# Operational Analysis of Roundabouts under Mixed Traffic Flow Condition

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By

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# NATIONAL INSTITUTE OF TECHNOLOGY

# ROURKELA-769008

## CERTIFICATE

This is to certify that project entitled, **— Operational Analysis of Roundabouts under Mixed Traffic Flow Condition** ‖ submitted by **R.Mallikarjuna** in partial fulfilment of the requirements for the award of **Master of Technology** Degree in **Civil Engineering** with specialization in **Transportation Engineering** at National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance. To the best of my knowledge, the matter embodied in this Project review report has not been submitted to any other University/ institute for award of any Degree or Diploma.

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# ABSTRACT

This thesis addresses the most important element of operational performance of roundabout traffic intersections in Rourkela on capacity analysis. The movements of the vehicles were observed at 5 roundabouts along ring road in Rourkela. Gap acceptance and follow up time were estimated for cars for one hour analysis. The relation between a roundabout performance measure and capacity is expressed in terms of degree of saturation (volume – Capacity ratio). The capacity analysis is done based on gap acceptance method that is adopted by Tanner based on the HCM 2010. The traffic movement data with vehicle characteristics were collected from 5 roundabouts in Rourkela. These 5 roundabouts are directly related to their approach leg numbers.

Approach entry capacity has been analysed for all 5 roundabouts at their legs. Effective capacity verses entry flow relationship have been developed in order to find out the causes of their over Saturation (v/c ratio greater than 0.85) And the result indicates; number of entry lanes, number of circulatory lanes and high traffic flow are the major causes of their over saturation.

Tanner models use the gap-acceptance theory (or critical headway) to simulate the behaviour of entering vehicles and vehicles circulating within the roundabout. Finding a safe gap (or headway) within circulating traffic stream to enter the roundabout is the controlling variable that determines the ability of approach vehicles to enter the roundabout. Current research work on roundabout models mostly concentrates on determining the capacity of an approach based on the entering and circulating flows. Approach capacity is calculated as a mathematical function of critical headway and follow-up headway.

Several roundabout capacity models exist and can be classified into two broad categories - theoretical and empirical. The Tanner model is based on gap- acceptance theory with gap-acceptance parameters. The Highway Capacity Manual (HCM 2010) roundabout tanner capacity model is an analytical (exponential regression) model with clear basis in gap-acceptance theory. The NCHRP Report 572model is based on empirical exponential regression) capacity model with no explicitly.

Capacity analysis results indicated that out of 5 roundabouts 1 of them has greater than 0.85 degree of saturation and this roundabout has critical for traffic flow because this has degree of saturation more than 0.85. This 0.85 value is recommended by analysis procedure of

tanner model. So roundabouts are designed to operate at less than 85 percent of their estimated capacity.

**Key words**: Round about, capacity, vehicles, Gap acceptance, Follow up time, Delay, Queue length, LOS, Degree of Saturation

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# Symbols

| | | |
|---|---|---|
| c | : | capacity of the subject lane, veh/h |
| $c_e$ | : | The entry capacity (pcu/h) |
| d | : | Average control delay, sec/veh |
| F(t) | : | the cumulative probability that the headway is less than or equal to t |
| $F_a(t)$ | : | Accepted gap t |
| $F_r(t)$ | : | Maximum rejected gap $t_r$ |
| $q_c$ | : | The circulating flow (pcu/h) |
| $Q_{95}$ | : | Queue length, veh |
| T | : | Time period [T=1 for hour analysis, T=0.25 for 15 minutes analysis |
| $t_c$ | : | The critical headway (sec) |
| $t_f$ | : | The follow-up time (sec) |
| $x$ | : | Volume to capacity ratio of the subject lane |
| α | : | the proportion of free vehicles |
| Δ | : | the minimum headway between the circulating vehicles (sec) |
| ƛ | : | the decay constant (sec) |

# Abbreviations

| | | |
|---|---|---|
| FHW | : | Federal Highway Administration |
| HCM | : | Highway Capacity Manual |
| LOS | : | Level of Service |
| NCHRP: | | National Cooperative Highway Research Program |
| MOE | : | Measure of Effectiveness |
| PCU | : | Passenger Car Unit |
| TRB | : | Transportation Research Board |
| TPR | : | transportation planning r |

# Chapter1

# Introduction

## 1. 1 General

A roundabout is an alternative form of intersection traffic control. Roundabouts are generally circular in shape, characterized by yield on entry and circulation around a central island. Roundabouts are appropriate for many intersections including locations experiencing high number of crashes, long traffic delays, and approaches with relatively balanced traffic flows. Roundabouts have the potential to resolve various traffic flow problems. Traffic volume on one approach is significantly higher that it prevents vehicles at any other approach from entering the roundabout especially at a downstream approach or the next following approach. Evaluation of junction capacity of roundabout is very important since it is directly related to delay, level of service, accident, operation cost, and environmental issues. There are three legs, four legs, five legs and six legs roundabouts in Rourkela and most of them have served more than 15 years. Since little attention has been paid to the design and capacity evaluation of the roundabouts, no one knows knows their capacities or level of services.

Tanner models use the gap-acceptance theory (or critical headway) to simulate the behaviour of entering vehicles and vehicles circulating within the roundabout. Finding a safe gap (or headway) within circulating traffic stream to enter the roundabout is the controlling variable that determines the ability of approach vehicles to enter the roundabout. Current research work on roundabout models mostly concentrates on determining the capacity of an approach based on the entering and circulating flows. Approach capacity is calculated as a mathematical function of critical headway and follow-up headway. This method is not sensitive to roundabout geometric parameters such as inscribed circle diameter, entry angle, etc. In addition, the level of traffic stream performance itself can influence driver behaviour and increasing the complexity of modelling roundabout operations.

Critical headway and follow-up headway are two important parameters to perform operational analyses of roundabout. Critical headway at roundabouts represents the minimum time interval in circulating flow when an entering vehicle can safely enter the roundabout. A driver would enter the roundabout when faced with any headway equal to or greater than the critical headway. Follow-up headway is the minimum headway between two entering vehicles, which can be calculated by the average difference between passage times of two entering vehicles accepting the same mainstream headway under a queued condition. In other words the follow-up headway is equal to the inter-vehicle headway on an approach at capacity. Increasing the follow-up time and critical gap decreases capacity.

Several roundabout capacity models exist and can be classified into two broad categories - theoretical and empirical. The Tanner model is based on gap- acceptance theory with gap-acceptance parameters. The Highway Capacity Manual (HCM 2010) roundabout tanner capacity model is an analytical (exponential regression) model with clear basis in gap-acceptance theory. The NCHRP Report 572model is based on empirical exponential regression) capacity model with no explicitly.

Therefore, road authorities and other concerned bodies need to conduct a comprehensive capacity and delay study of every roundabout. so they can think with solutions for the traffic congestions, traffic delays, queue length, Degree of Saturation and level of services.

***Vehicle Safety*:**

Roundabouts have fewer conflict points than traditional intersections and also require lower operating speeds for both the driver entering the roundabout and the driver driving in the roundabout. A conflict point is defined as a location where the paths of two motor vehicles or a vehicle and pedestrian queue, diverge, merge, or cross each other. The following figure is used to illustrate the reduction in conflict points:

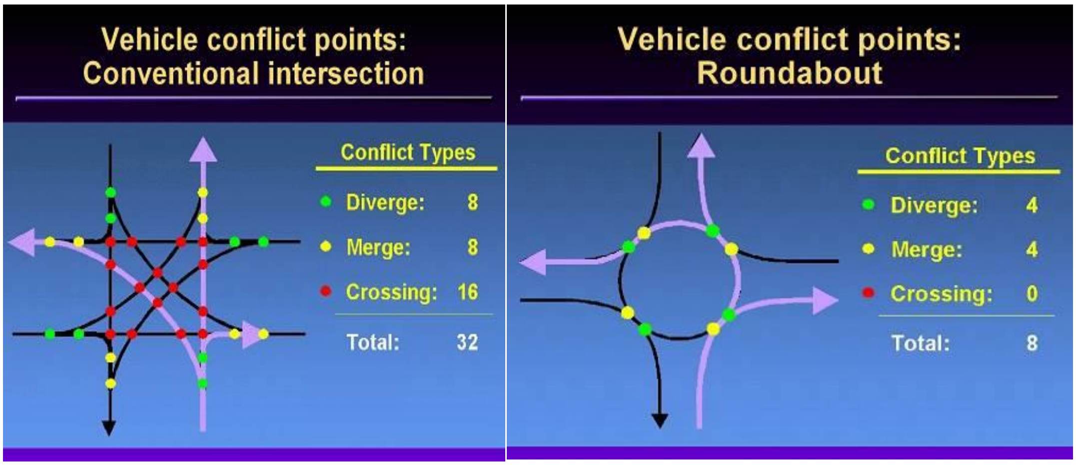



**Figure1-1: Vehicle Conflict Point Comparison**

At four-way stop roundabouts have about a 75% decrease in vehicle conflict points compared to a traditional intersection. Three types of conflicts are defined in the report: merge and diverge conflicts, and crossing conflicts. Crossing conflicts are frequently the most serious in terms of vehicular injuries and fatalities. At a traditional intersection accidents are frequently happen when a driver neglects to stoplight or stop sign. By eliminating crossing conflicts, roundabouts were designed dramatically lower the incidents of injuries and fatalities associated with conflict points

## 1.2 Problem Statement

Now days it is common to see traffic congestion at intersections of roundabouts in Rourkela at peak hours in the morning and evening. Hence the traffic police need to intervene in the situation to regulate the traffic flow. Otherwise it would be practically difficult to have normal traffic flows, particularly at roundabout junctions, which is more dependent on driver behaviour and balanced traffic flow between the approaches. This problem will continue and it may more difficult in the future due to the rapid growth of population and vehicle numbers in Rourkela. Poor road planning and sub-standard geometric conditions of roundabouts have a significant effect on roundabout capacity and traffic congestion.

Therefore, it is necessary to evaluate the capacity of roundabouts for proper traffic operation.

**Some of the problems related to capacity of roundabouts are**:

- Necessarily geometric features of roundabouts such as flare and apron do not exist.
- In few roundabouts, there are visibility problem caused by plants or elevated masonry. This causes the entering driver to delay on entering the circulating traffic and affecting the capacities of the roundabouts.
- Central islands of roundabouts are accessed by pedestrians.
- Absence of road marking signs and lights.

## 1.3 Objectives

The specific objectives of this research are:

- To compile available information regarding capacity analysis of roundabouts through literature review
- To select the appropriate methodology to evaluating the capacity of roundabouts for a mid-sized cities in Indian context

- To define the capacity and service levels of roundabout junctions for a mid-sized cities in Indian context

## 1.4 Organisation of report

This thesis consists of the six chapters:

- The first chapter of this thesis gives a general introduction of the overall thesis content and the general background of parameters involved in the analysis of roundabouts.
- The second chapter reviews the relevant literatures related to gap acceptance parameter and capacity studies.
- Chapter 3 discusses the study methodology carried out for this study.
- Chapter 4 discusses the study area and data collection procedure.
- Chapter 5 discusses the data analysis and results
- Chapter 6 summary, conclusion and scope for future work.

