Contents

1 Basic Parameters of Traffic Flow **2**

1.1 Overview . . . 2

1.2 Traffic Stream Parameters . . . 2

1.2.1 Speed . . . 3

1.2.2 Spot Speed . . . 3

1.2.3 Running speed . . . 3

1.2.4 Journey speed . . . 4

1.2.5 Time Mean Speed and Space Mean Speed . . . 4

1.3 Volume . . . 4

1.3.1 Variations of Volume . . . 5

1.3.2 Types of volume measurements . . . 5

1.3.3 Density . . . 6

1.4 Derived characteristics . . . 7

1.4.1 Headway . . . 7

1.4.2 Spacing . . . 7

1.4.3 Travel time . . . 7

1.5 The Time-Space Diagram . . . 8

1.5.1 Time-space diagram of a single vehicle . . . 8

1.5.2 For Many Vehicles . . . 9
1.6 Moving Observer Method for Stream Measurement . . . 11
1.7 Summary . . . 13
1.8 Problems . . . 13

2 Fundamental diagrams **14**

2.1 Overview . . . 14
2.2 Time mean speed (v_t) . . . 14
2.3 Space mean speed (v_s) . . . 14
2.4 Problem 1 . . . 15
2.5 Problem 2 . . . 15
2.6 Relation between time mean speed and space mean speed . . . 17
2.7 Fundamental relations of traffic flow . . . 18
2.8 Fundamental diagrams of traffic flow . . . 18
2.8.1 Speed-density diagram . . . 19
2.8.2 Speed flow relation . . . 20
2.9 Combined diagrams . . . 20
2.10 Summary . . . 22
2.11 Problems . . . 22

3 Traffic data collection **23**

3.1 Overview . . . 23
3.2 Data requirements . . . 23
3.2.1 Measurements at a point . . . 24
3.2.2 Measurements over short section . . . 24
3.2.3 Measurements over long section . . . 25
3.2.4 Measurement over an area . . . 25

3.2.5 Moving Observer Method for Stream Measurement . . . 25
3.3 Summary . . . 29
3.4 Problems . . . 29

4 Traffic stream models **30**

4.1 Overview . . . 30
4.2 Macroscopic stream models . . . 30
4.3 Greenshield's model . . . 30
4.4 Determination of speed flow relations from the field data . . . 33
4.5 Other models . . . 34
4.5.1 Greenberg's logarithmic model . . . 35
4.5.2 Underwood exponential model . . . 35
4.5.3 Pipes'generalized model . . . 36
4.5.4 Multiregime models . . . 36
4.6 Shock waves . . . 37
4.7 Macroscopic flow models . . . 38
4.8 Summary . . . 40
4.9 Problems . . . 40

5 Microscopic traffic flow modelling **41**

5.1 Overview . . . 41
5.2 Notation . . . 42
5.3 Car following models . . . 42
5.3.1 Pipe's Theory . . . 43
5.3.2 Forbes' Theory . . . 43
5.3.3 General Motors' Theory . . . 43
5.3.4 Optimal Velocity Model . . . 44

5.4 GM-CF model . . . 44
5.4.1 Basic Philosophy . . . 44
5.4.2 Follow-the-leader model . . . 45
5.5 Simulation Models . . . 49
5.5.1 applications of simulation . . . 49
5.5.2 Need for simulation models . . . 49
5.5.3 Classification of Simulation Model . . . 50
5.6 Summary . . . 51
5.7 Problems . . . 51

6 Capacity and Level of service **52**

6.1 Overview . . . 52
6.2 Capacity . . . 52
6.3 Level of service . . . 53
6.4 Types of facilities . . . 53
6.5 Highway capacity . . . 55
6.6 Factors affecting level of service . . . 55
6.7 Summary . . . 57
6.8 Problems . . . 57

7 Traffic signs **58**

7.1 Overview . . . 58
7.2 Principles of traffic control devices . . . 58
7.3 Communication tools . . . 59
7.4 Types of traffic signs . . . 60
7.4.1 Regulatory signs . . . 60
7.4.2 Warning signs . . . 62

7.4.3 Informative signs . . . 62
7.5 Summary . . . 64
7.6 Problems . . . 64

8 Road markings **65**

8.1 Overview . . . 65
8.2 Classification of road markings . . . 65
8.3 Longitudinal markings . . . 66
8.3.1 Centre line . . . 66
8.3.2 Traffic lane lines . . . 68
8.3.3 No passing zones . . . 68
8.3.4 Warning lines . . . 69
8.3.5 Edge lines . . . 70
8.4 Transverse markings . . . 70
8.4.1 Stop line . . . 71
8.4.2 Pedestrian crossings . . . 71
8.4.3 Directional arrows . . . 71
8.5 Object marking . . . 72
8.5.1 Objects within the carriageway . . . 73
8.5.2 Objects adjacent to carriageway . . . 73
8.6 Word messages . . . 74
8.7 Parking . . . 74
8.8 Hazardous location . . . 75
8.9 Summary . . . 75
8.10 Problems . . . 75

