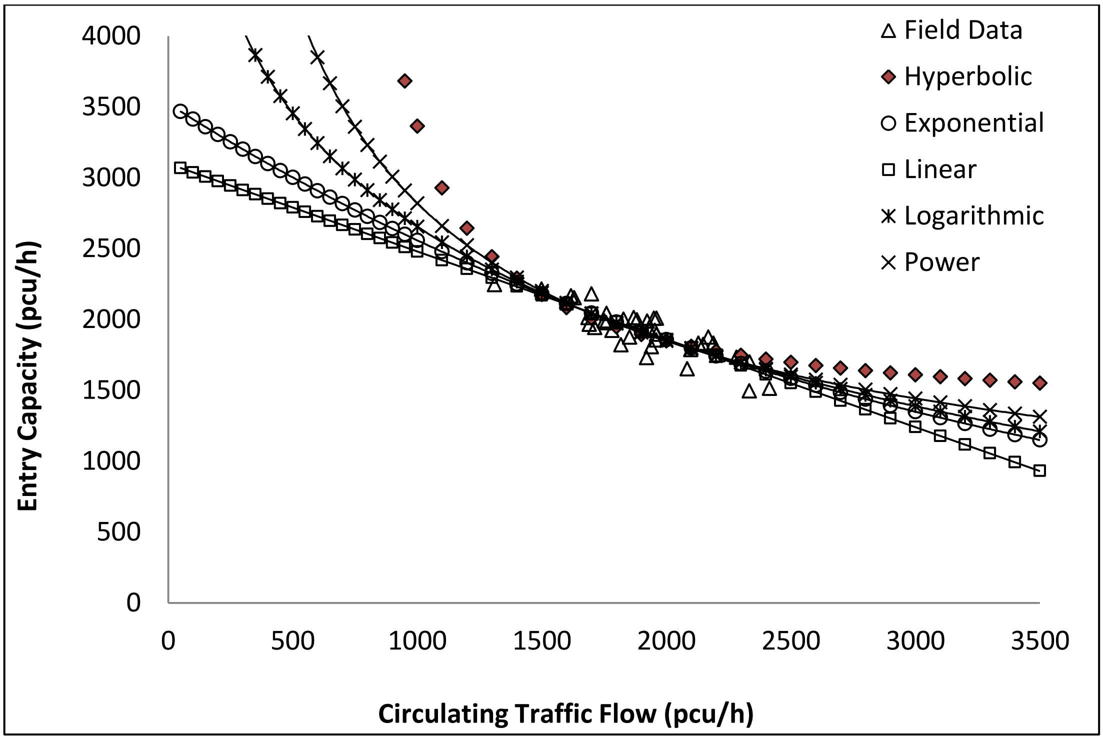

Figure B$_8$: Entry flow versus circulating flow at R$_9$

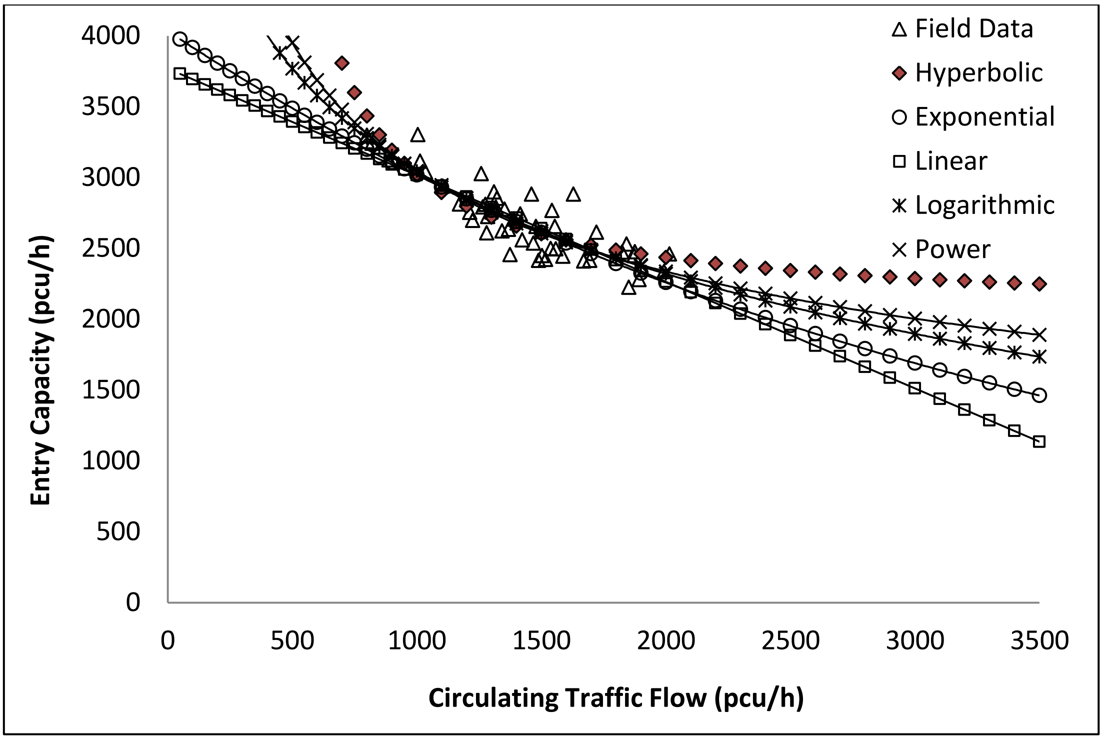

Figure B_9: Entry flow versus circulating flow at R_{10}

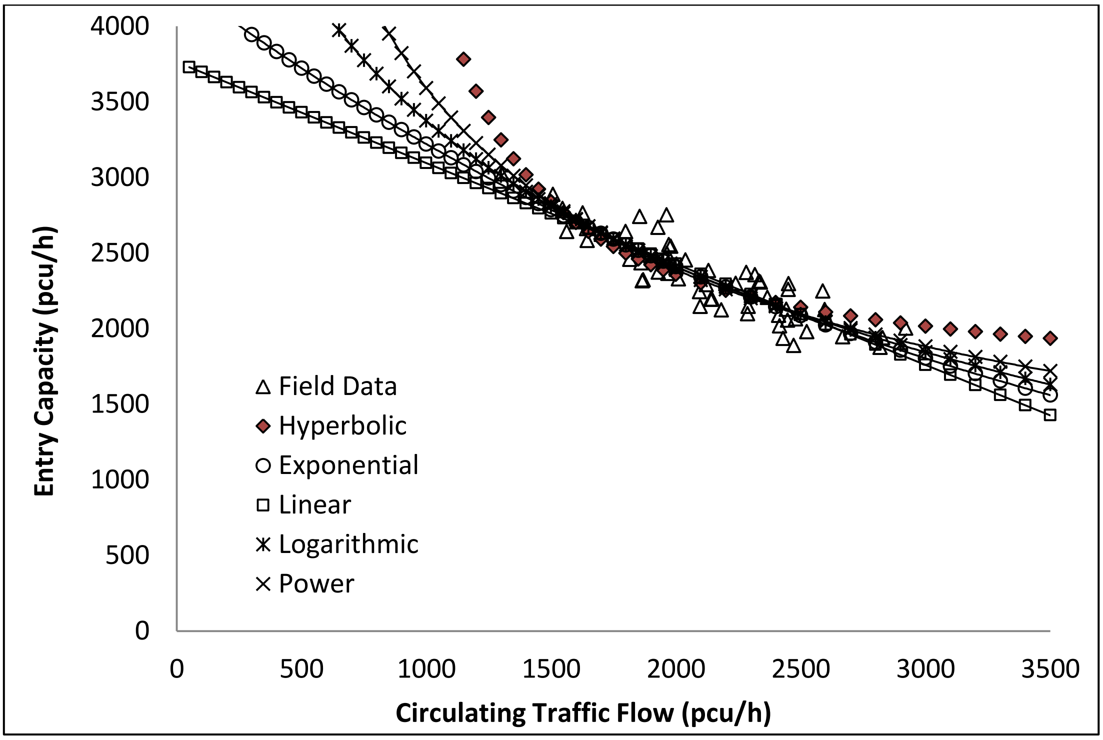

Figure B_{10}: Entry flow versus circulating flow at R_{11}

Table B_1: Hyperbolic and exponential entry capacity models for all roundabouts

Round -about ID	Hyperbolic Model	F Value	Significance F	Exponential Model	F Value	Significance F
R_1	$825*Q_c/(Q_c - 769)$	192	1.88E-17	$3009.0*e^{-0.00039*Qc}$	187	2.96E-17
R_2	$1508*Q_c/(Q_c - 341)$	63	1.11E-09	$3336.6*e^{-0.00036*Qc}$	90	1.19E-11
R_3	$1575*Q_c/(Q_c - 308)$	65	8.45E-09	$3215.5*e^{-0.00033*Qc}$	103	6.67E-11
R_4	$1738*Q_c/(Q_c - 234)$	51	5.89E-10	$3573.2*e^{-0.00029*Qc}$	262	1.12E-25
R_5	$1534*Q_c/(Q_c - 400)$	91	2.81E-12	$3514.3*e^{-0.00033*Qc}$	147	1.21E-15
R_6	$2203*Q_c/(Q_c - 113)$	94	2.83E-14	$3561.8*e^{-0.00034*Qc}$	205	8.55E-22