# Chapter2

# Review of Literature

Some of the literatures reviewed for this study on operational analysis of roundabouts are discussed in this chapter.

## 2.1 General

Siegloch was developed a linear-regression technique which used the gap data from queuing conditions to estimate both the critical gap and the follow-up headway in 1973. This technique recorded the $i^{th}$ gap with size $t_i$ and number$N_i$ of accepted vehicles. Then all data were categorized according to the number of accepted vehicles. Within each category the average gap size was calculated. As a result, a reduced data set of average gap size versus number of accepted vehicles was generated. Finally the average gap size was fit as a linear function of the number of accepted vehicles. Although being straightforward and generally giving good estimations, this method applied only to those conditions where queues appeared in the minor stream. Polus and Shmueli developed an entry-capacity model for roundabouts that includes outside diameter and circulating flow as input parameters in 1997. Six small to medium sized roundabouts in urban and suburban areas of Israel were included in this study. A separate regression model was developed for each roundabout studied because it was believed that the geometric characteristics of each site significantly affect its capacity. A general form of an exponential regression equation could be developed. Results from the developed model were compared with those obtained from Australian and German models. Flow and geometric data were collected from the six study sites. The capacity of each entry was defined as the maximum number of vehicles that can enter the roundabout in 1 hour under continuous queue conditions.

Polus and Shmueli (1999) further examined and evaluated the capacity model previously developed in their 1997 study. In addition, the study estimated a gap size above which gaps are not relevant to the gap acceptance process and evaluated the gap acceptance behaviour of drivers entering roundabouts as their waiting time on the approach leg increased. Al-Masaeid and Faddah developed an empirical model for estimating entry capacity as a function of circulating traffic and geometric characteristics in 1997. Ten roundabouts located throughout Jordan were studied. Regression analysis was used to develop the entry-capacity model and its performance was then compared with results of German, Danish, and French capacity models. Al-Masaeid used a logit analysis to develop models for estimating critical gap and move-up time at roundabouts in 1999. The first model predicts the probability that a random driver entering a roundabout will accept a given gap in the circulating stream based on geometric and gap characteristics. The second model estimates move-up time based on roundabout geometry and circulating traffic characteristics. Results from these models were incorporated into the Australian and German gap theoretical models to determine which of the two theoretical models is more appropriate for use in Jordan.

Hagring proposed a new capacity model for two-lane roundabouts based on previous studies (Hagring 1996, 1998) at Swedish roundabouts on the effects of origin- destination (OD) flows. The developed model was tested on two synthetic data sets and compared with another OD model proposed by Akçelik et al. (1996) and Akçelik (1997). The previous work by Hagring studied critical gap differences between the inner and outer entry lanes at two-lane roundabout approaches. A simplified model was developed relating critical gap to the length and width of the weaving section between adjacent approaches. The capacity model presented and evaluated in the current study was first developed in these older studies. Flannery et al. developed equations estimating the mean and variance of service time for a vehicle in the first position at an entry of a single-lane roundabout. With these estimates, the Pollaczek-Khintchine formula and Little’s law may then be used to estimate the average number of queued vehicles and the average total waiting time per vehicle, respectively. Service time is defined as the time spent in the first position of the queue prior to entering the circulating stream and includes the time spent waiting for an acceptable gap in the circulating stream, travel time to enter the circulating stream, and the headway for the subsequent circulating vehicle.

## 2.2 Analytical (Gap Acceptance) Vs Empirical Regression

There exist two distinct theories depends upon roundabout capacity/delay equations. These theories are the analytical or gap acceptance method, and the empirical method, which is based on geometrics and regression.

In Kimber’s initial laboratory report (1980) he states that the dependence of entry capacity on circulating flow depends on the roundabout geometry. Kimber defines five geometric parameters which have an effect on the capacity. These are entry width and flare, the inscribed circle diameter (a line that bisects the centre island and the circulating lane twice) and the angle and radius of the entry. In Kimber's 1989 paper he states that gap acceptance is not a good estimator of capacity in the United Kingdom. He also states that single-lane entries are the basis for the simplest case for gap acceptance models, while empirical models apply also to multilane entries. Kimber reasons that gap acceptance models do not increase capacity correctly when additional entry lanes are added. Kimber makes two interesting comments in his paper the first being that many circumstances exist where driver response to yield signs conforms to gap acceptance assumptions. He is not given sufficient description of gap acceptance roundabouts. The main flaw of the gap acceptance theory is that it poorly evaluates capacity for roundabouts. The second comment by Kimber is that because of driver behaviour and geometric variation is not safe to transfer theories from one country to another. Fisk, in a 1991 article, agreed that regression models should not be transferred from region to region or between roundabouts of different geometrical configurations..

Akcelik (1998) writes gap acceptance method presented in his report improves capacity prediction during heavy flow conditions and especially for multilane roundabouts with uneven approach demands. Many of the additional parameters used in SIDRA gap acceptance model based on the gap acceptance theory. The parameters that deal with the entering traffic stream include the inscribed diameter, average entry lane width, the number of circulating and entry lanes, the entry capacity (based on the circulating flow rate), and the ratio of the entry flow to the circulating flow. These additional model elements demonstrate the detailed nature of the SIDRA model. Another important component of Akcelik's formulation is the identification of the dominant and subdominant entry lanes based on their flows. The dominant lane has the highest flow rate, and all others are subdominant. The purpose of this component is that dominant and subdominant entry lanes can have different critical gap and follow up times. SIDRA also includes a passenger car equivalent (PCE) for heavy vehicles.

## 2.3 **Reviews on Capacity and Delay**

Roundabout capacity and delay analysis can be performed at several levels of detail. Akcelik (1998) mentions three methods for measurement capacity. These include analysis by total approach flow used in ARCADY, the British empirical regression based on simulation. Akcelik uses the lane-by-lane method for the purpose of allowing improved geometric modelling of the intersection. He points out that recognition of unequal lane utilization is important because it affects the capacity and performance of the roundabout.

Fisk states the lane utilization for entering lanes should be determined using travel time minimization or by equalizing queue lengths. It is also mentioned that the left lane will be served at a faster rate than the right lane and because of this travel time minimization would be a better predictor. Akcelik's use of dominant and subdominant lanes .so this is problem from a different angle. Fisk and Akcelik both recommend using a different critical gap and follow uptime for each lane. In Akcelik's model lane utilization ratio is determined by the degrees of saturation of the lanes. Lane group capacity is then calculated and flow rate for each lane is determined. Morlok (1978) states that behavioural studies of motorists indicate that motorists will choose their route based on the minimum travel time. This is compliments Fisk's statement of minimizing travel time. Minimizing travel time appears to be the most appropriate method to determine lane utilization for this formulation. Fisk describes the problem to be a mini-traffic assignment problem. For this model to be implemented into a travel forecasting model

## 2.4 Critical Gap and Follow up Time

Cassidy et al (1995) state that it is not possible to directly observe the mean critical gap. This report also states that there is no evidence that a single-valued gap acceptance function cannot be used to model driver behaviour reliably at a stop sign. Tian et al (2000) consider the many variables that can effect critical gap and follow up time. They state that geometry, turning movements, vehicle type and approach grade were found to affect these parameters. The Federal Highway Administration (FHWA) (2000) states that it is not desirable to locate roundabouts where grades are greater than four percent. Therefore, it is assumed that most roundabouts will not deal with grade as a factor.

The Transportation Research Board (HCM 1997) presents its critical gap range as 4.1 to 4.6 seconds, and the follow up time as 2.6 to 3.1 seconds. These values are for only single lane roundabouts. List et al (1994) determined the average critical gap to be from 2.8 to 4.0 seconds and the follow up time to range from 1.8 to 3.7 seconds. These values were most representative of the right lane. As stated earlier, the right lane will have a smaller critical gap and follow up time than the left lane, as the vehicles in the left lane have to cross the outside circulating lane. All of these gaps are consider smaller than the recommended critical gaps and follow up times for two-way stop controlled intersections. The Transportation Research Board lists these as 6.9 and 3.3 seconds for a right turn onto a four-lane road, which is analogous to the circulatory roadway of a multilane roundabout. Roundabout gaps and follow up times are smaller due to two reasons. The first is the ability for some vehicles to enter the circulating roadway without coming to a complete stop. If there are no queued vehicles in the entry lane the yield control allows vehicles to only slow to the speed at which they can safely negotiate the roundabout. The second reason is the flare of the roundabout.

# Chapter3
# Study Methodology

## 3.1 General

Capacity is the main determinant of the performance measures such as delay, queue length, critical headway and follow up time. The relationship between a given performance measure and capacity is often expressed in terms of degree of saturation (demand volume- capacity ratio).

### *I. Gap and Lag at Roundabouts*

A gap is defined as the time difference between two successive circulating vehicles passing the same reference point in a roundabout. The reference points most often chosen are the points where circulating vehicles either intersect entering vehicles (conflicting line) or exit the roundabout (exiting line). If an entering vehicle arrives at the yield bar after the gap has already started the remainder of the gap is termed lag. The National Cooperative Highway Research Program (NCHRP) Report 572 defines a lag as "the time from the arrival of the entering vehicle at the roundabout entry to the arrival of the next conflicting vehicle".

### *II. Critical Gap at Roundabouts*

Based on the above definition of gap (and lag), the critical gap is defined as the minimum gap that an entering driver will accept for entering the roundabout. The critical gap directly measuring in the field is not possible. In theory gap accepted by a driver is greater than or equal to his/her critical gap; a rejected gap is smaller than the critical gap. Therefore, although accepted and rejected gaps can be measured in the field, a critical gap cannot be directly measured. Critical gaps are estimated based on the quantified accepted and rejected gaps, and the point where accepted and rejected gaps are equally probable.

### *III. Follow-up Headway at Roundabouts*

Follow-up headway is defined as the time difference between two successive vehicles in the same lane entering the roundabout and using the same gap. The follow-up headway is similar in concept to the saturation headway used at signalized intersections. The saturation headway refers to "the average headway that can be achieved by a saturated, stable moving queue of vehicles passing through the signal". The follow-up headway also requires the saturated condition for successive entering vehicles. As a result not all headways within gaps are follow-up headways. Typically a headway threshold is set to represent the saturated condition. Only headways that are smaller than the threshold and within gaps are considered as follow-up headways.

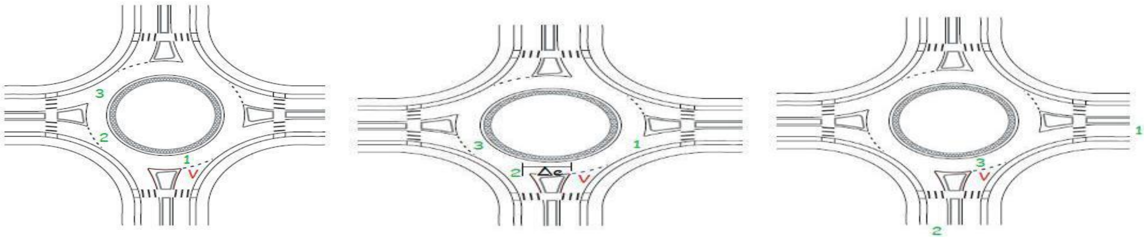
### *IV. Effects of exit Vehicles on Capacity*

For the estimation of the critical gap, gaps are measured by taking the difference in times when two successive circulating vehicles arrive the conflict point with the entering vehicle. However, if the following circulating vehicle exits before the conflict point, the gap cannot be measured that gap could have been perceived by the driver of the entering vehicle. Thus there may be discrepancy between the measured gap and the perceived gap.