9 Parking **76**

9.1 Overview . . . 76
9.2 Parking studies . . . 76
9.2.1 Parking statistics . . . 76
9.3 Parking surveys . . . 77
9.4 Ill effects of parking . . . 78
9.5 Parking requirements . . . 78
9.6 On street parking . . . 79
9.7 Off street parking . . . 81
9.8 Summary . . . 86
9.9 Problems . . . 86

10 Traffic intersections **87**

10.1 Overview . . . 87
10.2 Conflicts at an intersection . . . 87
10.3 Levels of intersection control . . . 88
10.3.1 Passive control . . . 88
10.3.2 Semi control . . . 89
10.3.3 Active control . . . 89
10.4 Types of intersection control . . . 90
10.4.1 At grade intersection control . . . 90
10.4.2 Grade separated intersection control . . . 91
10.4.3 Interchange . . . 91
10.5 Channalized intersection . . . 92
10.6 Clover-leaf intersection . . . 94
10.7 Summary . . . 94
10.8 Problems . . . 94

11 Traffic rotaries **95**

11.1 Overview . . . 95

11.2 Advantages and disadvantages of rotary . . . 95

11.3 Guidelines for the selection of rotaries . . . 96

11.4 Traffic operations in a rotary . . . 97

11.4.1 Design elements . . . 98

11.4.2 Design speed . . . 98

11.4.3 Entry, exit and island radius . . . 98

11.4.4 Width of the rotary . . . 99

11.5 Capacity . . . 99

11.6 Summary . . . 102

11.7 Problems . . . 102

12 Traffic signal design **103**

12.1 Overview . . . 103

12.2 Defintions and notations . . . 103

12.3 Signal design procedure . . . 104

12.3.1 Two phase signals . . . 106

12.3.2 Four phase signals . . . 106

12.4 Interval design . . . 108

12.5 Cycle time . . . 109

12.5.1 Effective green time . . . 111

12.5.2 Lane capacity . . . 111

12.5.3 Critical lane . . . 112

12.6 Determination of cycle length . . . 112

12.7 Problems . . . 115

13 Traffic signal design **116**

13.1 Overview 116

13.2 Green splitting 116

13.3 Pedestrian crossing requirements 118

13.4 Performance measures 118

13.5 Summary 123

13.6 Problems 123

Traffic engineering

April 4, 2006

Contents

Chapter 1

Basic Parameters of Traffic Flow

1.1 Overview

1.2 Traffic Stream Parameters

The traffic stream includes a combination of driver and vehicle behavior. The driver or human behavior being non-uniform, traffic stream is also non-uniform in nature. It is influenced not only by the individual characteristics of both vehicle and the human but also by the way a group such units interacts with each other. Thus a flow of traffic through a street of defined characteristics will vary by both location and time corresponding to the changes in the human behavior.

The traffic engineer, but for the purpose of planning and design assumes that these changes are within certain ranges which can be predicted. For eg., if the maximum permissible speed of a highway is 60 mph, the whole traffic stream can be assumed to move on an average speed of 40 mph rather than 100 or 20 mph.

Thus the traffic stream itself is having some parameters on which the characteristics can be predicted. The parameters can be mainly classified as : measurements of quantity, which includes density or concentration and volume or flow of traffic and measurements of quantity which includes speed. The traffic stream parameters can be macroscopic which characterises the traffic as a whole or microscopic which studies the behavior of individual

vehicle in the stream with respect to each other.

As far as the macroscopic characteristics are concerned, they can be grouped as measurement of quantity or quality as described above, i.e. volume, density, and speed. While the microscopic characteristics include the measures of separation, i.e. the headway or separation between vehicles which can be either time or space headway.

1.2.1 Speed

Speed is considered as the quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than a design aspects of the flow. It is defined as the rate of motion in distance per unit of time. Mathematically,

$$v = \frac{d}{t} \tag{1.1}$$

Where, v is speed of vehicle in m/s, d is distance travelled in m in time t seconds. Stream vehicles will be travelling at different speeds.

