CHAPTER 2
REVIEW OF LITERATURE

2.1 General

The research carried out earlier in the fields related to the present study is reviewed in this chapter. Though some of the areas of research cited are not directly related to the present work, the results of the works reviewed have a considerable bearing on the conceptual framework of the present study. Emphasis is placed on the research on saturation flow rate estimation, PCU factor development and back propagation based neural network modeling. The methodology suggested by Highway Capacity Manual (2000) is discussed elaborately to study the suitability of the application of this procedure to Indian urban traffic conditions and to suggest suitable modifications.

2.2 Webster’s Analysis

The pioneering study regarding the signalized intersections is reported by Webster (1958). The basic concepts proposed by Webster are still being followed and the signal design based on his theory is considered as a standard procedure in many countries. Webster has treated the saturation flow rate as an indicator of the capacity of the approach to a signalized intersection and based on extensive research, he has proposed an equation for the estimation of saturation flow rate. According to Webster, saturation flow rate in PCU/hour, S is given by $S = 525w$, where ‘w’ is the approach width available for the movement in m. In order to take into consideration the different types of vehicles Webster has also suggested a set of PCU factors to be adopted at a signalized intersection. His work deals extensively with the corrections and modifications to saturation flow rate under different phase patterns Webster observed that for given traffic and roadway factors, the average delay to a vehicle
depends on the cycle length. If the cycle is too short, the delay will be very high and if
the cycle is too long, then also the delay will be exceedingly high. Webster has
suggested that for given conditions, there will be a cycle time where the delays are
minimum and he called it as the optimum cycle time.

Saturation flow rate plays a major role in influencing the optimum cycle time.
Webster has also studied the vehicular delays at signalized intersections and he has
suggested an equation for the delay estimation based on simulation of traffic. The
equation is

\[ d = \frac{c(1-x)^2}{2(1-2x)} + \frac{x^2}{2q(1-x)} - 0.65 \left( \frac{c}{q^2} \right)^{1/3} X (2+5\lambda) \]

Where \( d \) = average delay to a vehicle on a particular arm in seconds;
\( \lambda \) = proportion of the cycle which is effectively green for the phase under
consideration, i.e., \( g/c \) ratio;
\( x \) = the degree of saturation, i.e., the ratio of flow to the maximum possible flow
under given settings and is given by \( q/\lambda s \);
\( q \) = The arrival rate on the approach, veh/sec;
\( s \) = saturation flow rate, veh/sec, and
\( c \) = cycle time in seconds.

Webster has also related delay to the approach volume and he has observed
that the average delay remains at a low value within the capacity of the approach and
once the flow nears the approach capacity the delay values increase very steeply.

2.3 Studies on Mixed Traffic Equivalency Factors

Nagaraja, George and John (1990) have conducted studies on lateral and linear
placements of vehicles under mixed traffic situation. Their research is based on the
observations of traffic in mid-blocks of Calicut city through videography. They have
proposed mixed traffic equivalents for different modes based on the influence area of a mode. They have reported that in mixed traffic situation the flow corresponding to capacity condition can have widely varying values depending upon the degree of freedom enjoyed by vehicles. It is also reported that the PCU values developed in the study are significantly different from the values proposed by earlier works because of high percentage of Auto-Rickshaws in the traffic stream. The lateral and linear spacings of vehicles in mixed traffic are studied by Anjaneyulu, Phani Kumar and Nagaraj (1998) using neural networks.

Justo and Tuladhar (1984) have proposed the concept of influence area of a vehicle that extends beyond the static dimension of a vehicle and based on the influence exerted by a particular mode the PCU factors are proposed for various modes. The probabilistic pavement occupancy concept is proposed by Agarwal, Jain and Khanna (1994) in their study on mixed traffic behavior at three legged intersections. The study observed that the effective traffic increases as the vehicles approach the intersection and based on the pavement occupancy of the vehicles at the intersection the researchers have proposed Passenger Car Equivalents for the mixed traffic conditions. Satish, Virendra Kumar and Sikdar (1990, 1993 and 1995) have proposed that the PCU factor for a mode in a mixed traffic environment does not depend only on the static dimensions and the operating characteristics of the vehicles and it is a dynamic factor. Based on their observations of traffic in the field they have suggested different sets of PCU factors that can help in the capacity estimation of the intersections.

Bhattacharya and Mandal (1980) investigated the traffic at the uncontrolled intersections of Calcutta city and Passenger Car Equivalents were proposed for different modes based on their observations. Tamizharasan et al (1993, 1995 and
1999) have used the saturated green time for the development of PCU factors and the
saturation flow rates based on these factors are proposed for the signalized
intersection approaches under heterogeneous traffic conditions. Tamizharasan
observed from his study that the saturation flow rate for a lane of 3.5 m works out to
be higher than 1800 PCU/hr, a value proposed by the HCM.

2.4 Studies on Performance Evaluation of Signalized Intersections

Kara and Raheel (2000) analyzed the impacts of different light duty trucks
(LDTs) on the capacity of signalized intersections. Regression analysis generated
estimates of headways associated with various categories of LDTs as well as
passenger cars and calculated passenger car equivalents. It was suggested that the
impacts of LDTs were to be given special consideration when analyzing the capacity
of signalized intersections. Bradon and Nagui (2002) summarized the results of an
empirical study of lane volume data and provided an evaluation of six lane selection
strategies used to estimate lane flows. The lane selection strategies were used as part
of the subgroup approach for estimating saturation flows. The evaluation indicated
that the selection strategy based on equal back of queue or cycle average queue
provides the best prediction of lane volumes and these results indicated three
international capacity guides using an equal flow ratio or degree of saturation strategy
for estimating lane flow need a large data collection effort to confirm the results.