To describe the method of considering the vehicles, the following case is considered:

Vehicle V is yielding to enter the roundabout, and Vehicles 1, 2 and 3 are the 1st, 2nd and 3rd vehicles respectively, which travel along the circulatory roadway heading towards the leg where Vehicle V is yielding. Vehicles 1 and 3 cross the leg where Vehicle V is yielding, but Vehicle 2 exits. Vehicle 1 crosses in front of Vehicle V at t1, Vehicle 2 exits at t2, and Vehicle 3 crosses in front of Vehicle V at t3. When the exiting vehicles are not considered, the only time-gap in front of Vehicle V would be measured as t3-t1 since Vehicle 2 did not reach the point of conflict. When the exiting vehicles are considered two gaps can be defined using the equivalent travel time ($\Delta e$). The time it would have taken for Vehicle 2 to travel from the exiting leg to the point of conflict if it had not exit. Thus the first gap is defined as $(t2 - t1) + \Delta e$, and the second gap is defined as $t3 - t2$. Zheng et al. (2011) found that the critical headway and the follow-up time were reduced when the exiting vehicles were considered.

However, these studies assumed a single value of the equivalent travel time for all vehicle types. Since the term $\Delta e$ is based on the free-flow speed of the circulating vehicles, it depends on the exiting vehicle.



Time instance $t_1$ Time instance $t_2$ Time instance $t_3$

**Figure3-1: Position of Circulating Vehicles at Various Time Instances**

## 3.2 Measure of Effectiveness (MOE)

***I. Volume to Capacity ratio***

Volume-to-capacity (V/C) ratios are the primary measure of effectiveness for evaluation against the operational performance. V/C ratios for roundabouts should be calculated based on the entry demand and capacity for the most critical approach (i.e. approach with the highest v/c ratio) for single-lane roundabouts and the most critical lane (i.e. individual lane with the highest v/c ratio) for multilane roundabouts.

II. ***Queuing***

Queuing estimates should be included with all near-term roundabout operational analyses (e.g., development applications, capital improvement projects). Depending on site-specific conditions and at the City discretion queuing analyses may be required for long-term operational analysis (e.g., transportation system plan, transportation planning rule (TPR)). Queues between roundabouts and adjacent intersections and/or driveways have the potential to impact the safety and efficiency of the roadway and intersection elements away from the intersection being analysed.

III. ***Delay***

Operational performance for roundabouts is measured against a V/C ratio to ensure a balanced comparison of alternative intersection forms delay estimates should be developed when comparing alternative intersection forms to the roundabout. As a general rule under the same traffic conditions, roundabouts typically will result in lower overall delay than traffic signals and all-way stop control but may result in higher overall delays than two-way stop control. Delay estimates can also be used to estimate vehicle emissions that result from various forms of intersection control.

### *IV. Level of Service*

Level of service should be defined by the delay values presented in Table 3-1. These values are consistent with HCM2010

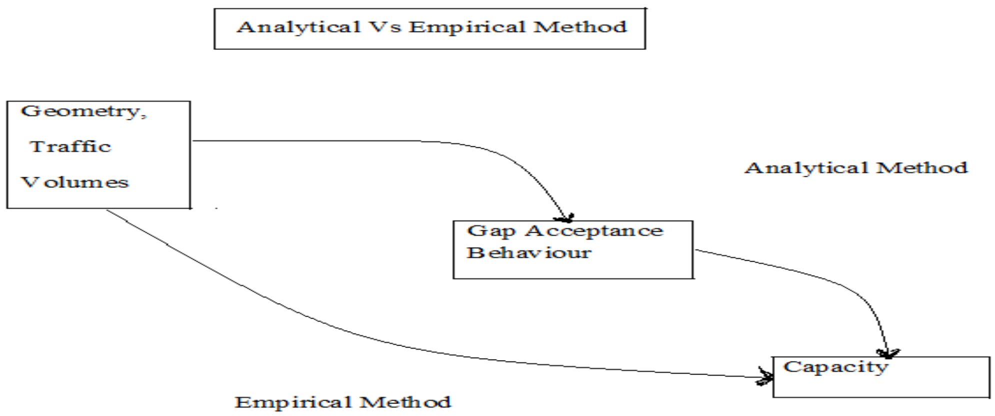
**Table3-1 HCM 2010 method for Level of Service definition based on Delay and V/C for vehicle with alternative methods for Roundabout**

| LOS | Control Delay (s/veh) | comments |
|---|---|---|
| A | d ≤10 | Usually no queue or conflicting traffic |
| B | 10< d ≤20 | Occasionally more than one in the queue |
| C | 20< d ≤35 | Not uncommon to have a standing queue of at least one vehicle |
| D | 35< d ≤50 | Delay is long enough to be an irritation to most drivers |
| E | 50< d ≤70 | Delay approaches most drivers tolerance level |
| F | d >70 | Approximate at capacity |

## 3.3 Methods

There are two different theories or methodologies to assess the capacity of the roundabouts. These theories are:

(i) Analytical methods
(ii) Empirical methods



## 3.3.1 Analytical methods

### 3.3.1.1 Gap Acceptance Capacity Models

Roundabout capacity has been estimated using various capacity models developed based on the gap acceptance theory. The gap acceptance method estimates the capacity based on the distribution of headways within the circulating flow, the critical-headway and the follow-up time.

***Headway Distribution***

The gap acceptance models assume that the headways (i.e. the time between consecutive vehicles passing the conflict point) of the circulating flow follows a certain distribution. The distribution follows an M1 (negative exponential), M2 (shifted negative exponential), or M3 (bunched exponential) (Cowan, 1997). The distributions are expressed as follows:

$$F(t) = 1 - e^{-\lambda t} \qquad \text{for } t \geq 0 \quad \text{(M1)}$$

$$F(t) = 1 - e^{-(\lambda t - \Delta)} \qquad \text{for } t \geq 0 \quad \text{(M2)}$$

$$F(t) = 1 - \alpha . e^{-(\lambda t - \Delta)} \qquad \text{for } t \geq 0 \quad \text{(M3)}$$

Where

F(t) = the cumulative probability that the headway is less than or equal to t

Δ = the minimum headway between the circulating vehicles (sec)

ƛ = the decay constant (sec)

α =the proportion of free vehicles

The decay constant λ is calculated using the following expression (Cowan, 1997)

$$\lambda = \frac{q_c . \alpha}{1 - q_c . \Delta}$$

Where

$q_c$=The circulating flow (pcu/h)

All distributions were developed based on the assumption that the arrival of vehicles follows a Poisson distribution.

The M1 distribution is the simplest form but does not headway. The M2 distribution is the M1 distribution with headways shifted by minimum non-zero headway. The M3 distribution has additional assumption of bunching‖ of vehicles within the circulating flow in congested conditions. Trout beck (1994) suggested the proportion of free vehicles at a roundabout is dependent on the circulating flow as follows:

$$\alpha=0.75(1-q_c)$$

Alternatively, Akçelik (2003) suggested that α can be estimated using the following equation

$$\alpha = \max\left[\frac{1-q_c}{1-(1-k_d)\Delta q_c}, 0.001\right]$$

Where

$k_d$ is a constant ( 2.2 for roundabouts).

The above equations assume that the proportion of free vehicles decreases as the circulating flow increases due to shorter headways.

***Critical Headway***

Critical headways are estimated using the distributions of gap acceptance and rejection data. Three methods are commonly used for estimating the critical headway:

1) The graphical method

2) The maximum likelihood method

3) The probability equilibrium method

The graphical method determines the critical headway by using cumulative distributions of individual entry vehicles accepted and rejected gaps. A gap is considered accepted if the driver of the entering vehicle perceives that the gap is sufficiently long enough for them to enter the roundabout .Otherwise the gap is rejected. The critical headway is then determined at the point of intersection between the two cumulative distributions curves of the accepted gaps and rejected gaps plotted on the same graph.

The maximum likelihood method (Troutbeck, 1989) assumes that the probability distribution function (PDF) of the critical headway ($F_{tc}$(t)) follows a lognormal distribution. The parameters of this PDF are obtained by maximizing the following likelihood function:

$$L = \prod_{i=1}^{n} [F_a(t) - F_r(t)]$$

Where

$F_a(t)$ = Accepted gap t

$F_r(t)$= maximum rejected gap $t_r$

However, this method only accounts for the maximum rejected gap, not all rejected gaps. Also it requires iterative calculation to maximize the above likelihood function.

To overcome these limitations the probability equilibrium method assumes that the probability distribution function (PDF) of the critical headway is described as follows (Wu, 2006).

$$F_{tc}(t) = \frac{F_a(t)}{F_a(t) + 1 - F_r(t)}$$

Where

$F_{tc}(t)$ = The PDF of the critical headway

If a time gap t is sorted in an ascending order, j = 1, 2…… N, the critical headway is calculated using the following expression.

$$t_c = \sum_{j}^{N} [P_{tc}(t_j).(t_j + t_{j-1})/2]$$

Where

$P_{tc}(t_j)$= the frequencies of the estimated critical headways between j and j-1. This method does not assume the distribution of gaps and accounts for all relevant rejected gaps, not only the maximum rejected gaps unlike the maximum likelihood method.

The critical headway is negatively correlated with higher circulating flow and higher speed of the circulating flow (Xu and Tian, 2008). Also the critical headway is affected by the waiting time of entrance vehicles (Polus et al., 2003). As waiting time increases, drivers will become more aggressive and will accept shorter gaps. Consequently this will reduce the critical headway. This could lead to forced entry manoeuvre, also known as gap forcing. When vehicles accept gaps which are shorter than the gap required to enter, the speed of the circulating flow will decrease.

### 3.3.1.2 Tanner Capacity Model

The headway distribution functions can be used in conjunction with gap-acceptance parameters to derive the capacity estimation models. These models are macroscopic analytical models which express the capacity in an exponential function of the circulating flow. The exponential function is reasonable because the rate of reduction in capacity generally decreases as the circulating flow increases and capacity never reaches zero. For example, the capacity model adapted in the Highway Capacity Manual (HCM) 2000 (TRB, 2000) assumes that headways follow an M1 distribution and is described as follows:

$$c_e = \frac{3600. q_c. e^{-q_c t_c}}{1 - e^{-q_c t_f}}$$

Where

$c_e$=the entry capacity (pcu/h)

$t_c$= the critical headway (sec)

$t_f$= the follow-up time (sec)

This capacity model was revised in the HCM 2010 (TRB, 2010) as follows

$$c_e = \frac{3600}{t_f}. e^{-(\frac{t_c - 0.5t_c}{3600})q_c}$$

The above capacity model is an exponential regression model developed based on a gap acceptance theory (Akçelik, 2011) .the HCM 2000 the critical headways were assumed to be different for different roundabout geometry. Geometry is classified in terms of the numbers of circulating lanes and entry lanes. In this model, shorter critical headways were used for a multi-lane roundabout than a one-lane roundabout.