1.2.2 Spot Speed

Spot speed is the instantaneous speed of a vehicle at a specified location. Spot speed can be used to design the geometry of road like horizontal and vertical curves, super elevation etc. Location and size of signs, design of signals, safe speed, and speed zone determination, all require speed data. Accident analysis, road maintenance, and congestion are the modern fields of traffic engineer, which uses spot speed data as the basic input. Spot speed can be measured using an enoscope, pressure contact tubes or direct timing procedure or the radar speedometer or by different photographic methods.

1.2.3 Running speed

Running speed is the average speed maintained over a particular course while the vehicle is moving and is found by dividing the length of the course by the time the vehicle is in motion.

1.2.4 Journey speed

Journey speed is the effective speed of the vehicle on a journey between two points and is the distance between the two points divided by the total time taken for the vehicle to complete the journey, where the time includes any stopped time due to delays.

If the journey speed is less than running speed, it indicates that the journey follows a stop-go condition with enforced acceleration and deceleration. The spot speed here may vary from 0 to some maximum in excess of the running speed. While uniformity between journey and running speeds denotes comfortable travel conditions.

1.2.5 Time Mean Speed and Space Mean Speed

Time mean speed is defined as the average speed of all vehicles passing a point on a highway over some specified time period. Space mean speed is defined as the average speed of all vehicles occupying a given section of a highway over some specified time period. Both will always be different from each other except in the unlikely event that all vehicles re travelling at the same speed. Time mean speed is a point measurement while space mean speed is a measure relating to length of highway or lane, i.e. the mean speed of vehicles over a period of time at a point in space is time mean speed and the mean speed over a space at a given instant is the space mean speed. Space mean speed can be determined by speeds extracted from video images by recording the distance travelling by all vehicles between a particular pair of frames, i.e.

1.3 Volume

There are practically two ways of counting the number of vehicles on a road. One is volume, which is defined as the number of vehicles that pass a point on a highway or a given lane or direction of a highway during a specific time interval. The measurement is carried out by counting the number of vehicles,n_t, passing a particular point in one lane in a defined period t, and is denoted by the parameter q(vehicles/hour),

$$q = \frac{n_t}{t} \tag{1.2}$$

Volume is expressed in planning and design field taking a day as the measurement of time. Four daily volume

parameters are widely used:

1.3.1 Variations of Volume

The variation of volume with time, i.e. month to month, day to day, hour to hour and within a hour is also as important as volume calculation. Volume variations can be observed from season to season. Volume will be above average in a pleasant motoring month of summer, but will be more pronounced in rural than in urban area. But this is the most consistent of all the variations and affects the traffic stream characteristics the least.

Weekdays, Saturdays and Sundays will also face difference in pattern. But comparing day with day, patterns for routes of a similar nature often show a marked similarity, which is useful in enabling predictions to be made.

The most significant variation is from hour to hour. The peak hour observed during mornings and evenings of weekdays, which is usually 8 to 10 per cent of total daily flow or 2 to 3 times the average hourly volume. These trips are mainly the work trips, which are relatively stable with time and more or less constant from day to day.

1.3.2 Types of volume measurements

1. Average Annual Daily Traffic(AADT): The average 24-hour traffic volume at a given location over a full 365-day year, i.e. the total number of vehicles passing the site in a year divided by 365.

2. Average Annual Weekday Traffic(AAWT): The average 24-hour traffic volume occurring on weekdays over a full year. It is computed by dividing the total weekday traffic volume for the year by 260.

3. Average Daily Traffic(ADT): An average 24-hour traffic volume at a given location for some period of time less than a year. It may be measured for six months, a season, a month, a week, or as little as two days. An ADT is a valid number only for the period over which it was measured.

4. Average Weekday Traffic(AWT): An average 24-hour traffic volume occurring on weekdays for some period of time less than one year, such as for a month or a season.

The relationship between AAWT and AWT is analogous to that between AADT and ADT. Volume in general

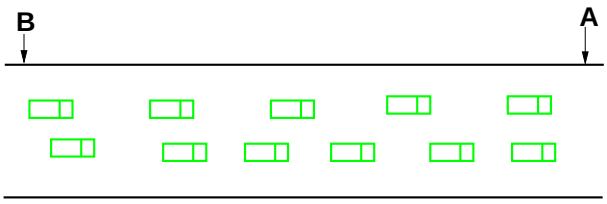


Figure 1.1: Illustration of density

is measured using different ways like manual counting, detector/sensor counting, moving-car observer method, etc. Mainly the volume study establishes:

- the importance of a particular route w.r.t. the other routes,
- the distribution of traffic on road, and
- the fluctuations in flow.

All which eventually determines the design of a highway and the related facilities. Thus, volume is treated as the most important of all the parameters of traffic stream.