Satish Chandra and Upendra Kumar (2003) have proposed a concept to
estimate the PCU factor for a mode in a mixed traffic environment utilizing area
concept. It was found that the PCU for a vehicle type increases linearly with the width
of carriageway. This was attributed to the greater freedom of movement on wider
roads and therefore a greater speed differential between a car and a vehicle type. The
capacity of a two lane road also increases with total width of the carriageway and the
relationship between two follows a second degree curve and this relationship was used to derive the adjustment factors for substandard lane widths. Ebrahim, Zeinab and Reza (2004) proposed a simulation model for vehicular dynamics using probabilistic cellular automata and evaluated the delay experienced by the traffic for specified time intervals and some traffic responsive signalization algorithms were proposed for optimum signalization of traffic lights based on the concept of cut-off queue length and cut-off density.

Huang and Jianping (2004) studied the cyclist behavior such as crossing speeds, crossing gap/lag acceptance and group riding behavior at a signal controlled intersection and statistical data analysis was conducted to determine the intersecting group behavior of cyclists useful for understanding the performance of mixed traffic at signalized intersections and to build microscopic simulation models. Li, Yue and Wong (2004) studied a gray system theory-based method for the quantitative evaluation and ranking of the operational safety performance of signalized intersections in urban areas under mixed traffic conditions. Five index parameters, the degree of saturation, the average stopped delay, queue length, the conflict ratio and separation ratio are proposed in the method and the results showed that the method can be used to comprehensive performance evaluation and ranking of signalized intersections with urban road network systems.

Panos and Kervin (2005) studied the potential effects of wet conditions on signalized intersection LOS. A methodology was developed for the derivation of probability of rainfall. The study revealed that signalized intersection operations become less efficient as saturation flow rate, effective green time and progression were affected by the wet weather. Rafael, Perez and Andrew (2005) conducted saturation flow studies for Indiana and the results were confirmed that the default
parameters such as heavy vehicle equivalency factor and the protected left-turn adjustment factor recommended by HCM were adequate for Indiana. Base saturation flow was found to vary strongly across locations though geometric and traffic conditions were nearly identical. Regression analysis of saturation flow rate identified that the size of the town has impact on the base saturation flow rate. The study provided a convincing argument to add a population adjustment factor to the HCM for saturation flow rates.

Sang and Benekohal (2005) compared control delays from CORSIM simulation to the HCM (2000) for oversaturated signalized intersections for all arrival types. The results showed that HCM delays were different to CORSIM delays for all arrival types for all degrees of saturation. Zong and Ning (2006) proposed a capacity estimation model for a signalized intersection capacity with a short right turn lane considering the probabilistic nature of traffic flow and the effect of queue blockage to the Short lane section. A simulation study revealed that saturation flow rate increases with the increase of the length of the right turn lane. The proposed Model was validated against CORSIM and capacity enhancement was achieved with a short right turn lane compared to the shared lane situation.

Chang and Sonny (2011) developed a model for the actual peak-hour factors as a function of volume to capacity ratio and the functional classification of roadways. The model was validated with HCM capacity manual and results showed that the recommended peak-hour factors resulted in higher average intersection delays compared to the HCM default value.

2.5 Saturation Flow Studies under Mixed Traffic Situation

Sarna and Malhotra (1967) have developed an equation for the saturation flow rate under mixed traffic conditions. The basic concept of the positive linear
relationship between the saturation flow rate and the approach width is reflected in their model also, but the coefficients and the constant changed. The equation proposed by them is \( S = 431.7W + 103.5 \), where \( S = \text{Saturation flow rate, PCU/hr} \); and \( W = \text{Approach width available for the movement under consideration, m} \).

The estimated saturation flows in this case are far lower than those proposed by the equation of Webster.

Bhattacharya and Mandal (1980) have proposed an equation for the estimation of saturation flow rate as a function of approach width. According to them, the Saturation flow rate is given by the formula

\[
S = 490W - 360
\]

Where \( S = \text{Saturation flow rate, PCU/hr} \), and \( W = \text{Approach width available for the movements under consideration, m.} \)

It is quite clear from the equation that the estimated saturation flow values by Bhattacharya’s equation are also very much less than those proposed by Webster.

Bikram Das (1984) proposed an approach for the determination of the saturation flow rates under mixed traffic conditions. He has developed saturation flow rate curves based on the composition of traffic. Satish Chandra and Sikdar (1993) have studied the capacity of a signalized intersection under mixed traffic conditions and according to their observations the saturation flow rate estimation for the Indian urban conditions needs a different approach that gives due emphasis for the dominating presence of smaller vehicles such as two wheelers. One of the significant observations from their study is that the increased approach width results in lesser discharge because of the increased freedom to maneuver enjoyed by the smaller vehicular modes. The dynamic PCU factors are proposed based on the saturation flow rate and the approach width. According to their work, the saturation flow rates for
straight and right directional movements at a signalized intersection can be estimated as:

\[ S = 1241 + 293 \, W \]

\[ S_r = 1895 + 250 \, W - 31735.7/R \]

Where \( S \) = Saturation flow rate for through movement, PCU/hr;

\( S_r \) = Saturation flow rate for the right turning movement, PCU/hr;

\( W \) = Approach width available for the movement under consideration, m; and

\( R \) = Radius of right turn, m

Ravinder (1997) has reported in his study that the direct application of Webster’s equation gives lower saturation flow rates than what are actually observed. He has also reported that the 2/3 wheeler composition in urban traffic is very high ranging from 60 to 90 percent. He has proposed different saturation flow rate equations for straight and right directions at a signalized intersection. According to the observation, the saturation flow rates can be estimated as

\[ S_{sz} = -185.527 \, W^2 + 4989.18 \, W - 13202.8 \]

\[ S_{r2} = -586.79 \, W^2 + 7272.88 \, W - 11382.4 \]