The follow-up time for all roundabout geometry is 3.19 s. The capacity models were also derived using the M2 distribution and an M3 distribution as shown in Eq. respectively (Tanner, 1967; Troutbeck, 1986)

$$c_e = \frac{3600.q_c(1-\Delta q_c).e^{-q_c(t_c-\Delta)}}{1-e^{-q_c t_f}}$$

$$c_e = \frac{3600.q_c.\alpha.e^{-\lambda(t_c-\Delta)}}{1-e^{-\lambda t_f}}$$

The first step of the formulation is including the effect of vehicles on the capacity of entry stream at roundabouts by using a heavy vehicle equivalent for gap acceptance. This parameter represents the passenger car equivalents (PCE) of a vehicle show in the table 3-2

**Table 3-2Conversion to Passenger Car Equivalents**

| **Vehicle Type** | **Passenger Car Equivalent(PCE)** |
|---|---|
| Motor Cycle | 0.5 |
| Bicycle | o.5 for roundabout |
| Private car | 1 |
| Bus , tractor, truck | 3.5 |

***Delay***

The below equation provides a delay estimation model to be used in determining delay for each approach or critical lane. This model is based on the HCM 2010.This delay model and is consistent with recommendations from NCHRP Report 572. The delay estimates resulting from this model should be used to determine LOS according to the thresholds identified in Table 3-1.

$$d = \frac{3600}{c} + 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{\frac{3600}{c}x}{450T}}\right] + 5$$

Where

d =Average control delay, sec/veh

$x$ =Volume to capacity ratio of the subject lane

c =capacity of the subject lane, veh/h

T=Time period [T=1 for hour analysis, T=0.25 for 15 minutes analysis

***Queuing***

Queue lengths should be estimated using below equation for each single-lane approach and for the critical lane on each multilane approach. As shown the below equation will result in the 95th-percentile queue to occur during the peak period

$$Q_{95} = 900T\left[(x-1)+\sqrt{(1-x)^2+\frac{\frac{3600}{c}x}{150T}}\right](\frac{c}{3600})$$

Where

$Q_{95}$ = Queue length, veh

$x$ =Volume to capacity ratio of the subject lane

c =capacity of the subject lane, veh/h

T=Time period [T=1 for hour analysis, T=0.25 for 15 minutes analysis

## 3.3.2 Empirical Capacity Models

Empirical capacity models are the models developed using the data collected from the existing roundabouts. These models do not require gap-acceptance behaviour parameters. Instead, they directly describe the entry capacity as a function of the circulating flow. Some models include factors associated with the geometry of the roundabout.

Basic roundabout geometric features are shown in Figure. The main factors effecting capacity are the approach width, entry width and entry angle**.** In general wider entry width and approach width increase the entry capacity. The entry angle is related to the curvature of the approaching roadway, and a more direct path towards the circulating flow will increase the entry capacity. An inscribed circle diameter of 50m or less will have little effect on capacity. Wider circulating road width will increase the capacity of the circulating flow.

### 3.3.2.1 UK Capacity Model

The UK roundabout capacity formula is based on Kimber's study in 1980. The first approach is a linear approximation used to determine the entry capacity of a roundabout.

$$c_e = F - f_c . q_c$$

Where

*F* is a factor associated with the entry width, entry angle and width of the circulating flow and $f_c$ is a constant that depends on the geometry of the circle (in particular, inscribed circle diameter).

### 3.3.2.2 Germany's Capacity Model

In Germany they use an approach similar to that of the UK. German researchers investigated both regression and gap theory and decided to utilize the UK regression analysis. UK linear approximation is an exponential regression line. It was used to describe the entry/circulating flow relationship between the entry capacity and the circulating flow based on the data collected from 10 roundabouts.

$$c_e = Ae^{\frac{-Bq_c}{10000}}$$

Where

*A* and *B* are the parameters associated with geometric factors including the number of circulating lanes, and the number of entrance lanes. The model calibrated for single entry lane, single circulating lane roundabouts in the U.S. is as follows (NCHRP, 2007):

$$c_e = 1130e^{-0.001q_c}$$

Recently, continuing research from the federal government in Germany shows that the linear function instead of an exponential function has a better agreement of the variance of data. The new capacity formula is:

$$c_e = C + Dq_c$$

Where

$c_e$= entering capacity (vph)

$q_c$ =circulating flow (vph)

C and D are parameters show in table

**Table3-3 Parameters for Linear Regression**

| **Number of Lanes** | | **Defined Parameters** | |
|---|---|---|---|
| Entry | Circulating Roadway | C | D |
| 1 | 1 | 1218 | -0.74 |
| 1 | 2-3 | 1250 | -0.53 |
| 2 | 2 | 1380 | -0.50 |
| 2 | 3 | 1409 | -0.42 |

# Chapter4

# Study area and data collection

## 4.1 Study area

The Rourkela is a steel city. This city contains more population and more traffic problems so we can reduce traffic flow with increase capacity of roundabouts. The essential geometric and peak hour traffic data are collected at roundabouts. That roundabouts are chosen based on the principle of possible representative of the target population of roundabouts regarding size and numbers. Rourkela has many roundabouts and the chosen roundabouts have three legs and four legs in order to fully represent the size of the roundabouts. Actually, most of these roundabouts were built before 15 years ago when rotary and traffic circles are popular but now the drivers have to operate in accordance to modern roundabout traffic rules. Since tanner model does not depend on geometric elements, but they are more dependent on traffic rules. So that collecting traffic data and observing some geometric features possible to carry out the capacity analysis. The chosen roundabout names are as following table 4-1

**Table 4-1 Location of Studied Roundabouts and Dates of Video Footage**

| Roundabouts | Date of video taking | Time of day |
|---|---|---|
| Sector-2 Chowk | 27/11/2013 | 9.15am to 10.15 am |
| Sail Chowk | 28/11/2013 | 5.00pm to 6.00pm |
| Ambagan Chowk | 12/12/2013 | 9.00am to 10.00am |
| Plant Side Chowk | 13/12/2013 | 9.00am to 10.00 am |
| Traffic Gate C howk | 14/12/2013 | 4.30pm to 5.30 pm |

## 4.2 Data Collection

Gap acceptance/rejection, follow-up time and free-flow speed are collect from the video for the roundabouts. Any unusual driver behaviour such as gap-forcing behaviour, violation of the right-of-way, and unnecessarily tentative drivers was noted. All the data is collect manually. The traffic data collected should indicate the existing peak hour traffic conditions. Data are collected with the aid of a video camera to record the entry and exit of vehicles at two roundabouts in Rourkela. The video enabled information on volume, delay and speed and gap acceptance to be determined. The use of a video camera is noteworthy because it permits the use of the minimum number of personnel and the tapes can be reviewed several times to obtain the most accurate information. The video is used to determine the rejected gaps or lags of drivers approaching the roundabout and eventually the accepted gaps or lags that the drivers used to merge into the roundabout plus the follow-up times in instances where there is a queue.

The vehicles summarized as shown in table 4-2 on approach leg and in table 4-3on intersection. The data is collected for one hour or 60 minutes duration.

**Table 4-2 Summarized vehicle volume on each leg at peak hour**

| **Round about** | **Leg No.** | **Heavy Vehicles** | **Light Vehicles** | | | **Total Number of Vehicles** | **Total Traffic (PCU)** |
|---|---|---|---|---|---|---|---|
| | | | **Cars & autos** | **Motor cycles &bicycles** | **Total** | | |
| SECTOR-2 CHOWK | E | 23 | 125 | 377 | 502 | 525 | 394 |
| | W | 16 | 82 | 369 | 451 | 467 | 323 |
| | N | 35 | 284 | 653 | 937 | 972 | 733 |
| | S | 29 | 337 | 557 | 894 | 923 | 717 |
| SAIL CHOWK | E | 18 | 91 | 196 | 287 | 305 | 252 |
| | W | 4 | 47 | 135 | 182 | 186 | 129 |
| | N | 159 | 517 | 2219 | 2736 | 2895 | 2183 |
| | S | 112 | 398 | 1076 | 1474 | 1586 | 1328 |
| AMBAGAN | E | 51 | 249 | 443 | 692 | 743 | 649 |
| | W | 9 | 178 | 259 | 437 | 446 | 339 |
| | N | 49 | 292 | 934 | 1226 | 1275 | 931 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| CHOWK | S | 44 | 268 | 686 | 954 | 998 | 765 |
| PLANT SIDE CHOWK | E | 5 | 102 | 153 | 255 | 260 | 196 |
| | W | 46 | 161 | 272 | 433 | 479 | 458 |
| | N | 96 | 445 | 892 | 1337 | 1433 | 1227 |
| | S | 125 | 528 | 933 | 1461 | 1586 | 1432 |
| TRAFFIC GATE CHOWK | E | 22 | 104 | 153 | 257 | 279 | 258 |
| | W | 6 | 98 | 189 | 287 | 293 | 213 |
| | N | 45 | 573 | 1533 | 2106 | 2151 | 1497 |
| | S | 142 | 451 | 993 | 1444 | 1586 | 1445 |

The movement of traffic on the approaches or legs and the traffic volume in term of passenger car unit and these data necessary for the analysis. As explained in the table 3-2 the passenger car equivalent factors are used to convert the number of vehicles to passenger car equivalent.