1.3.3 Density

Density is defined as the number of vehicles occupying a given length of highway or lane and is generally expressed as vehicles per km/mile. One can photograph a length of road x, count the number of vehicles, n_x, in one lane of the road at a point of time and derive a rate per unit distance. i. e.

$$k = \frac{n_x}{x} \tag{1.3}$$

Density is also equally important as volume but from a different angle as it is the measure most directly related to traffic demand. Again it measures the proximity of vehicles which in turn measures the freedom to maneuver and comfort of drivers.

From the figure 2.1, density = $\frac{number\ of\ vehicles\ in\ between\ the\ points\ A\ and\ B}{distance\ AB}$

1.4 Derived characteristics

1.4.1 Headway

The microscopic character related to volume is the time headway or simply headway. Time headway is defined as the time difference between any two successive vehicles when they cross a given point. Practically, it involves the measurement of time between the passage of one rear bumper and the next past a given point. If all headways h in time period, t, over which flow has been measured are added then,

$$\sum_{1}^{n_t} h_i = t \tag{1.4}$$

Thus, flow is the inverse of th average time headway h, i.e.

$$q = \frac{n_t}{t} = \frac{n_t}{\sum_1^{n_t} h_i} = \frac{1}{\bar{h}} \tag{1.5}$$

Where, n_t is number of vehicles in time t.

1.4.2 Spacing

Another related parameter is the space headway. It is defined as the distance between corresponding points of two successive vehicles at any given time. It involves the measurement from a photograph, the distance from rear bumper of lead vehicle to rear bumper of following vehicle at a point of time. If all th space headways in distance x over which concentration has been measured are added,

$$\sum_{1}^{n_x} = x \tag{1.6}$$

$$k = \frac{n_x}{x} = \frac{n_x}{\sum_1^{n_x} s_i} = \frac{1}{\bar{s}} \tag{1.7}$$

Where, $\bar{s}$ is average space headway.

1.4.3 Travel time

It is the time of travel taken for a journey to get complete. As the speed increases, travel time required to reach the destination also decreases. Thus travel time is inversely proportional to the speed.

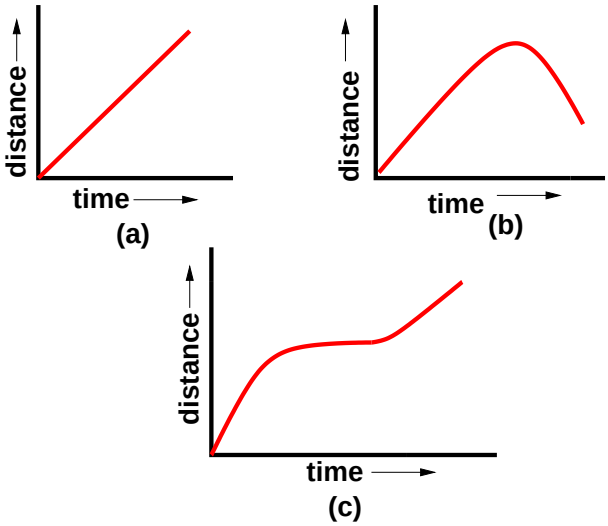


Figure 1.2: Time space diagram for a single vehicle

1.5 The Time-Space Diagram

1.5.1 Time-space diagram of a single vehicle

Taking one vehicle at a time, analysis can be carried out on the position of the vehicle with respect to time. This analysis will generate a graph which gives the relation of its position on a road stretch relative to time. This plot thus will be between distance x and time t and x will be a functions the position of the vehicle for every t along the road stretch. This graphical representation of $x(t)$ in an (t, x) plane is a curve which we call a trajectory. The trajectory provide an intrusive, clear and complete summary of vehicular motion in one dimension.

In figure 1.2(a), the the distance x goes on increasing with respect to the origin as time progresses. The vehicle is moving at a smooth condition along the road way.

In figure 1.2(b), the vehicle through at first moves with a smooth pace after reaching a position reverses its direction of movement.

In figure 1.2(c), the vehicle in between becomes stationary and maintains the same position.