Where \( S_{sz} \) = Saturation flow rate for straight, vph;

\( S_{r2} \) = Saturation flow rate for right, vph

\( W \) = Width available for the movement under consideration, m;

The applicability of HCM procedure to the Indian urban traffic conditions is studied by Madhusudhana Rao (1997). He observed that the HCM procedure can be adopted for the mixed traffic dominated by the 2/3 wheelers, if the saturation flow rate can be computed with a modified equation. Sreenivasa Reddy (1998) has conducted studies on the saturated green times and he observed that the Hyderabad urban traffic is dominated by 2/3 wheeler group with an average proportion of 0.70.
Sankara Rao (1999) made a study on saturation flow rates at urban intersection making use of overall headway. He has suggested that the estimation of saturation flow rate under mixed traffic conditions has to consider the composition of traffic. Capacity and Level of service of signalized urban intersections is studied by

Sunil Anand (1999) and he has suggested that saturation flow rate in vph can be estimated using an equation $S=2000W$, where $S$ is the saturation flow rate in vph and $W$ is the width available for the movement in m. Hari Narayana Murthy (1999) has proposed an equation for the estimation of the saturation flow rate at signalized intersections where the slow moving vehicles composition is high. He has proposed that the Equivalent Passenger Car Units (EPCU) are dynamic in nature and the saturation flow rate can be computed as

$$S = 1619 + 463W$$

Bhanu Murthy (2000) has proposed an equation for saturation flow rate at signalized intersections. He has suggested that the estimation of saturation flow rate under mixed traffic conditions has to consider the proportion of highly maneuverable vehicle group. According to the observations the saturation flow rate can be computed as

$$S= -5554 + 12001P_{HMV} + 557 W$$

Where $S$ = Saturation flow rate, vph;

$P_{HMV}$ = Proportion of highly maneuverable vehicles; and

$W$ = Width available for the combined straight and right movement from an approach,

m. Yanming et al (2012) studied the effect of different states of bicycles crossing a signalized intersection. The saturation flow rates of right turn and left turn vehicles under the influence of bicycles are modeled considering different stages of bicycles and results could supplement the HCM method of intersection capacity analysis and
design of intersection signal timing design.

Prasanna Kumar et al (2012) have used an alternate method to estimate saturation flow rate for capacity analysis of signalized intersection under mixed traffic condition and made an attempt to adopt the alternate method to suit the field conditions.

2.6 Highway Capacity Manual Method of Capacity Analysis of Signalized Intersections

Highway capacity Manual (2000) treats the capacity analysis of the signalized intersections from a different perspective compared to the capacity computations of the highway facilities. The capacity of a highway facility is primarily related to the geometric features of the highway as well as to the composition of the traffic using the facility. Geometrics are fixed or non-varying component of a highway. Thus, allowing for some variation in the traffic composition over time, the capacity of a highway is generally a stable value which can be significantly improved by improving the geometrics. At the signalized intersection an additional element is introduced into the concept of capacity, namely the time allocation.

A traffic signal essentially allocates time among the conflicting traffic movements seeking use of the same physical space. The way in which time is allocated to different movements has a significant impact on the operation of the intersection and the capacity of the approaches as well as the intersection. HCM methodology addresses the capacity and level of service of the individual approaches of the intersection and the level of service for the intersection as a whole. Capacity is evaluated in terms of the ratio of the demand flow rate to capacity \((v/c \text{ ratio})\) while level of service is evaluated on the basis of average stopped delay for vehicle \((sec/veh)\).
The capacity of the intersection as a whole is not addressed, because the design and signalization of intersection focuses on the accommodation of major movements on the approaches comprising the intersection. Capacity is, therefore, only meaningful as applied to these major movements and approaches. Evidently as per the HCM the capacity of an intersection is highly dependent on the signalization present. Given the range of potential signal control schemes, this capacity is far more variable than for other types of facilities, where capacity is mainly dependent on the physical geometry of the roadway. In effect, signalization, which can be changed frequently and quickly, allows considerable latitude in the “management” of the physical capacity of the intersection space and geometry. Though HCM treats the concepts of capacity and level of service as central to the analysis of the intersections as they are for all other types of facilities, the two concepts are not as strongly correlated as they are for other facilities. In the case of highway facilities, same analysis yields both the capacity and the level of service; but these two are to be separately analyzed for a signalized intersection and they are not simply related to each other.

Capacity analysis of the intersections results in the computation of v/c ratios for individual movements and a composite v/c ratio for the sum of the critical movements or lane groups within the intersection. The v/c ratio is the actual or projected rate of flow on an approach or a designated lane group during the15-minute peak period divided by the capacity of the approach or the lane group. Level of service is based on the average stopped delay per vehicle for various movements within the intersection. While v/c ratio affects delay, there are other parameters that more strongly affect such as the quality of progression, length of green phases, cycle lengths and others. Thus, for any given v/c ratio, a range of delay values may result,
and vice/versa. For this reason, both the capacity and level of service of the intersection must be carefully examined.

2.6.1 Capacity of Signalized Intersection

Capacity at a signalized intersection is defined for each approach. Intersection approach capacity is the maximum flow rate (for the approach under consideration) which may pass through the intersection under prevailing roadway, traffic and signalization conditions. Capacity of an intersection approach depends on the saturation flow rate. Saturation flow rate is defined as the maximum rate of flow that can pass through the given intersection approach or lane group under prevailing roadway and traffic conditions, assuming that the approach or lane group had 100 percent of real time available as effective green time. The lane group can be defined as a set of single or more number of traffic streams that are released simultaneously during the same green phase. The saturation flow rate is given a symbol and is expressed in terms of vehicles per hour of effective green. The capacity of a given lane group or approach may be stated as

\[ C_i = S_i \times (g/c)_i \]

Where \( C_i \) = capacity of lane group or approach \( i \), in vph;

\( S_i \) = saturation flow rate of the approach or lane group \( i \), in vphg; and

\( (g/c)_i \) = green ratio for lane group or approach \( i \).