**Table 4-3 Summarized vehicles volume on intersections at peak hour**

| Roundabout | Heavy Vehicles | Light Vehicles | | | Total Number of Vehicles | Total Traffic (PCU) | Percentage of Heavy Vehicles |
|---|---|---|---|---|---|---|---|
| | | Cars & autos | Motor & Bi cycles | Total | | | |
| SECTOR-2 CHOWK | 103 | 828 | 1956 | 2784 | 2887 | 2167 | 4 |
| SAIL CHOWK | 293 | 1053 | 3626 | 4679 | 4972 | 3892 | 6 |
| AMBAGAN CHOWK | 153 | 987 | 2322 | 3309 | 3462 | 2684 | 4 |
| PLANT SIDE CHOWK | 272 | 1236 | 2250 | 3486 | 3758 | 3313 | 7 |
| TRAFFIC GATE CHOWK | 215 | 1226 | 2868 | 4094 | 4309 | 3413 | 5 |

Figure 4-1 clearly shows the maximum and the minimum numbers of vehicle traffic at junction of roundabouts. The reason for this can mostly be attributed to land use. The maximum number of vehicles occurs at Sail chowk and minimum number of vehicles occurs at Sector-2 chowk

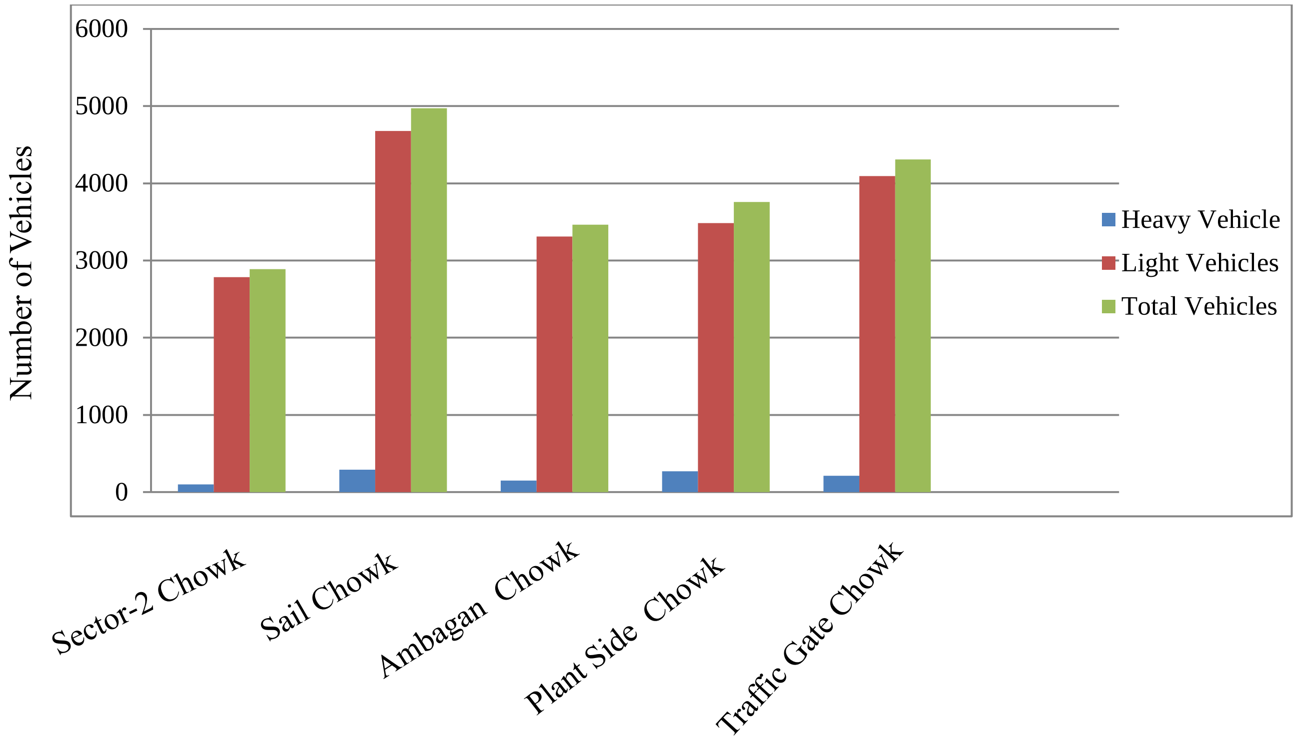



**Figure 4-1Maximum Peak Hour Vehicles Volume Distribution on Intersections**

From table 4-4 it is observed that there is unbalanced traffic flow at legs or approaches of roundabouts. However, it is not recommended to build roundabouts as traffic control devices when there is unbalanced traffic on the legs

**Table 4-4 Summarized Entry Traffic Flow on Approach Legs of Roundabout**

| **Roundabout** | **Leg No.** | **Entry Traffic flow on legs(PCU)** | **Percentage of Traffic Share** |
|---|---|---|---|
| SECTOR-2 CHOWK | E | 394 | 18 |
| | W | 323 | 15 |
| | N | 733 | 34 |
| | S | 717 | 33 |
| SAIL CHOWK | E | 252 | 6 |
| | W | 129 | 3 |

| | | | |
|---|---|---|---|
| | N | 2183 | 56 |
| | S | 1328 | 34 |
| AMBAGAN CHOWK | E | 649 | 24 |
| | W | 339 | 13 |
| | N | 931 | 35 |
| | S | 765 | 29 |
| PLANT SIDE CHOWK | E | 196 | 6 |
| | W | 458 | 14 |
| | N | 1227 | 37 |
| | S | 1432 | 43 |
| TRAFFIC GATE CHOWK | E | 258 | 8 |
| | W | 213 | 6 |
| | N | 1497 | 44 |
| | S | 1445 | 42 |

# Chapter5

# Data analysis and Results

## 5.1 General

Taking in to consideration all the above summarized data, we can proceed to the capacity analysis using tanner formula based on HCM 2010 and model proposed in NCHRP Report 572. However, some extra data is required to represent driver behaviour.

Gap-acceptance parameters, critical gap and follow up headway were measured during the traffic flow count. So better results are obtained and it is good to do it simultaneously with the traffic count. When it is measured with the traffic count on different legs at the same roundabout, different results of critical and follow up headway can be observed. Critical and follow up headway collected from roundabouts which is attached in the APPENDIX-A and also show in table 5-1

**Table 5-1 Critical gap and follow up time of each leg of round about**

| **Leg No.** / **Round about** | **N** | | **S** | | **E** | | **W** | |
|---|---|---|---|---|---|---|---|---|
| | **Critical gap(sec)** | **Follow up time (sec)** | **Critical gap(sec)** | **Follow up time (sec)** | **Critical gap(sec)** | **Follow up time (sec)** | **Critic al gap (sec)** | **Follow up time (sec)** |
| SECTOR-2 CHOWK | 4.01 | 2.5 | 3.83 | 2.6 | 3.64 | 2.93 | 3.4 | 3.6 |
| SAIL CHOWK | 3.13 | 2.41 | 3.10 | 2.7 | 3.75 | 2.68 | 4.87 | 3.24 |
| AMBAGAN CHOWK | 3.05 | 2.72 | 3.10 | 2.8 | 3.57 | 2.78 | 3.84 | 3.00 |
| PLANT SIDE CHOWK | 3 | 2.54 | 3 | 2.25 | 4 | 2.87 | 3.32 | 2.73 |
| TRAFFIC GATE CHOWK | 3.11 | 2.39 | 3.2 | 2.5 | 3.6 | 3.34 | 4.51 | 2.88 |

## 5.2Analysis of Results

Tanner capacity analysis produced the following results. The performance is measured with v/c ratio or degree of saturation and level of service based on HCM manual. To estimate capacity on the approach legs of roundabout using Tanner model, NCHRP Report 582 proposed model and German linear model show table 5-2.

**Table 5-2 Estimation of Capacity on the Approach Leg**

| **Round about** | **Leg No.** | **Traffic Count At Legs (V)** | **TANNER MDEL(Tanner)** | | **NCHRP REPORT MODEL** | | **GERMAN LINEAR MODEL** | |
|---|---|---|---|---|---|---|---|---|
| | | | **Capacity (C)** | **Degree Of Saturation (V/C)** | **Capacity(C)** | **Degree Of Saturation (V/C)** | **Capacity (C)** | **Degree Of Saturation(V/C)** |
| Sector-2 Chowk | E | 394 | 881 | 0.45 | 652 | 0.60 | 811 | 0.48 |
| | W | 323 | 796 | 0.40 | 605 | 0.53 | 756 | 0.43 |
| | N | 733 | 1063 | 0.69 | 858 | 0.85 | 1014 | 0.723 |
| | S | 713 | 1188 | 0.60 | 908 | 0.78 | 1056 | 0.675 |
| Sail Chowk | E | 252 | 710 | 0.35 | 436 | 0.58 | 513 | 0.49 |
| | W | 129 | 204 | 0.63 | 173 | 0.74 | 159 | 0.81 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | N | 2183 | 1426 | 1.53 | 1029 | 2.12 | 1148 | 1.90 |
| | S | 1328 | 1229 | 1.08 | 955 | 1.39 | 1094 | 1.21 |
| Ambagan Chowk | E | 649 | 713 | 0.91 | 422 | 1.54 | 489 | 1.33 |
| | W | 339 | 729 | 0.46 | 525 | 0.64 | 650 | 0.52 |
| | N | 931 | 1118 | 0.83 | 789 | 1.18 | 952 | 0.977 |
| | S | 765 | 1129 | 0.68 | 858 | 0.80 | 1014 | 0.75 |
| Plant Side Chowk | E | 196 | 613 | 0.32 | 390 | 0.50 | 431 | 0.45 |
| | W | 458 | 776 | 0.60 | 426 | 1.07 | 496 | 0.92 |
| | N | 1227 | 1236 | 0.99 | 850 | 1.44 | 1007 | 1.22 |
| | S | 1432 | 1481 | 0.97 | 974 | 1.47 | 1108 | 1.29 |
| Traffic Gate Chowk | E | 258 | 548 | 0.39 | 438 | 0.58 | 516 | 0.50 |
| | W | 213 | 442 | 0.48 | 334 | 0.64 | 335 | 0.64 |
| | N | 1497 | 1410 | 1.06 | 999 | 1.49 | 1127 | 1.30 |
| | S | 1445 | 1305 | 1.10 | 942 | 1.53 | 1083 | 1.33 |

There is a polynomial relationship between total entry flow at leg and Capacity at Leg. Figure 5-1 clearly shows the relationship between entry flow at leg and capacity at leg with a reasonable R-squared or coefficient of determination

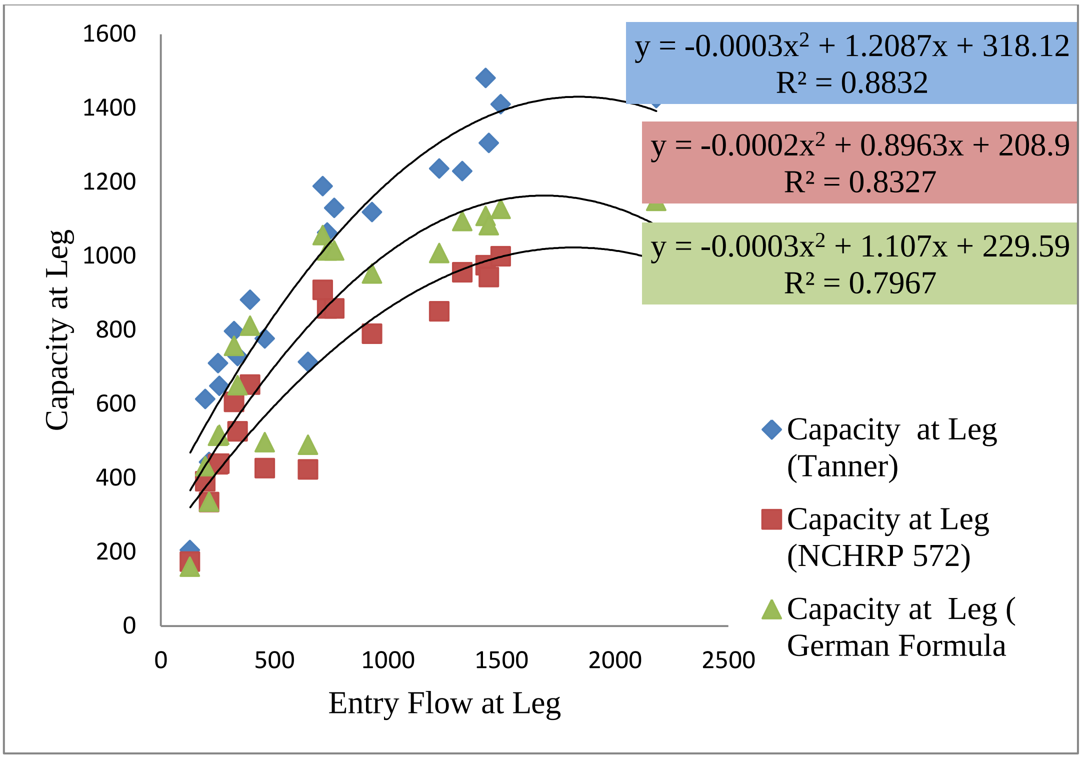



**Figure 5- 1 Entry Flow Vs Effective Capacity of each**

The curve fitting techniques result shows it is a not linear relationship but it is a polynomial one. The polynomial curve root mean square (coefficient of determination) does not have significant result which is 0.8. Even if the curve does not fit from the distribution of the values, we can observe that there is an increase the capacity when the entry flow increases to 1500 after it is decrease