R_8	$1995*Q_c/(Q_c - 207)$	102	1.96E-15	$3617.8*e^{-0.00031*Qc}$	177	4.42E-21
R_9	$1275*Q_c/(Q_c - 621)$	71	6.09E-10	$3523.2*e^{-0.00032*Qc}$	106	4.05E-12
R_{10}	$2039*Q_c/(Q_c - 325)$	124	1.27E-14	$4033.6*e^{-0.00029*Qc}$	126	9.52E-15
R_{11}	$1562*Q_c/(Q_c - 675)$	161	1.12E-17	$4303.7*e^{-0.00029*Qc}$	199	1.44E-19

Table B_2: Linear and logarithmic entry capacity models for all roundabouts

Round -about ID	Linear Model	F Value	Significance F	Logarithmic Model	F Value	Significance F
R_1	$-0.5144*Q_c + 2420$	155	7.60E-16	$-1049*\ln(Q_c) + 9347$	195	1.38E-17
R_2	$-0.7244*Q_c + 3046$	85	2.25E-11	$-959*\ln(Q_c) + 8951$	89	1.33E-11
R_3	$-0.6963*Q_c + 3010$	104	6.19E-11	$-857*\ln(Q_c) + 8232$	97	1.34E-10
R_4	$-0.6053*Q_c + 3280$	280	1.54E-26	$-977*\ln(Q_c) + 9380$	203	1.28E-22
R_5	$-0.7216*Q_c + 3220$	151	7.64E-16	$-1030*\ln(Q_c) + 9653$	138	3.51E-15
R_6	$-0.8428*Q_c + 3396$	187	8.23E-21	$-737*\ln(Q_c) + 7599$	206	7.74E-22