Another parameter used in the methodology of HCM is flow ratio. The flow ratio for a given approach or lane group is defined as the ratio of the actual flow rate for the approach or lane group, \( v \), to the saturation flow rate. The flow ratio is given the symbol, \( (v/s)_i \), for the approach or lane group \( i \).

The ratio of low rate to capacity, \( v/c \), is given the symbol X in intersection analysis. This new symbol is introduced to emphasize the strong relationship
between capacity and signalization conditions and also for consistency with the earlier works that refer to this variable as “degree of saturation”. Thus for a given lane group or approach $i$:

$$X_i = \left( \frac{v_i}{c_i} \right) = \frac{V}{C} \left( \frac{g_i}{C} \right) = \frac{v_i}{s_i} \frac{g_i}{C} = \left( \frac{v_i}{s_i} \right) \frac{g_i}{C}$$

Where $X_i = \left( \frac{v_i}{c_i} \right)$ is the $v/c$ ratio for the lane group or approach $i$:

- $v_i$ = actual flow rate for the lane group or approach $i$, in vph;
- $s_i$ = saturation flow rate for the lane group or approach $i$, in vphg;
- $g_i$ = effective green time for the lane group or the approach $i$, in seconds; and

$$C = \text{cycle time of the signal, in seconds.}$$

Values of $X_i$ range from 0.00 when the flow rate is zero to 1.00 when the flow rate equals capacity. The capacity of the entire intersection is not a significant concept as per HCM methodology and is not specifically defined. Rarely do all the movements at an intersection become saturated at the same time of day. It is the ability of the individual approaches to accommodate the movements with efficiency. Another capacity concept of utility in the analysis of signalized intersections is the critical $v/c$ ratio, $X_c$. This is a $v/c$ ratio for the entire intersection as whole, considering only the lane groups or approaches that have the highest flow ratio, $v/s$ for a given signal phase. The critical $v/c$ ratio for the intersection is defined in terms of critical lane groups or approaches and can be computed as

$$X_c = \frac{\sum_i (v/s)_{ci} X_i (C/(C-L))}{X_i}$$

where

- $X_c = \text{critical v/c ratio for the intersection; }$
- $\sum_i (v/s)_{ci} = \text{the summation of flow ratios for all critical lane groups or approaches, } i;$
- $C = \text{Cycle length in seconds; and}$
- $L = \text{total lost time per cycle, in seconds.}$
The equation is useful in evaluating the overall intersection with respect to the geometrics and the total cycle length provided. It gives the v/c ratio for all critical movements, assuming that the green time is appropriately or proportionally allocated. It is therefore, possible to have a v/c ratio less than 1.00, and still have individual movements oversaturated within the cycle time. A critical v/c ratio less than 1.00, however, does indicate that all movements in the intersection can be accommodated within the defined signal cycle and phase sequence by proportionately allocating the green time. In essence, the total available green time in the phase sequence is adequate to handle all the movements, if properly allocated.

2.6.2 Level of Service for Signalized Intersection

Level of Service (LOS) for signalized intersections is defined in terms of delay. Delay is a measure of driver discomfort, frustration, fuel consumption and lost travel time. Delay may be measured in the field, or estimated using the methods suggested by HCM. Delay is a complex measure, and depends on a number of variables, including the quality of progression, the cycle length, the green ratio and the v/c ratio for the lane group or approach under consideration. The various levels of service and corresponding delay ranges suggested by HCM are as follows.

<table>
<thead>
<tr>
<th>Level of service</th>
<th>Stopped delay Per vehicle (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>( \leq 5.0 )</td>
</tr>
<tr>
<td>B</td>
<td>5.1 to 15.0</td>
</tr>
<tr>
<td>C</td>
<td>15.1 to 25.0</td>
</tr>
<tr>
<td>D</td>
<td>25.1 to 40.0</td>
</tr>
<tr>
<td>E</td>
<td>40.1 to 60.0</td>
</tr>
<tr>
<td>F</td>
<td>&gt; 60.0</td>
</tr>
</tbody>
</table>
2.6.3 **Operational Analysis of a Signalized Intersection**

The operational analysis considers the full details of each of the four components: demand or service flow rates at the intersection, signalization parameters, geometric characteristics of the intersection and the delay or LOS that results from these characteristics. The analysis results in the determination of capacity and level of service for each approach or lane group and the level of service for the intersection as a whole. The methodology requires that the detailed information be given concerning the geometrics, traffic and the signalization at the selected intersection. Because of the complexity involved, the analysis is divided into five distinct modules as given below.

**Input Module**: This module focuses on the definition of all the required information on which subsequent computations are based. It includes all necessary data on intersection geometry, traffic volumes and signalization.

**Volume Adjustment Module**: Demand volumes are generally stated for a peak hour in terms of vehicles per hour (vph). The volume adjustment module converts these to flow rates for a peak 15-minute period and accounts for the effect of lane distribution. The lane groups for the analysis are also defined in this module.

**Saturation flow rate Module**: This module is used for computation of the saturation flow rate for each of the lane groups established. It is based on adjustment of an ideal saturation flow rate to reflect a variety of prevailing conditions.

**Capacity Analysis Module**: In this module the capacity and LOS of each lane group are computed based on the volumes and saturated flow rates. The critical v/c ratio for the entire intersection is also computed here.
**Level of service Module:** In this module, delay is computed for each of the approaches or lane groups established for analysis. Delay measures are aggregated for approaches and for the intersection as a whole and levels of service are determined. A diagrammatic illustration of the operational analysis methodology and its modules is presented in the form of a flow chart through Figure 2.1. Each of these modules are discussed in detail in subsequent articles.