The estimation of capacity for the intersection of roundabouts is summarized in Tables 5-3, 5-4 and 5-5. The performance is measured with v/c ratio or degree of saturation and level of service based on HCM manual

**Table 5-3 Capacity estimation of roundabouts following Tanner's Model**

| Round about | Total Vehicle Flow | Capacity | Degree of Saturation(V/C) |
|---|---|---|---|
| Sector-2 Chowk | 2167 | 3928 | 0.55 |
| Sail Chowk | 3892 | 3569 | 1.09 |
| Ambagan Chowk | 2684 | 3689 | 0.73 |
| Plant Side Chowk | 3313 | 4106 | 0.81 |
| Traffic Gate Chowk | 3413 | 3805 | 0.84 |

**Table 5-4 Estimation of Capacity using Model proposed in NCHRP REPORT 572**

| Round about | Total Vehicle Flow | Capacity | Degree of Saturation(V/C) |
|---|---|---|---|
| Sector-2 Chowk | 2167 | 3023 | 0.72 |
| Sail Chowk | 3892 | 2593 | 1.50 |
| Ambagan Chowk | 2684 | 2594 | 1.03 |
| Plant Side Chowk | 3313 | 2640 | 1.255 |
| Traffic Gate Chowk | 3413 | 2713 | 1.258 |

**Table 5-5 Capacity estimation of roundabouts following German linear Model**

| **Round about** | **Total Vehicle Flow** | **Capacity** | **Degree of Saturation(V/C)** |
|---|---|---|---|
| Sector-2 Chowk | 2167 | 3637 | 0.595 |
| Sail Chowk | 3892 | 2914 | 1.335 |
| Ambagan Chowk | 2684 | 3105 | 0.864 |
| Plant Side Chowk | 3313 | 3042 | 1.089 |
| Traffic Gate Chowk | 3413 | 3061 | 1.11 |

There is polynomial relationship between total entry flow at intersection and degree of saturation (v/c) at intersection of roundabouts. Figure 5-2 clearly shows the relationship entry flow and degree of saturation (v/c) with a reasonable R-squared or coefficient of determination

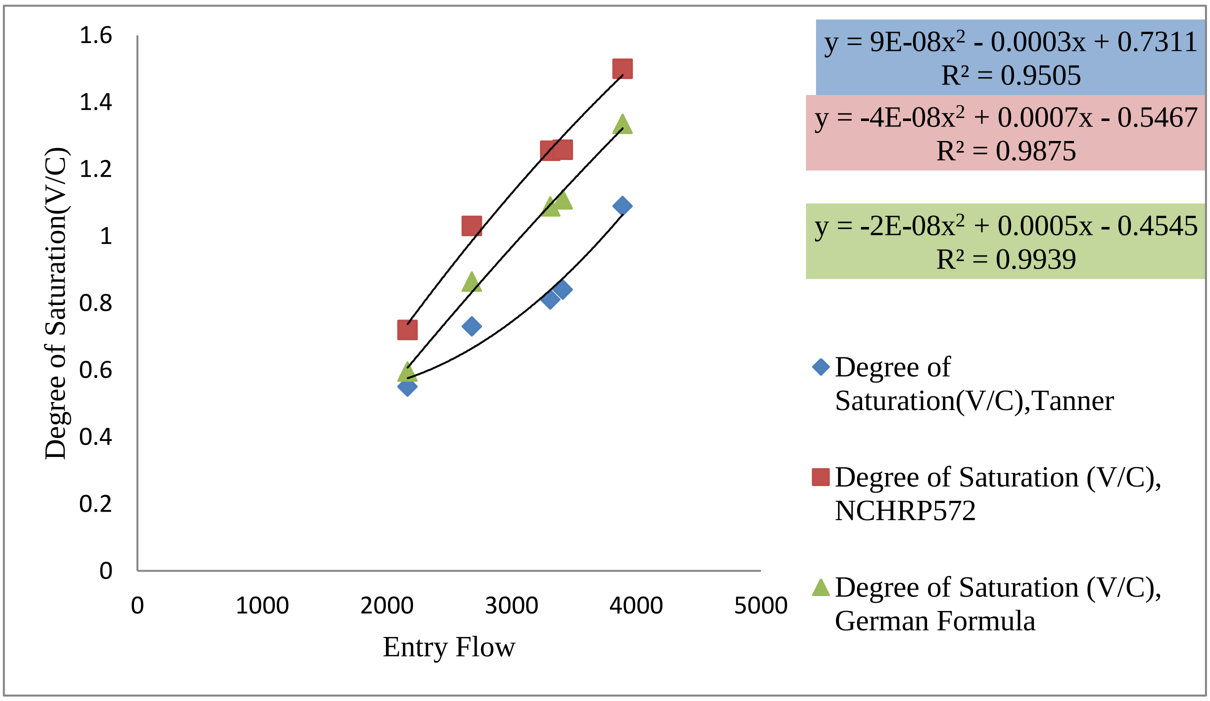



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**Figure 5-2 Entry Flow Vs Degree of Saturation (V/C)**

The curve fitting techniques result shows it is a not linear relationship but it is a polynomial one. The polynomial curve root mean square (coefficient of determination) does not have significant result which is 0.9. Even if the curve does not fit from the distribution of the values, we can observe that there is an increase the degree of saturation when the entry flow increases

From Table 5-6 it is seen that 1 roundabout has very low effective capacity compared to their entry flow. That is within the range of F LOS. Actually the intersection performance or capacity depends on the approaches or legs performance and always their v/c ratio is taken from the maximum v/c ratio of the approaches.

**Table 5-6 Summarised analysis results on the intersection based on tanner model**

| **Round about** | **Total vehicle flow (PCU)** | **Capacity** | **Delay** | **Queue length** | **Degree of Saturation** | **Level Of Service (LOS)** |
|---|---|---|---|---|---|---|
| Sector-2 Chowk | 2167 | 3928 | 7.03 | 3.65 | 0.55 | A |
| Sail Chowk | 3892 | 3569 | 179.42 | 191.13 | 1.09 | F |
| Ambagan Chowk | 2684 | 3689 | 8.60 | 7.983 | 0.73 | A |
| Plant Side Chowk | 3313 | 4106 | 9.57 | 12.4 | 0.81 | A |
| Traffic Gate Chowk | 3413 | 3805 | 13.85 | 23.34 | 0.84 | B |

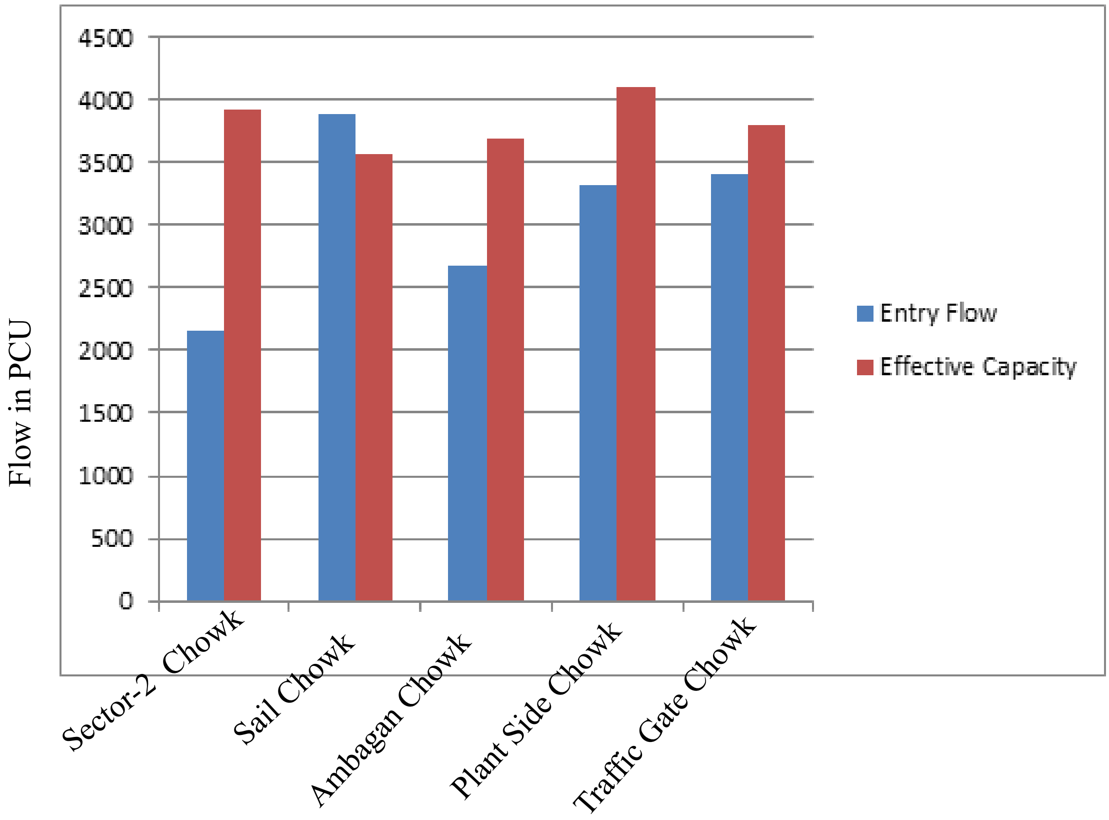


**Figure 5-3 Peak Flow Vs Effective Capacity**

Figure 5-3 shows peak flow or entry flow verses effective capacity and it clearly shows the maximum flow occurs at the Sail chowk ,maximum capacity occurs at Plant Side chowk and minimum flow occur at Sector-2 chowk ,minimum capacity occur at Sail chowk

From Table 5-7 it is seen that east leg of Ambagan chowk, north and south leg of Sail, Plant Side and Traffic Gate roundabouts have very low effective capacity than their entry flow. They are within the range of E to F LOS. Actually the capacity depends on the approaches or legs performance and always their v/c ratio is taken from the maximum v/c ratio of the approaches. Below Figure 5- also shown peak flow or entry flow verses effective capacity.