From the figure, steeply increasing section of $x(t)$ denote a rapidly advancing vehicle and horizontal portions of $x(t)$ denote a stopped vehicle while shallow sections show a slow-moving vehicle. A straight line denotes

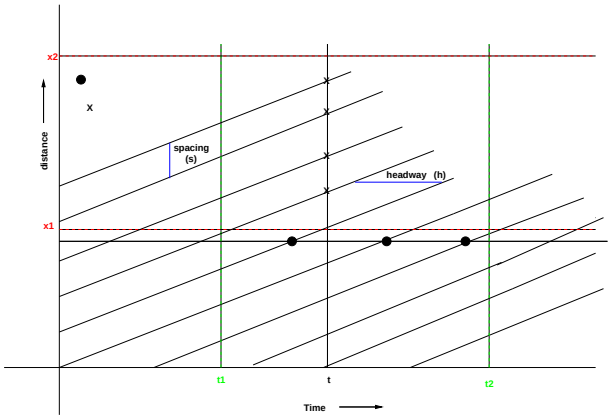


Figure 1.3: Time space diagram for many vehicles

constant speed motion and curving sections denote accelerated motion; and if the curve is concave downwards it denotes acceleration. But a curve convex section denotes deceleration.

1.5.2 For Many Vehicles

Analysis of problems where many vehicles interact while proceeding along a common right-of-way, the time space diagram becomes an invaluable tool. The major fields of these time-space diagrams are for estimating of the safe passing sight distances on two-lance bidirectional highways from the acceleration and speed characteristics of the passing, passed and opposing vehicles and a;sp for scjedi; on trains along a single track rail line with passing allowed to predetermined sidings. Time-space diagram helps to identify and correct errors that many have been made during formulation.

A time-space diagram of a set of vehicles assumed to be travelling at constant speed is shown in figure 1.3.

From the figure 1.3, the number of vehicles passing the stretch of the road at time $= T_1$ is 4, and the density by definition,

$$k = \frac{4\ vehicles}{x2 - x_1} \tag{1.8}$$

and at a distance x_1 from origin, an observer can count 6 vehicles within time t_1 to t_2, i.e.,

$$volume, q = \frac{3\ vehicles}{t_2 - t_1} \tag{1.9}$$

Again the averages taken at a specific location (i.e., time ranging over an interval) are called time means and those taken at an instant over a space interval are termed as space means.

Another related definition which can be given based on the time-space diagram is the headway. Space headway is defined as the distance between corresponding points of two successive vehicles at any given time. Thus, the vertical gap between any two consecutive lines represents space headway. The reciprocal of density otherwise gives the space headway between vehicles at that time.

Similarly, time headway is defined as the time difference between any two successive vehicles when they cross a given point. Thus, the horizontal gap between the vehicles represented by the lines gives the time headway. The reciprocal of flow gives the average time headway between vehicles at that point.

From the fig., flow, $q = \frac{m}{t}$ for the interval T, where m is the number of vehicles. For eg., at $T = t_2 - t_1$, $q = \frac{6}{t_2 - t_1}$. From fig., it is clear that for long observation periods including many vehicles, i.e., $T, m \to \infty$, with comparable headways;

$$T \approx \sum_{i=1}^{m} h_i, \tag{1.10}$$

$$q^{-1} = \frac{T}{m} \simeq \frac{1}{m} \sum_{i=1}^{m} h_i = \bar{h} \tag{1.11}$$

Where, h_i is the time headway and $\bar{h}$ is the average time headway. Similarly, number of vehicles, n, over a stretch of road of a given length, l, lead to the concept of density.

$$k = \frac{n}{l} \tag{1.12}$$

$$k^{-1} = \frac{L}{n} \simeq \frac{1}{n} \sum_{i=1}^{n} s_j = \bar{s} \tag{1.13}$$

Where, s_j is the space headway and $\bar{s}$ is average space headway.

Consider the case of a road section v Km long. The snapshot of this situation is assumed to be taken at time t = 0. Let us assume that all the vehicles are moving at the same speed, u in Kmph. And, let the density be k in vehicles/Km. An observer standing at A starts counting the vehicles at t = 0 and stops when t = 1 hr. The last vehicle which is at v distance from A will take 1 hour to reach him as it is travelling at speed u in Kmph. The number of vehicles passing, the observer starting with vehicle 1 within 1 hour be N. Then, $N = K.u$, i.e. the observer would count $k.u$ vehicles/hr. But, as per definition this is taken as volume,

Therefore, $q = u \times k$

This is the fundamental equation of traffic flow. Speed is important among the parameters of traffic flow for the reason that it measures the quality of travel, which is of main concern for a normal user and driver. In a moving stream, each vehicle moves with different speed. This leads the traffic engineer to decide upon an average value for the distribution of the discrete vehicle speeds. This is represented by either time mean speed or space mean speed. Other concepts like spot speed, running speed, journey speed are also important.