**2.6.3.1 Input Module**

In this module all the input information needed for operational analysis is to be recorded in a comprehensive manner. The date needed is detailed and varied, and it falls into three main categories:

1. **Geometric conditions:** Intersection geometry is generally presented in a diagrammatic form and must include all the relevant information such as approach grades, the number and width of lanes on each approach and parking conditions. The existence of exclusive left or right turns is to be noted.
Fig. 2.1 Flow Chart Showing Operational Analysis Procedure

INPUT MODULE
- Geometric Conditions
- Traffic conditions
- Signalization conditions

VOLUME ADJUSTMENT MODULE
- Peak -hour factor
- Lane group establishment
- Volume assignment to lane groups

SATURATION FLOW RATE MODULE
- Ideal Saturation Flow Rate
- Adjustments
- Adjusted Saturation Flow

CAPACITY ANALYSIS MODULE
- Computation of lane group capacities
- Lane group v/c ratios
- Aggregate results

LEVEL OF SERVICE MODULE
* Computation of lane Group capacities
* Aggregate delays
* Level of service determination
2. Traffic conditions: Traffic volumes for each movement from each approach are to be specified. Vehicle type distribution is to be quantified as the percentage of heavy vehicle (%HV) in each movement, where all the vehicles with more than four wheels touching the pavement are considered to be “heavy vehicles”. Pedestrian flows are needed, as these will interfere with permitted right or left turn movements. In this module a parameter known as “arrival type” is used to quantify the quality of progression on the approaches. Five arrival types are defined in this connection. Type 1 indicates a dense platoon arriving at the intersection at the beginning of the red phase. Type 2 indicates a dense platoon arriving in the middle of the red phase or a dispersed platoon arriving throughout the red phase. Type 3 represents totally random arrivals. This occurs when the arrivals are widely dispersed throughout the red and green phases and / or the approach is totally uncoordinated with other signals. Type 4 indicates a situation where a dense platoon arrives during the middle of the green phase or a dispersed platoon arriving throughout the green phase. Type 5 is defined as a dense platoon arriving at the beginning of the green phase. Another traffic condition of interest is the activity in parking lanes adjacent to analysis lane groups. Parking activity is measured in terms of the parking maneuvers per hour within 250 ft of the intersection, \( N_m \).

3. Signalization conditions: A Complete information regarding signalization is needed for operational analysis of signalized intersection. This includes a phase diagram illustrating the phase plan, cycle length, green times and change intervals.

2.6.3.2. Volume Adjustment Module

Three major analytical steps are performed in this module, i.e., (1) movement volumes are adjusted to flow rates for a peak 15-minute period, (2) lane groups for the analysis are established and (3) lane group flows are adjusted to account for
unbalanced lane utilization. Each of these steps are discussed in more detail in the following sections:

1. Adjustment of movement volumes: The peak 15-minute volume is projected to hourly flow rate in this module by using appropriate peak hour factor, PHF, which may be defined for the intersection as a whole, or for the approach, or for the lane group under consideration. Then computation can be expressed in the form

\[ V_p = \frac{V}{PHF} \]

Where
- \( V_p \) = flow rate during peak 15-minute period, in vph;
- \( V \) = hourly volume, in vph;
- \( PHF \) = peak-hour factor.

Because all intersection movements may not peak at the same time, it is valuable to observe the 15-min flows directly and to select critical periods for analysis. The conversion of peak 15-min flow into hourly flow using PHF assumes that all the movements peak during the same 15-min period, and therefore is a conservative approach.

2. Determination of lane groups for analysis: The operational analysis procedure is disaggregate in the sense that it is designed to consider individual intersection approaches and individual lane groups within the approaches. It is therefore necessary to establish appropriate lane groups for the analysis. A” lane group” is defined as one or more lanes on an intersection approach serving one or more traffic movements. Where two or more lanes are included in a single lane group for analysis purposes, all subsequent computations treat this combination as a single entity. Some common lane group schemes adopted are given in the Table.2.1.
Table 2.1 Common Lane Group Combinations

<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>Movements by Lanes</th>
<th>Lane Group Possibilities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LT+TH+RT</td>
<td>(1)</td>
</tr>
<tr>
<td>2</td>
<td>EXC LT</td>
<td>(2)</td>
</tr>
<tr>
<td></td>
<td>TH+RT</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>LT+TH</td>
<td>(1)</td>
</tr>
<tr>
<td></td>
<td>TH+RT</td>
<td>(2)</td>
</tr>
<tr>
<td></td>
<td>OR</td>
<td></td>
</tr>
</tbody>
</table>

3. Adjustment for lane distribution: Flow rates in each lane group are to be adjusted to reflect unequal lane utilization and that exercise is done in this module. Where more than one lane are included in a lane group, flow may not be equal in both the lanes. The lane utilization adjustment reflects this and increases the analysis flow rate to reflect the flow in the lane with the highest utilization. Thus

\[ v = v_g \times U \]

where \( v \) = adjusted demand flow rate for the lane group, vph;

\[ v_g = \text{unadjusted demand flow rate for the lane group, vph; and} \]

\( U \) = lane utilization factor.