**Table 5-7 Summarised analysis results on the approach leg based on tanner model**

| Round about | Leg No. | Total Vehicle Flow(PCU) | Capacity | Delay | Queue length | Degree of Saturation | Level Of Service (LOS) |
|---|---|---|---|---|---|---|---|
| Sector-2 Chowk | E | 394 | 881 | 12.42 | 2.43 | 0.45 | B |
| | W | 323 | 796 | 12.50 | 1.983 | 0.40 | B |
| | N | 733 | 1063 | 15.8 | 6.43 | 0.69 | B |
| | S | 713 | 1188 | 12.55 | 4.42 | 0.60 | B |
| Sail Chowk | E | 252 | 710 | 12.79 | 1.604 | 0.35 | B |
| | W | 129 | 204 | 51.45 | 4.56 | 0.63 | E |
| | N | 2183 | 1426 | 968.75 | 386.4 | 1.53 | F |
| | S | 1328 | 1229 | 184.23 | 75.52 | 1.08 | F |
| Ambagan Chowk | E | 649 | 713 | 50.83 | 19.04 | 0.91 | E |
| | W | 339 | 729 | 14.125 | 2.523 | 0.46 | B |
| | N | 931 | 1118 | 23.21 | 12.896 | 0.83 | C |
| | S | 765 | 1129 | 14.88 | 6.2 | 0.68 | B |
| Plant Side Chowk | E | 196 | 613 | 13.68 | 1.402 | 0.32 | B |
| | W | 458 | 776 | 16.532 | 4.376 | 0.60 | B |
| | N | 1227 | 1236 | 71.52 | 39.86 | 0.99 | F |
| | S | 1432 | 1481 | 50.95 | 36.62 | 0.97 | E |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Traffic Gate Chowk | E | 258 | 648 | 14.1 | 1.90 | 0.39 | B |
| | W | 213 | 442 | 20.6 | 2.705 | 0.48 | C |
| | N | 1497 | 1410 | 149.8 | 73.00 | 1.06 | F |
| | S | 1445 | 1305 | 214.2 | 89.35 | 1.10 | F |

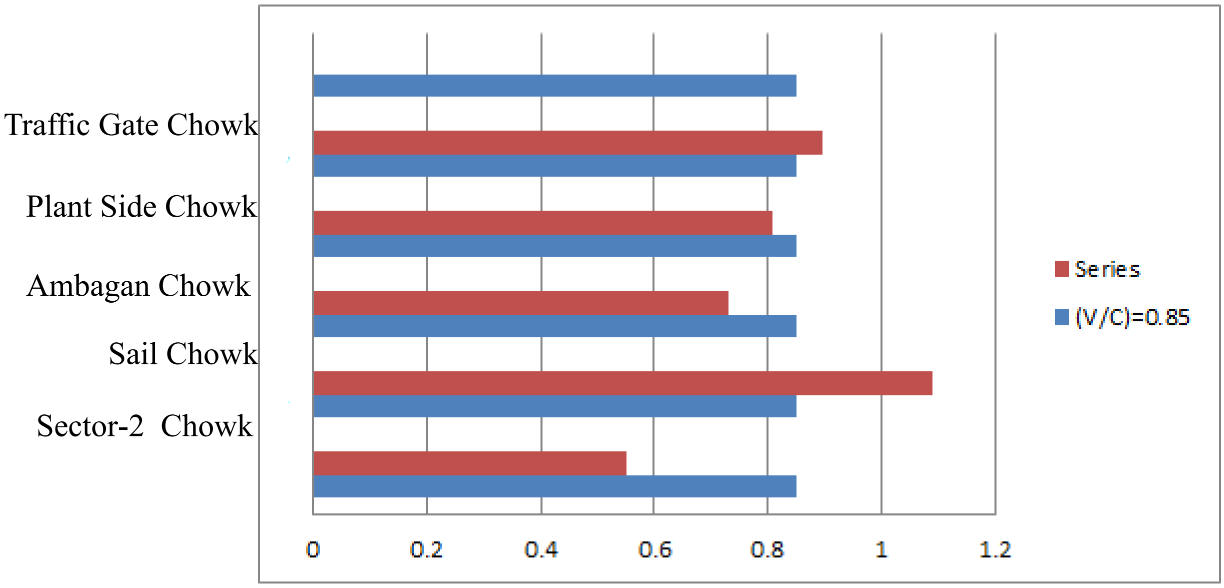


**Figure 5-4 Degree of Saturation at Intersection**

To give a clear picture of the result Figure 5-4 also presents degree of saturation with 0.85 being the recommended limit by HCM.

Sail chowk has higher entry flow at their intersection more than 3500 and their v/c ratio is very high and more than 1. From this we can observed that their higher traffic flow may lead to higher (v/c) ratio however it is so ahead of schedule there is no option choose without observing other parameters and legs capacity analyses results. For all intersection, lane-by-lane capacity has been done and capacity at legs, degree of saturation and opposing circulatory flow has been summarized as shown in Table 5-8

**Table 5-8 Summarized capacity analysis on the approach or legs based on tanner model**

| Round about | Leg No. | Traffic count at legs (PCU) | Opposing circulatory flow (pcu/hr) | Capacity at Leg | Degree of Saturation (V/C) | (V/C)>0.85 |
|---|---|---|---|---|---|---|
| SECTOR-2 CHOWK | E | 394 | 550 | 881 | 0.447 | -0.403 |
| | W | 323 | 624 | 796 | 0.406 | -0.444 |
| | N | 733 | 275 | 1063 | 0.69 | -0.16 |
| | S | 717 | 218 | 1187 | 0.604 | -0.246 |
| SAIL CHOWK | E | 252 | 952 | 710 | 0.355 | -0.495 |
| | W | 129 | 1876 | 204 | 0.63 | -0.22 |
| | N | 2183 | 94 | 1426 | 1.53 | **0.68** |
| | S | 1328 | 168 | 1228 | 1.08 | **0.23** |
| AMBAGAN CHOWK | E | 649 | 984 | 713 | 0.91 | **0.06** |
| | W | 339 | 767 | 729 | 0.46 | -0.378 |
| | N | 931 | 359 | 1118 | 0.83 | -0.02 |
| | S | 765 | 275 | 1129 | 0.677 | -0.173 |
| PLANT SIDE CHOWK | E | 196 | 1064 | 613 | 0.32 | -0.53 |
| | W | 458 | 976 | 776 | 0.59 | -0.26 |
| | N | 1227 | 285 | 1236 | 0.99 | **0.14** |
| | S | 1432 | 148 | 1481 | 0.967 | 0.117 |
| TRAFFIC | E | 258 | 948 | 648 | 0.40 | -0.45 |
| | W | 213 | 1219 | 442 | 0.482 | -0.368 |

| | | | | | | |
|---|---|---|---|---|---|---|
| GATE CHOWK | N | 1497 | 123 | 1410 | 1.06 | **0.21** |
| | S | 1445 | 182 | 1304 | 1.11 | **0.26** |

By observing the v/c >0.85 column from above Table **5-8** which is based on HCM (Capacity Manual of Highway) manual , we can easily identify the which legs are in a critical condition shown in Table 5-9.

**Table 5-9 Legs with Critical Condition (V/C>0.85)**

| **Roundabout** | **No. of legs(V/C)>0.85** |
|---|---|
| Sail chowk | 2 (N&S) |
| Ambagan Chowk | 1 (E) |
| Plant Side Chowk | 2 (N&S) |
| Traffic Gate Chowk | 2 (N&S) |

A total of 7 legs are in critical condition.

Before we investigate the reason for their inadequacy it is better to see the assumption on the theory in respect of direct relationships of capacity at legs and opposing circulatory flow. Capacity at legs is influenced by the average entry lane width and number of entry lane. Since it was first developed considering opposing circulatory flows vs capacity at legs relationship as it was mentioned using curve fitting techniques. The developed relationship is indicated in figure 5-5 below.

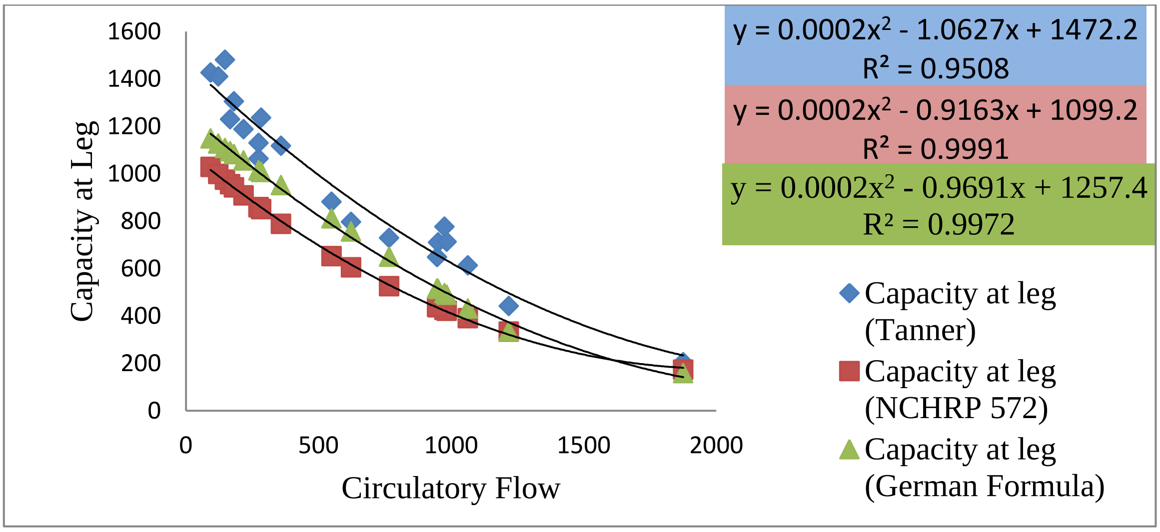


**Figure 5- 5 Opposing circulatory flow vs Capacity at leg**

The curve fitting techniques result shows it is a not linear relationship but it is a polynomial one. The polynomial curve root mean square (coefficient of determination) does not have significant result which is 0.9. Even if the curve does not fit from the distribution of the values we can observe that there is decrease the capacity when the circulating flow increases.

## 5.3 Service level of Roundabouts

It is possible to identify the problem of approach leg of roundabouts using Table 5-10 which shows v/c > 0.85, traffic volume of entry flow at legs and traffic volume of circulatory flow, entry and circulation lane numbers

**Table 5-10 Summary of the condition of the roundabouts**

| Roundabout | Leg No. | Problem |
|---|---|---|
| Sail chowk | N | There is high traffic even if the circulatory lane number is 2 which means that it cannot handle the traffic |
| | S | Circulation lane number not adequate |
| Ambagan chowk | E | Entry lane number not adequate |
| Plant Side Chowk | N | Entry lane number not adequate |
| | S | Entry lane number not adequate |
| Traffic Gtae Chowk | N | Circulation lane number not adequate |
| | S | Circulation lane number not adequate |

# Chapter 6

# Summary, conclusion and scope of future work

## 6.1 Summary

Based on the literature reviewed, different countries have their own methods of Capacity Analysis, which is sent by different researchers, but we can categorize them into totally Roundabout Geometry depends on the approach leg that is the Empirical Method. Gap acceptance approach that driver behaviour, type of vehicle, circulating and entering splits and conflicting circulating flow are included in Analytical Method.