Spot speed is the instantaneous speed of a vehicle at any specified point.

$$SMS = \frac{d}{\frac{\sum t_i}{n}} = \frac{n.d}{\sum t_i} \tag{1.14}$$

and, if the average time to travel a specific distance is taken, it is time mean speed, i.e.

$$TMS = \frac{\sum \frac{d}{t_i}}{n} \tag{1.15}$$

where, d is distance travelled by i^{th} vehicle in travel time t_i, and n is the number of travel times observed. From the above discussion, we can also conclude that space mean speed is a harmonic mean while time mean speed is a simple average of speeds of various vehicles.

Relation between TMS and SMS

The relation between the time-mean speed and space-mean speed has been given by Wardrop, and is given as: Let time mean speed is denoted as $\bar{v}_t$, and space mean speed is denoted as $\bar{v}_s$.

$$\bar{v}_t = \bar{v}_s + \frac{\sigma_s^2}{\bar{v}_s} \tag{1.16}$$

The above equation shows that the time mean speed is never less than the space mean speed.

1.6 Moving Observer Method for Stream Measurement

Determination of any of the two parameters of the traffic flow will provide the third one by the equation $q = u.k$. The most practically used method is the moving observer method, which provides simultaneous measurements of stream variables.

Consider a stream which is in the north bound direction. Two different cases of motion are considered. The first case considers the traffic stream to be moving and observer to be stationary. If n_o is the number of vehicles overtaking the observer during a period, T, then

$$q = \frac{n_o}{T} \tag{1.17}$$

The second case assumes that the stream is stationary but the observer moves with speed V_o, if n_p is the number of vehicles overtaken by observer over a length L, then

$$k = \frac{n_p}{L} \tag{1.18}$$

or

$$n_p = K.V_o.T \tag{1.19}$$

Now consider the case when the observer is moving within the stream. In that case m_o vehicles will overtake the observer and m_p vehicles will be overtaken by the observer in the test vehicle. If the difference is taken,

$$m = m_o - m_p = q.T - k.V_o.T \tag{1.20}$$

$$\frac{m}{T} = q - k.V_o \tag{1.21}$$

This equation is the basic equation of moving observer method, which relates q, k to the counts $m, T and V_o$ that can be obtained. For generating another equation, the test vehicle is run twice once with the traffic stream and another one against traffic stream, i.e.

$$\frac{m_w}{T_w} = q - k.V_w \tag{1.22}$$

$$\frac{m_a}{T_a} = q + k.V_a \tag{1.23}$$

Where, a, w denotes against and with traffic flow. Solving the above:

$$= \frac{m_w + m_a}{T_w + T_a} \tag{1.24}$$

Now to calculate space mean speed:

$$\frac{m_w}{T_w} = q - k.V_w = q - \frac{q}{u}V_w \tag{1.25}$$

$$= q - \frac{q}{u}(\frac{L}{T_w}) \tag{1.26}$$

$$\frac{m_w}{T_w} = q(1 - \frac{L}{u} \times \frac{1}{T_w}) \tag{1.27}$$

$$\frac{m_w}{T_w} = q(1 - \frac{T_{avg}}{T_w}) \tag{1.28}$$

Because $\frac{L}{u}$ is the time that it takes the average vehicle in the stream to transverse length L,

$$\frac{m_w}{q} = T_w(1 - \frac{T_{avg}}{T_w}) = T_w - T_{avg} \tag{1.29}$$

$$T_{avg} = T_w - \frac{m_w}{q} = \frac{L}{u} \tag{1.30}$$

$$u = \frac{L}{T_{avg}} = \frac{L}{T_w - \frac{m_w}{q}} \tag{1.31}$$

Thus two parameters of the stream can be determined. For increase accuracy and reliability, the test is performed a number of times and the average results are taken.

1.7 Summary

1.8 Problems

Chapter 2

Fundamental diagrams

2.1 Overview

2.2 Time mean speed (v_t)

As noted earlier, time mean speed is the average of all vehicles passing a point over a duration of time. It is the simple average of spot speed. Time mean speed v_t is given by,

$$v_t = \frac{1}{n} \sum_{i=1}^{n} v_i \tag{2.1}$$

where v is the spot speed of i^{th} vehicle, and n is the number of observations. In many speed studies, speeds are represented in the form of frequency table. Then the time mean speed is given by,

$$v_t = \frac{\sum_{i=1}^{n} q_i v_i}{\sum_{i=1}^{n} q_i} \tag{2.2}$$

where q_i is the number of vehicles having speed v_i, and n is the number of such speed categories.