Lane utilization factor varies from 1.00 to 1.10 depending on the number of through lanes in the lane group.
2.6.3.3 Saturation Flow Module

In this module, a saturation flow rate for each lane group is computed. The saturation flow rate is the flow in vph that could be accommodated by the lane group assuming that the green phase is continuously available for it. An ideal saturation flow rate of 1800 passenger cars per hour of green per lane (pcphgpl) is taken as the basis and depending on various adjustment factors the value is converted into vehicles per hour of green (vphg). The computation is as follows;

\[ s = s_o N f_W f_{HV} f_g f_{bb} f_a f_{RT} f_{LT} \]

Where \( s \) = saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group under prevailing conditions, vphg;

\( s_o \) = ideal saturation flow rate per lane, usually 1800 pcphgpl;

\( N \) = number of lanes in the lane group;

\( f_W \) = adjustment factor for lane width (where 12ft lanes are standard and Adjustments are done accordingly)

\( f_{HV} \) = adjustment factor for the presence of heavy vehicles in the traffic;

\( f_g \) = adjustment factor for approach grade;

\( f_p \) = adjustment factor for the influence of parking activity in a parking lane adjacent to the lane group;

\( f_{bb} \) = adjustment factor for the blocking effect by buses stopping in the Intersection area;

\( f_a \) = adjustment factor for area type;

\( f_{RT} \) = adjustment factor for the right turns in the lane group; and

\( f_{LT} \) = adjustment factor for the left turns in the lane group

Where detailed data defining each of the above factors is not available, a default value of 1600 pcphgpl is suggested for \( s \).
2.6.3.4 Capacity Analysis Module

In the capacity analysis module, computational results of the previous modules are used to compute key capacity variables, which include the following:

a) Flow ratio for each lane group
b) Capacity of each lane group
c) v/c ratio for each lane group
d) Critical v/c ratio for the overall intersection

Flow ratios are computed by dividing the adjusted demand flow, v, computed in the volume adjustment module, by the adjusted saturation flow rate, s, computed in the saturation flow rate module. The capacity of the lane group is computed from the following equation:

$$c_i = s_i \cdot X(g/C)_i$$

The v/c ratio for each lane group is computed directly, by dividing the adjusted flow v by the capacity calculated using the equation:

$$X_i = (v/c)_i$$

The final capacity parameter of interest is the critical v/c ratio, Xc, for the entire intersection, computed from the following equation:

$$X_c = \Sigma_i (v/s)_{ci} \cdot X \cdot [C/(C-L)]$$

This ratio indicates the proportion of available capacity, which is being used by the vehicles in the critical lane groups. If this ratio exceeds 1.00, one or more of the critical lane groups will be oversaturated. It is an indication that the intersection design, cycle length, phase plan and/or signal timing inadequate for the existing demand. A ratio less than 1.0, indicates that these parameters are adequate to handle all critical flows without demand exceeding capacity, assuming that green times are proportionately allocated. Where phase splits are not proportional, some movement
demand s may exceed movement capacities even when the critical v/c ratio is less than 1.00.

2.6.3.5 Level of Service Module

In this module, the average stopped delay per vehicle is estimated for each lane group and averaged for approaches. Intersection delay as well as intersection LOS is also calculated in this module. The LOS is directly related to delay value and the delay is computed for each lane group assuming random arrivals, using the following equation:

\[
D = \frac{[1 - \frac{g}{C}]^2}{C} + \frac{173X^2 [(X-1) + \sqrt{(X-1)^2+(16X/c)}]}{[1-(\frac{g}{C})(X)]}
\]

Where:
- \(d\) = average stopped delay for vehicle for the lane group, sec/veh;
- \(C\) = cycle length, sec;
- \(g/C\) = green ratio for the lane group;
- \(X\) = v/c ratio for the lane group; and
- \(c\) = capacity of the lane group.

The first term in the equation accounts for the uniform delay, i.e., the delay occurring if the arrivals are uniformly distributed over time in the subject lane group. The second term of the equation accounts for the incremental delay of random arrivals over uniform arrivals and for the additional delay due to cycle failure. The equation yields reasonable results for values of \(X\) between 0.0 and 1.0. It is often useful to compute the uniform delay and incremental delay terms as separate values. This allows the analyst to see the relative contribution of individual cycle failures to total delay. The equation can be written in the form

\[d = d_1 + d_2\]
Where $d_1 =$ first term uniform delay in sec/veh; and

$d_2 =$ second term incremental delay in sec/veh.

When the signal progression is favorable to the subject lane group, delay will be considerably less than that for radon arrivals. Similarly, when the signal progression is unfavorable, the delay will be considerably higher than that for random arrivals. The variation of delay with progression quality decreases as the v/c ratio approaches 1.00 and is greater for pretimed signals. To take care of the quality of the progression, an adjustment factor for progression is used in this module.

Aggregating the delay estimates for the approach as well as the intersection is also carried out in this module. This is done by computing the weighted averages, where the lane group delays are weighted by the adjusted flow rates in the lane groups. Thus the delay for an approach is computed as

$$d_A = \frac{\Sigma_i d_i v_i}{\Sigma_i v_i}$$

where $d_A =$ delay for the approach A in sec/veh;

$d_i =$ delay for lane group I on approach A in sec/veh; and

$v_i =$ adjusted flow for lane group; in veh/hr.

Approach delays can further be aggregated to provide the average delay for the entire intersection

$$d_I = \frac{\Sigma_A d_A v_A}{\Sigma_A v_A}$$

where  $d_I =$ average delay for per vehicle for the intersection in sec/veh; and

$v_A =$adjusted flow for approach A in veh/hr.

The module also includes computation of the LOS for individual approaches as well as the intersection.
2.7 Application of HCM Procedure to Indian Urban Traffic Situation: Issues and Problems

It can be seen that there are two crucial steps in the HCM procedure for determining the operational performance of a signalized intersection. They are saturation flow rate determination in the saturation flow module and the delay determination in the level of Service module. Both these modules depend heavily on various adjustment factors. The saturation flow module assumes an ideal saturation flow rate of 1800 pcphgpl and uses various adjustment factors to convert the same into vphgpl. Unfortunately, in the case of Indian traffic situation no such ideal saturation flow rate is available, nor is it directly applicable as the passenger car is not a dominant mode. In the process of converting the saturation flow rate from pcphgpl into vphgpl, eight adjustment factors are used.