## 6.2 Conclusion

Rourkela roundabouts capacity analysis results indicate the most of the legs of roundabouts are in serious problems or over saturation. Based on observed actual field conditions it is common to see that at peak hours, the traffic police need to regulate the traffic at these roundabouts since traffic control devices cannot function or regulate the traffic. As the study uncovered the real issues are identified with deficiency of number of entry lanes, number of circulatory lanes, high traffic flow and unbalanced traffic on the approaches o roundabout. Besides most of the roundabouts were built more than 15 years ago with obscure service limits.

All the input parameters of empirical method for capacity analysis do not exist at Rourkela Roundabouts. Thus only analytical method was carryout the capacity analysis with parameter using Tanner Formula based on HCM 2010.

High traffic entry flows at Sail Chowk roundabout was found to be more than 3500. This traffic is very high to be accommodated by the roundabout. In addition there is also high traffic flow (2183) at north leg of Sail Chowk that show high percentage of traffic volume share (56%), which is not recommended for roundabouts. Maximum capacity occurs at Plant Side chowk and minimum flow occurs at Sector-2 chowk, minimum capacity occurs at Sail chowk. East leg of Ambagan chowk, north and south leg of Sail, Plant Side and Traffic Gate roundabouts have very low effective capacity than their entry flow. They are within the range of E to F LOS. So these legs are in critical condition. The entry lanes of east leg of Ambagan chowk, north and south legs of Plant Side chowk are not adequate. The circulatory lane of south leg of Sail chowk, north and south legs of Traffic Gate chowk are not adequate.

## 6.3 Scope of Future work

After study or peak hour data collection for the roundabouts that have high and unbalanced traffic flow, their replacement with other junction type is suggested. The roundabouts which are located at the ring road are not providing the expected service levels. Since this device connects high-speed primary road and access road. Therefore replacement of these roundabouts by other intersection type is suggested after careful study.

Since the collected data for the analysis was limited especially regarding peak hour traffic the chart developed by this current research just understands on the subject of my exploration. In this respect, further study is prescribed with more data accumulation so as to refine the chart and for use of roundabout traffic services improvement. The refined chart can help the Rourkela City Road Authority when taking measures to improve roundabout intersections. They can additionally utilize it as a part of determining the traffic capacity identifying with area utilization. Therefore, if more traffic is generated because of new land use the charts can be used to easily forecast traffic in respect of each roundabout

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# APPENDIX-A

# (SUMMARISED TRAFFIC DATA)

**Critical gap and follow up time of roundabouts:**

**SECTOR-2 CHOWK**

| Leg No. / S.No. | N | | S | | E | | W | |
|---|---|---|---|---|---|---|---|---|
| | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time |
| 1 | 3.20 | 2.35 | 4.78 | 3.75 | 3.51 | 3.10 | 3.74 | 3.30 |
| 2 | 4.19 | 1.75 | 5.80 | 2.50 | 2.97 | 2.97 | 4.25 | 2.97 |
| 3 | 4.00 | 3.20 | 3.59 | 3.80 | 4.40 | 5.54 | 3.22 | 6.06 |
| 4 | 3.75 | 2.98 | 4.78 | 1.68 | 1.92 | 1.38 | 4.00 | 2.60 |
| 5 | 5.25 | 2.42 | 4.00 | 2.51 | 4.55 | 2.28 | 2.25 | 1.87 |
| 6 | 4.18 | 1.50 | 3.75 | 1.94 | 3.14 | 3.75 | 2.89 | |
| 7 | 3.20 | 3.37 | 2.71 | 2.45 | 4.26 | 2.60 | | |
| 8 | 5.41 | 2.94 | 2.97 | 2.98 | 4.78 | 1.85 | | |
| 9 | 3.23 | 2.71 | 2.45 | 1.89 | 3.22 | | | |
| 10 | 4.42 | 1.84 | 3.50 | 2.54 | | | | |
| 11 | 3.28 | | | | | | | |
| Total | 44.11 | 28.01 | 38.33 | 26.04 | 32.75 | 23.47 | 20.35 | 16.8 |
| Average | 4.01 | 2.5 | 3.83 | 2.60 | 3.64 | 2.93 | 3.39 | 3.36 |

**SAIL CHOWK**

| Leg No. / S.No. | N | | S | | E | | W | |
|---|---|---|---|---|---|---|---|---|
| | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time |
| 1 | 1.94 | 1.69 | 4 | 2.30 | 5.62 | 3.74 | 5.19 | 3.68 |
| 2 | 2.18 | 1.42 | 3.74 | 5.55 | 4.26 | 2.80 | 4.56 | 2.97 |
| 3 | 2.75 | 2.35 | 2.95 | 1.70 | 2.19 | 2.52 | | |
| 4 | 3.20 | 2.98 | 2.19 | 1.94 | 2.94 | 1.68 | | |
| 5 | 2.19 | 1.64 | 3.79 | 2.53 | | | | |
| 6 | 2.01 | 1.39 | 3.24 | 2.35 | | | | |
| 7 | 3.57 | 3.21 | 1.87 | 2.61 | | | | |
| 8 | 3.22 | 2.73 | 2.14 | 3.75 | | | | |
| 9 | 4.06 | 3.30 | 3.51 | 4.02 | | | | |
| 10 | 2.88 | 2.69 | 3.34 | 1.97 | | | | |
| 11 | 3.05 | 1.84 | 2.49 | 2.49 | | | | |
| 12 | 2.97 | 2.71 | 3.52 | 3.53 | | | | |
| 13 | 5.10 | 2.99 | | 1.62 | | | | |
| 14 | 4.71 | 2.08 | | 1.51 | | | | |
| 15 | | 3.18 | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Total | 43.83 | 36.20 | 36.78 | 37.87 | 15.01 | 10.74 | 9.75 | 6.47 |
| Average | 3.13 | 2.41 | 3.10 | 2.70 | 3.75 | 2.68 | 4.87 | 3.24 |

**AMBAGAN CHOWK**

| Leg No. / S.No. | N | | S | | E | | W | |
|---|---|---|---|---|---|---|---|---|
| | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time |
| 1 | 3.75 | 2.70 | 3.48 | 2.62 | 2.97 | 1.72 | 5.29 | 2.73 |
| 2 | 1.88 | 3.10 | 2.97 | 2.70 | 4.63 | 2.51 | 4.78 | 3.14 |
| 3 | 2.28 | 2.35 | 3.75 | 5.39 | 5.18 | 4.98 | 3.22 | 2.68 |
| 4 | 2.13 | 4.50 | 2.21 | 2.51 | 4.42 | 2.17 | 4.25 | 3.74 |
| 5 | 3.43 | 2.62 | 3.23 | 1.94 | 3.38 | 3.68 | 2.89 | 2.30 |
| 6 | 2.92 | 2.95 | 4.10 | 1.72 | 2.92 | 1.45 | 3.77 | 2.98 |
| 7 | 3.07 | 1.65 | 2.45 | 2.62 | 3.71 | 2.55 | 2.72 | 2.54 |
| 8 | 5.45 | 1.93 | 3.45 | 3.47 | 2.75 | 2.36 | | |
| 9 | 3.59 | 2.32 | 2.19 | 2.71 | 2.19 | 3.61 | | |
| 10 | 2.97 | 3.14 | 2.72 | 2.45 | | | | |
| 11 | 2.37 | 2.48 | 3.39 | | | | | |
| 12 | 2.71 | 2.87 | | | | | | |
| 13 | 3.15 | | | | | | | |
| Total | 39.70 | 32.61 | 33.94 | 28.13 | 32.15 | 25.03 | 26.92 | 20.11 |
| Average | 3.05 | 2.72 | 3.10 | 2.80 | 3.57 | 2.78 | 3.84 | 3.00 |

**PLANT SIDE CHOWK**

| Leg No. / S.No. | N | | S | | E | | W | |
|---|---|---|---|---|---|---|---|---|
| | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time |
| 1 | 2.94 | 2.35 | 2.19 | 1.68 | 2.68 | 3.00 | 2.98 | 2.62 |
| 2 | 2.71 | 1.68 | 3.72 | 1.49 | 4.26 | 3.97 | 4.23 | 2.98 |
| 3 | 2.20 | 2.73 | 2.68 | 2.35 | 3.78 | 2.32 | 5.25 | 3.30 |
| 4 | 3.31 | 2.30 | 3.17 | 2.51 | 3.24 | 2.65 | 2.78 | 2.82 |
| 5 | 3.74 | 2.97 | 4.25 | 1.87 | 4.75 | 2.42 | 1.94 | 2.71 |
| 6 | 2.22 | 3.13 | 2.96 | 1.94 | 5.19 | | 2.51 | 1.95 |
| 7 | 3.23 | 2.19 | 3.62 | 2.42 | | | 3.89 | |
| 8 | 2.97 | 3.45 | 2.75 | 2.38 | | | 2.97 | |
| 9 | 2.45 | 1.94 | 2.45 | 1.65 | | | | |
| 10 | 4.00 | 2.74 | 1.75 | 2.68 | | | | |
| 11 | 3.68 | | 4.08 | 2.91 | | | | |
| 12 | | | 2.52 | 3.15 | | | | |
| 13 | | | 2.98 | 2.28 | | | | |
| Total | 33.45 | 25.45 | 39.07 | 29.31 | 23.90 | 14.36 | 26.55 | 16.38 |
| Average | 3.00 | 2.54 | 3.00 | 2.25 | 4.00 | 2.87 | 3.32 | 2.73 |

**TRAFFIC GATE CHOWK**

| Leg No. / S.No. | N | | S | | E | | W | |
|---|---|---|---|---|---|---|---|---|
| | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time | Critical Gap | Follow Up Time |
| 1 | 2.70 | 1.93 | 1.78 | 2.60 | 3.76 | 2.51 | 4.25 | 3.46 |
| 2 | 2.20 | 1.87 | 2.82 | 3.22 | 3.48 | 3.35 | 6.06 | 2.86 |
| 3 | 3.25 | 2.39 | 2.56 | 2.99 | 2.85 | 5.40 | 4.78 | 2.15 |
| 4 | 4.18 | 2.46 | 5.52 | 1.98 | 2.39 | 2.70 | 3.42 | 3.05 |
| 5 | 3.75 | 2.52 | 2.97 | 2.73 | 5.29 | 2.62 | 4.02 | |
| 6 | 2.45 | 2.89 | 4.95 | 1.65 | 3.86 | 3.45 | | |
| 7 | 2.71 | 3.16 | 1.68 | 2.84 | | | | |
| 8 | 4.68 | 4.95 | 2.74 | 1.45 | | | | |
| 9 | 1.92 | 1.72 | 3.36 | 2.52 | | | | |
| 10 | 2.89 | 2.97 | 4.47 | 3.08 | | | | |
| 11 | 3.48 | 2.48 | 2.23 | | | | | |
| Total | 34.21 | 26.34 | 35.08 | 25.06 | 21.63 | 20.03 | 22.53 | 11.52 |
| Average | 3.11 | 2.39 | 3.2 | 2.5 | 3.60 | 3.34 | 4.51 | 2.88 |