2.3 Space mean speed (v_s)

The space mean speed also averages the spot speed, but spatial weightage is given instead of temporal. This is derived as below. Consider unit length of a road, and let v is the spot speed of $i_t h$ vehicle. The time t_i this

vehicle takes to complete the unit distance is given by $\frac{1}{v_i}$. If there are n vehicles, then the average travel time t_s is given by,

$$t_s = \frac{\Sigma t_i}{n} = \frac{1}{n} Sigma \frac{1}{v_i} \tag{2.3}$$

If t_{av} is the average travel time, then average speed $v_s = \frac{1}{t_s}$. Therefore,

$$v_s = \frac{n}{\sum_{i=1}^{n} \frac{1}{v_i}} \tag{2.4}$$

This is simply the harmonic mean of the spot speed. If the spot speeds are expressed as a frequency table, then,

$$v_s = \frac{\sum_{i=1}^{n} q_i}{\sum_{i=1}^{n} \frac{q_i}{v_i}} \tag{2.5}$$

where q_i vehicle will have v_i speed and n_i is the number of observations.

2.4 Problem 1

If the spot speeds are 50, 40, 60,54 and 45, then find the time mean speed and space mean speed.

Solution $v_t = \frac{\Sigma v_i}{n} = \frac{50+40+60+54+45}{5} = \frac{249}{5} = 49.8$ $v_s = \frac{n}{\Sigma \frac{1}{v_i}} = \frac{5}{\frac{1}{50}+\frac{1}{40}+\frac{1}{60}+\frac{1}{54}+\frac{1}{45}} = \frac{5}{0.12} = 48.82$

2.5 Problem 2

The results of a speed study is given in the form of a frequency distribution table. Find the time mean speed and space mean speed.

speed range	frequency
2-5	1
6-9	4
10-13	0
14-17	7

Solution

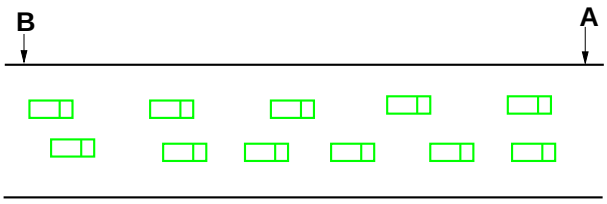


Figure 2.1: Illustration of density

speed range	average speed (v_i)	volume of flow (q_i)	$v_i q_i$	$\frac{q_i}{v_i}$
2-5	3.5	1	3.5	2.29
6-9	7.5	4	30.0	0.54
10-13	11.5	0	0	0
14-17	15.5	7	108.5	0.45
total		12	142	3.28

$v_t = \frac{\Sigma q_i v_i}{\Sigma q_i} = \frac{142}{12} = 11.83$ $v_s = \frac{\Sigma q_i}{\Sigma \frac{q_i}{v_i}} = \frac{12}{3.28} = 3.65$ Inorder to understand the concept of time mean speed and space mean speed, following illustration will help. Let there be a road stretch having two sets of vehicle as in figure 2.1. The first vehicle is travelling at 10m/s with 50 m spacing, and the second set at 20m/s with 100 m spacing. Therefore, the headway of the slow vehicle h_s will be 50 m divided by 10 m/s which is 5 sec. Therefore, the number of slow moving vehicles observed at A in one hour n_s will be $60/5 = 12$ vehicles. The density K is the number of vehicles in 1 km, and is the inverse of spacing. Therefore, $K_s = 1000/50 = 20 vehicles/km$. Therefore, by definition, time mean speed v_t is given by $v_t = \frac{12*10+12*5}{24} = 15 m/s$. Similarly, by definition, space mean speed is the mean of vehicle speeds over time. Therefore, $v_s = \frac{20*10+10*20}{30} = 13.3 m/s$ This is same as the harmonic mean of spot speeds obtained at location A; ie $v_s = \frac{24}{12*\frac{1}{10}+12*\frac{1}{20}} = 13.3 m/s$. It may be noted that since harmonic mean is always lower than the arithmetic mean, and also as observed , space mean speed is always lower than the time mean speed. In other words, space mean speed weights slower vehicles more heavily as they occupy they occupy the road stretch for longer duration of time. For this reason, in many fundamental traffic equations, space mean speed is preferred over time mean speed.