One factor, $f_w$, depends on the actual lane width available. As the ideal saturation flow rate is defined for a lane of 12 feet width, the adjustment factor brings down the saturation flow rate whenever the lane width is less than 12 feet. Any lane having a width of more than 16 feet is treated as equivalent to 2 lanes.

The adjustment factor for heavy vehicles, $f_{HV}$ ranges from 1.00 to 0.87. Under ideal conditions no heavy vehicles are expected in the traffic stream and therefore, when the percentage of heavy vehicles is zero, the adjustment factor is 1.00. The increase in the percentage of heavy vehicles effects a reduction in the saturation flow rate and the adjustment factors are suggested accordingly. It is quite clear that the adjustment factors are to be determined based on the actual saturation flows measured at different levels of heavy vehicle population.

Another adjustment factor $f_g$ pertains to the approach grade. An upward grade in the approach results in lower saturation flow rate as the grade adversely affects the
release rate. Similarly a downward grade facilitates faster release implying a higher saturation flow rate. The adjustment factors are given accordingly for different percentages of downhill and uphill grades. In case of a downhill grade of 2 percent, the factor is 1.01 and for a 6 percent down grade the factor is 1.03. In case of uphill grade, the factor ranges from 0.97 at 6 percent grade to 0.99 at 2 percent grade. When the approach is level, the adjustment factor is 1.00.

One more important adjustment factor is $f_p$, adjustment factor for the existence of a parking lane and parking activity adjacent to the subject lane group. The degree of influence of parking on saturation flow rate increases with parking activity, expressed in terms of number of parking maneuvers per hour. In case of a single lane in subject lane group, the adjustment factor for parking changes from 0.90 to 0.70 if the number of parking maneuvers in the adjacent parking lane changes from 0 to 40.

Unfortunately the Indian urban traffic system does not have any parking lane concept. Though on street parking is permitted near the intersection, there is no systematic regulation and control and the quantification of the influence of parking activity on the saturation flow rate is extremely difficult. Types of vehicles involved in parking are also very crucial while judging the influence of parking on the saturation flow rate. The adjustment factors suggested by HCM can not be directly applied to Indian traffic conditions because of this contrasting scenario.

A factor $f_{bb}$, called as bus blockage factor is also recommended by HCM in the operational analysis procedure. This factor accounts for the impact of local transit buses stopping near the intersection. The influence of the buses on the saturation flow rate depends on the activity in terms of number of buses stopping near the
intersection per hour. As this number is higher, greater will be the influence on the saturation flow rate.

One of the factors suggested is the area type adjustment factor, which accounts for the inefficiency of intersections near busy areas to function effectively because of congestion. A factor of 0.90 is suggested for CBD areas and for other areas a factor 1.00 is recommended by HCM. The implication is that 10 percent of the capacity of the approach is lost due to congestion resulting by the business activity in the vicinity.

There are two factors related to turning movements $f_{RT}$ for the right turns and $f_{LT}$ for the left turns. The turning adjustment factors depend on a variety of parameters. The most important characteristic is the manner in which the turns are accommodated in the phase plan. The turns may be “protected turns” indicating the provision of an exclusive turning phase during which no conflicts are possible; or “permitted turns”, which are to be made through a conflicting pedestrian or vehicular traffic stream. Obviously the turn adjustment factors depend on the phase plan and the conflicting pedestrian or vehicular traffic in case of permitted phasing. The turn factors basically account for the fact that the turning movements can not be made at the same saturation flow rate as through movements. They consume more of the available green time and the intersection capacity is consequently reduced.

The above discussion makes it very clear that if the procedure suggested by the HCM is to be adopted for saturation flow rate adjustment under Indian urban traffic scenario, two things are essential. One is the determination if an ideal saturation flow rate in pcp/h/gpl to start with and the next is developing the adjustment factors that can be effectively applied to Indian traffic conditions. The absence of lane concept makes it extremely difficult to evolve many of the adjustment factors.
The dominance of smaller modes questions the validity of having an ideal saturation flow rate in pcphgpl. The influence of the adjacent activities on the saturation flow rate needs a highly comprehensive study that needs to encompass innumerable combinations of situations. It appears to be more logical to develop and equation that can directly give the saturation flow rate for an approach in veh/hr, based on the most important parameters, namely the available width and the proportion of highly maneuverable vehicles. Such an equation can eliminate the use of various adjustment factors and the complexity involved in the adoption of HCM procedure can be considerably reduced.

Another aspect of debate for the applicability of HCM procedure to Indian conditions is the determination of progression factor in the LOS module. The delay estimates are made based on this progression factor. If the arrival pattern is purely random, the delay estimated by Webster’s equation need not be adjusted for progression conditions. But the arrival pattern may not be random at all locations. If the upstream intersection is also a signalized one, the platoon getting released in the upstream may have either a favorable progression at the intersection under consideration or an unfavorable progression depending on the signal offset. If the signal progression is favorable to the subject lane group, delays will be considerably less than those expected during random arrivals. Otherwise the delays may exhibit a higher range of values. The variation of delay with progression quality decreases as the v/c ratio approaches 1.00. The variation of delays is also more for a pretimed signal compared to the vehicle actuated signals. As the conditions become more and more favorable for progression, the value reduces so that the expected delay values will be less than those expected for purely random arrival pattern. The determination of arrival type becomes very crucial in delay estimation and LOS determination. The
question that needs to be answered is whether the same progression factor values can be adopted for our urban traffic conditions.

The issue of debate is the LOS criteria based on stopped delay values. If the value suggested by HCM procedure is to be adopted, almost all the intersection approaches at any signalized intersection would come under LOS D or E. But the traffic conditions in reality may be reflecting LOS B or C from the point of view of speeds and congestion levels. This discrepancy also needs to be thoroughly examined for Indian urban traffic conditions. Suitable criteria reflecting the LOS is to be evolved for a signalized urban intersection keeping in view of the multimodal environment.