R_8	$-0.7776*Q_c + 3429$	175	5.59E-21	$-860*\ln(Q_c) + 8557$	167	1.89E-20
R_9	$-0.6195*Q_c + 3099$	109	2.67E-12	$-1153*\ln(Q_c) +10619$	102	6.37E-12
R_{10}	$-0.7523*Q_c + 3768$	115	4.06E-14	$-1045*\ln(Q_c) +10263$	138	1.87E-15
R_{11}	$-0.6676*Q_c + 3763$	186	5.52E-19	$-1393*\ln(Q_c) +12997$	203	9.20E-20

Table B_3: Power entry capacity models for all roundabouts

Roundabout ID	Power Model	F Value	Significance F
$\mathbf{R_1}$	$385127{*}\ Q_c^{-0.743}$	215	2.37E-18
$\mathbf{R_2}$	$57169{*}\ Q_c^{-0.463}$	85	2.24E-11
$\mathbf{R_3}$	$38325{*}\ Q_c^{-0.407}$	91	2.70E-10
$\mathbf{R_4}$	$73924{*}\ Q_c^{-0.484}$	152	2.18E-19
$\mathbf{R_5}$	$73282{*}\ Q_c^{-0.485}$	128	1.33E-14
$\mathbf{R_6}$	$17352{*}\ Q_c^{-0.281}$	189	6.73E-21
$\mathbf{R_8}$	$28451{*}\ Q_c^{-0.347}$	156	1.05E-19
$\mathbf{R_9}$	$190598{*}\ Q_c^{-0.610}$	95	1.60E-11
$\mathbf{R_{10}}$	$41648{*}\ Q_c^{-0.379}$	142	1.16E-15
$\mathbf{R_{11}}$	$208572{*}\ Q_c^{-0.588}$	202	9.95E-20

APPENDIX C:

Tests of Normality for Field and Model Entry Capacity

Table C_1: Descriptives for field entry capacity

	Statistic	**Std. Error**	**Z-value**
Mean	2114.67	53.52	
Lower Bound	2006.67		
Upper Bound	2222.68		
5% Trimmed Mean	2112.59		
Median	2131.00		
Variance	123159.18		
Std. Deviation	350.94		
Minimum	1501.000		
Maximum	2770.000		
Range	1269.000		
Interquartile Range	506.000		
Skewness	0.109	0.361	**0.30**
Kurtosis	-0.784	0.709	**-1.11**

Table C_2: Descriptives for entry capacity model of medium size roundabouts

	Statistic	**Std. Error**	**Z-value**
Mean	2096.98	47.16	
Lower Bound	2001.81		
Upper Bound	2192.15		
5% Trimmed Mean	2093.62		
Median	2072.00		
Variance	95626.36		
Std. Deviation	309.24		
Minimum	1481		
Maximum	2751		
Range	1270		
Interquartile Range	391		

Skewness	0.274	0.361	**0.76**
Kurtosis	-0.435	0.709	**-0.61**

Table C_3: Tests of Normality

	Kolmogorov-Smirnov			Shapiro-Wilk		
	Statistic	**DF**	**Sig.**	**Statistic**	**DF**	**Sig.**
Field_Entry_Capacity	0.064	43	**0.20**	0.97	43	**0.323**
Model_Entry_Capacity	0.072	43	**0.20**	0.98	43	**0.649**

Z-values (Skewness and Kurtosis) lie within the range of critical value (-1.96 to 1.96) for field and model entry capacity as given in Table C_1 and C_2 respectively. Sig. values for field and model entry capacity are greater than 0.05 as given in Table C_3. This signifies that the field and model entry capacity values are normally distributed.

APPENDIX D:

Tests of Normality for Field and Regression Model Entry Capacity

Table D$_1$: Descriptives for field entry capacity

	Statistic	Std. Error	Z-value
Mean	2114.67	53.52	
Lower Bound	2006.67		
Upper Bound	2222.68		
5% Trimmed Mean	2112.59		
Median	2131.00		
Variance	123159.18		
Std. Deviation	350.94		
Minimum	1501.000		
Maximum	2770.000		
Range	1269.000		
Interquartile Range	506.000		
Skewness	0.109	0.361	**0.30**
Kurtosis	-0.784	0.709	**-1.11**

Table D$_2$: Descriptives for regression entry capacity model

	Statistic	Std. Error	Z-value
Mean	2094.93	47.12	
Lower Bound	1999.83		
Upper Bound	2190.03		
5% Trimmed Mean	2091.60		
Median	2070.00		
Variance	95479.54		
Std. Deviation	309.00		
Minimum	1480		
Maximum	2748		
Range	1268		
Interquartile Range	391		

Skewness	0.272	0.361	**0.75**
Kurtosis	-0.439	0.709	**-0.62**

Table D_3: Tests of Normality

	Kolmogorov-Smirnov			Shapiro-Wilk		
	Statistic	**DF**	**Sig.**	**Statistic**	**DF**	**Sig.**
Field_Entry_Capacity	0.064	43	**0.20**	0.97	43	**0.323**
Model_Entry_Capacity	0.072	43	**0.20**	0.98	43	**0.648**

Z-values (Skewness and Kurtosis) lie within the range of critical value (-1.96 to 1.96) for field and model entry capacity as given in Table D_1 and D_2 respectively. Sig. values for field and model entry capacity are greater than 0.05 as given in Table D_3. This signifies that the field and model entry capacity values are normally distributed.

APPENDIX E:

Calculation of Entry Capacity on Roundabouts

An example of the application of the procedure to estimate the entry capacity of a roundabout (R_7) is presented here with the following features:

Central Island Diameter = 50 m;

Circulating Roadway Width = 10 m;

Entry Width = 14.7 m;

Exit Width = 15.5 m;

Circulating Traffic Flow = 2000 pcu/h;

<u>Method 1: Based on the circulating traffic flow</u>

The roundabout as given in example comes under medium size roundabout category. Therefore, equation (6.2) will be used for estimating the entry capacity.

$$Q_e = 3483 * e^{-0.00030 * Q_c}$$

$$Q_e = 3483 * e^{-0.00030 * 2000}$$

The estimated entry capacity (Q_e) = **1912 pcu/h**

<u>Method 2: Based on the calibrated HCM (2010) model</u>

The calibrated HCM (2010) model (parameters given in Table 6.8) will be used for estimating the entry capacity on roundabout.

$$Q_e = f_a * A * e^{-B * Q_c}$$

Where,

A = 3147

B = 0.00034

The adjustment factor for 50 m central island diameter is 1.133 (taken from Table 6.8).

$$Q_e = 1.133 * 3147 * e^{-0.00034 * 2000 Q_c}$$

The estimated entry capacity (Q_e) = **1806 pcu/h**

Method 3: Based on circulating traffic flow and physical parameters of roundabout

The developed regression equation (7.8) will be used for estimating the entry capacity on roundabout.

$$Q_e = 589.90 * e^{-0.00030Q_c} * D^{0.39515} * CW^{0.09940}$$

$$Q_e = 589.90 * e^{-0.00030*2000} * 50^{0.39515} * 10^{0.09940}$$

The estimated entry capacity (Q_e) = **1910 pcu/h**

Table E_1: Entry capacity estimated at different circulating traffic flow

Circulating Traffic Flow (pcu/h)	Entry Capacity (pcu/h)		
	Method 1	Method 2	Method 3
200	3280	3331	3277
400	3089	3112	3086
600	2909	2908	2906
800	2740	2716	2737
1000	2580	2538	2578
1200	2430	2371	2428
1400	2288	2215	2286
1600	2155	2070	2153
1800	2030	1933	2028
2000	1912	1806	1910
2200	1800	1688	1798
2400	1695	1577	1694
2600	1597	1473	1595

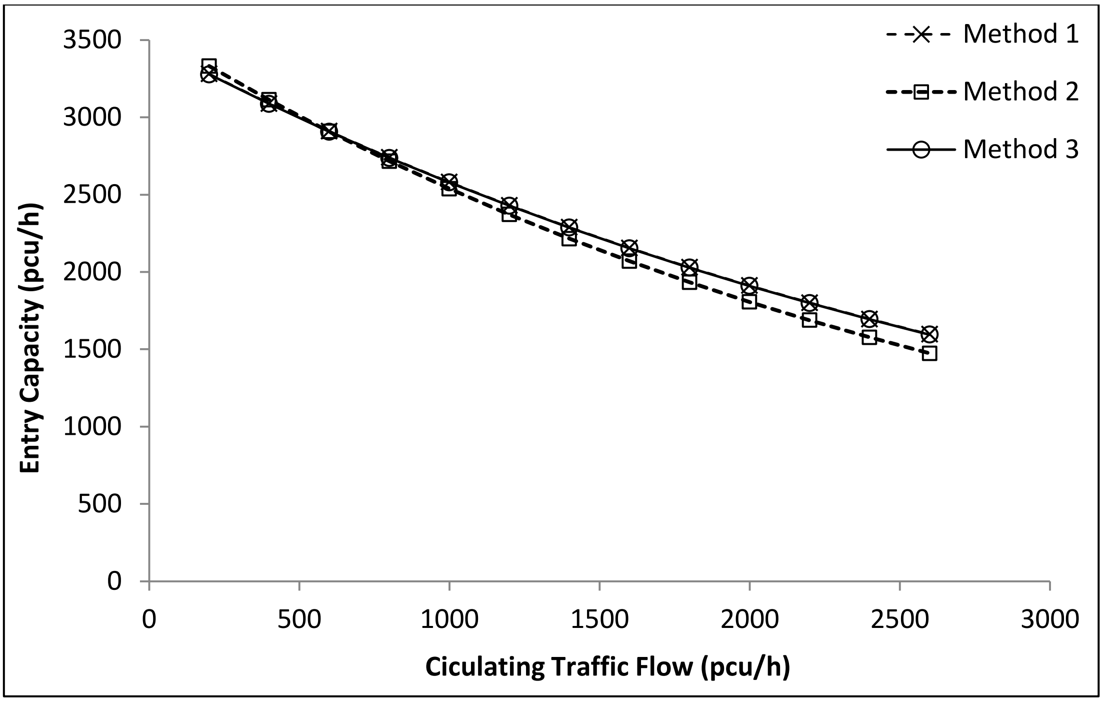

Figure E_1: Entry capacity versus circulating traffic flow