2.8 Application of Neural Networks in the Performance Evaluation of Signalized Urban Intersections

Trabia and Ande [1999] have developed a fuzzy logic-based adaptive traffic signal controller for an isolated four-approach intersection with through and left-turning movements. The fuzzy controller uses vehicle loop detectors, placed upstream of the intersection on each approach, to measure approach flows and estimate queues. Data from these measurements are used to decide, via two-stage fuzzy logic procedure, whether to extend or terminate the current signal phase. The performance of the two-stage fuzzy controller was compared to that of the traffic-actuated controller for different traffic conditions on a simulated four-approach intersection. The two-stage fuzzy logic controller resulted better performance especially under non-recurring traffic conditions. Changing conditions in traffic volume can cause vehicular delay, if the traffic signal controllers are not adjustable, that is, the parameters of the controller remain the same in changing traffic situations. Bingham [2001] has presented reinforcement learning method for neuro-fuzzy traffic signal
control. Neural network is used in fine-tuning the membership functions of a fuzzy traffic controller. The neural learning algorithm used was reinforcement learning, which gives credit for successful control actions and punishes for poor control actions.

Heung and Ho [1998] have applied a complex fuzzy logic controller for traffic control at a road junction. In this case too the control system has an ability to adapt itself under various traffic conditions. A GA-based offline learning algorithm is employed to generate the fuzzy rules in the situation where the complexity of the controller has increased because the increase in the complexity of junction. The proposed fuzzy logic is also hierarchical approach thus trying to reduce the number of fuzzy rules in the system. Results showed that hierarchical fuzzy logic controller (HFLC) perform better than an ordinary fixed-time traffic controller does under both constant and time varying flow-rates. The effectiveness of urban traffic control systems greatly depends on its ability to react upon changes in traffic patterns.

Roozemond [2001] has investigated the applicability of autonomous intelligent agents in Urban Traffic Control (UTC) and especially in real-time urban intersection control. Proposed system can also adapt itself, based upon internal rules and its environment, at changing environments. The UTC model is primarily based on several Intelligent Traffic Signaling Agents (ITSA) and some authority agents. The proposed system seemed to improve the use of the capacity of intersections. Henry, Farges and Gallego [1998] have developed a neuro-fuzzy control method for controlling of traffic lights of an intersection. System showed good results for simple and medium-complexity intersections but poor performance on a complex intersection. The neuro-fuzzy controller was mixed with the optimal control to
improve performance. Results with this implication were found good for all the intersections tested.

One of the numerous traffic signal control methods has been presented in a paper of Wei et al [2001]. Their paper presents a fuzzy logic adaptive traffic signal controller for an isolated four-approach intersection with through and left-turning movements. The controller has the ability to make adjustments to signal timing in response to observed changes. Also the “urgency degree” term, which can describe the different user’s demands for green time is involved into the fuzzy decision making algorithm. The three level model of fuzzy control can determine whether to extend or terminate the current signal phase and select the sequences of the phases. The fuzzy parameters can be tuned off-line or instead the multi-objective genetic algorithm can be used for the same purpose. Results showed better performance for the fuzzy controller than the traffic-actuated controller. Correspondingly Lin, Kwan and Tung [1997] have introduced a versatile traffic flow model capable of making optimal traffic predictions. Furthermore this model can be used to evaluate various traffic-light timing plans. It also provides a framework for implementing adaptive traffic signal controllers based on fuzzy logic technology. The study of Niittymäki and Nevala [2001] discusses the fuzzy traffic signal control in general and presents some results of fuzzy signal control. Traffic signal control is a control problem with number of complex and sometimes conflicting variables and objects. The final hypothesis is that fuzzy signal control can achieve better performance compared to traditional vehicle actuated signal control.

Beauchamp, Rodriquez and Muniz [1997] have developed a new fuzzy logic approach for traffic control. The developed system is a fuzzy logic based phase sequencer (PS) for signalized intersection control. The phase sequencer operates in
conjunction to the fuzzy logic controller for traffic systems (FLC-TS). PS decides when to finish a phase and also determines what should be the next phase based on the traffic demand and the time elapsed since the last time maneuver was attended. Results did not show a significant difference between the FLC-TS and the PS + FLC-TS. The adaptive tuning of the membership functions and rule base of the PS might result better performance. Ramp controlling or ramp metering is a technique to limit the number of vehicles entering a freeway. Usually, the main goal of the ramp metering system is to avoid congestion and reduce vehicle’s total travel time.

2.9 Scope of the Present Study

The review of the earlier studies and the critical discussion clearly brings out the need for a study that can evolve a procedure to compute the saturation flow rate for the Indian urban traffic scenario dominated by the highly maneuverable group without lane discipline and operating on non-standard carriageway widths.

Thus the scope of the present study is:

a) To understand and analyses the saturation flow rates at signalized urban intersection approaches;

b) To establish a relationship between the saturation flow rates and the two independent variables, i.e., the lane group width and the proportion of highly maneuverable vehicles;

c) To compute the saturation flow rates directly in terms of veh/hour for various geometric and traffic conditions using the relationship developed.

d) To establish an Artificial Neural Network model based on the data obtained from the field studies to predict the saturation flows.
2.10 Summary

The earlier works reported by various researchers on subjects relevant to the present study are reviewed in this chapter. Emphasis is placed on the studies on saturation flow rate estimation under mixed traffic conditions. The Highway Capacity Manual method of analyzing the operational performance of a signalized intersection is reviewed in detail and the issues and problems in applying the same to Indian traffic conditions are thoroughly discussed.