2.6 Relation between time mean speed and space mean speed

Consider a stream of vehicles with a set of substream flow q_1,q_2, $\ldots q_i$, $\ldots q_n$ having speed v_1,v_2, $\ldots v_i$, $\ldots v_n$. From the fundamental relation we have,

$$q = k \times u_s \tag{2.6}$$

$$q_i = k_i \times u_i \tag{2.7}$$

$$\Sigma q_i = q \tag{2.8}$$

$$\Sigma k_i = k \tag{2.9}$$

$$f_i = \frac{k_i}{k} \tag{2.10}$$

$$v_s = \frac{\Sigma k_i v_i}{k} (by\ definition) \tag{2.11}$$

$$v_t = \frac{\Sigma q_i v_i}{q} (by\ definition) \tag{2.12}$$

$$= \frac{\Sigma k_i {u_i}^2}{q} (from\ equation\ 2) \tag{2.13}$$

$$= \frac{k \Sigma f_i {u_i}^2}{q} (from\ equation\ 5) \tag{2.14}$$

$$= \frac{\Sigma f_i {u_i}^2}{u_s} \tag{2.15}$$

$$= \frac{\Sigma f_i (u_s + (u_i - u_s))^2}{u_s} \tag{2.16}$$

$$= \frac{\Sigma f_i (u_s)^2 + (u_i - u_s)^2 + 2.u_s.(u_i - u_s)}{u_s} \tag{2.17}$$

$$= \frac{\Sigma f_i {u_s}^2}{u_s} + \frac{\Sigma f_i (u_i - u_s)^2}{u_s} + \frac{2.u_s.\Sigma f_i (u_i - u_s)}{u_s} \tag{2.18}$$

$$= u_s \Sigma f_i + \frac{\sigma^2}{u_s} + 0 \tag{2.19}$$

$$v_t = u_s + \frac{\sigma^2}{u_s} \tag{2.20}$$

2.7 Fundamental relations of traffic flow

The relationship between the fundamental variables of traffic flow, namely speed, volume, and density is called the fundamentalrelations of traffic flow. This can be derived by a simple concept. Let there be a road with length v km, and assume all the vehicles are moving with vkm/hr.(Fig 2.1. Let the number of vehicles counted by observer at A for one hour be N. By definition, the number of vehicles counted in one hour is flow(q). Therefore,

$$q = n_1 \tag{2.21}$$

By definition, density is the number of vehicles in unit distance. Therefore number of vehicles n_2 is given as:

$$k \times v = n_2 \tag{2.22}$$

Since all the vehicles have speed v, the number of vehicles counted in 1 hour and the number of vehicles in the stretch of distance v will also be same.(ie $n_1 = n_2$. Therefore,

$$q = k \times v \tag{2.23}$$

This is the fundamental equation of traffic flow. Please note that, v in the above equation refers to the space mean speed.

2.8 Fundamental diagrams of traffic flow

The flow and density varies with time and location. The relation between the density and the corresponding flow on a given stretch of road is referred to as the fundamental diagram of traffic flow. Some characteristics of an ideal flow-density relationship is listed below:

1. When the density is zero,since there is no vehicles on the road, flow will also be zero.

2. When the number of vehicles gradually increases the density also increases, so is the flow.

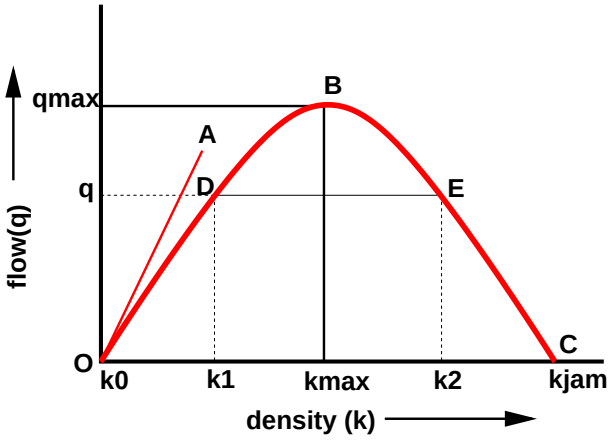


Figure 2.2: Flow density curve

3. When more and more vehicles are added , it reaches a situation where vehicles can't move. This is referred to as the jam density or the maximum density. At jam density, flow will be zero because the vehicles are not moving.

4. There will be some density between zero k and jam k, when the flow is maximum.

The point O refers to the case with zero k and zero q. The point B refers to the maximum flow and the corresponding density is k_max. The point C refers to the maximum density k_jam and the corresponding flow is zero. The slope of the line OA gives the mean free flow speed, ie the speed with which a vehicle can travel when there is no flow. It can also be noted that points D and E correspond to same flow but has two different densities. Further, the slope of the line OD gives the mean speed at density k_1 and slope of the line OE will give mean speed at density k_2. Clearly the speed at density k_1 will be higher since there are less number of vehicles on the road.

2.8.1 Speed-density diagram

Similar to the flow-density relationship, speed will be maximum, referred to as the free flow speed, and when the density is maximum, the speed will be zero. The most simple assumption is that this variation of speed with density is linear as shown by the solid line in figure **??**

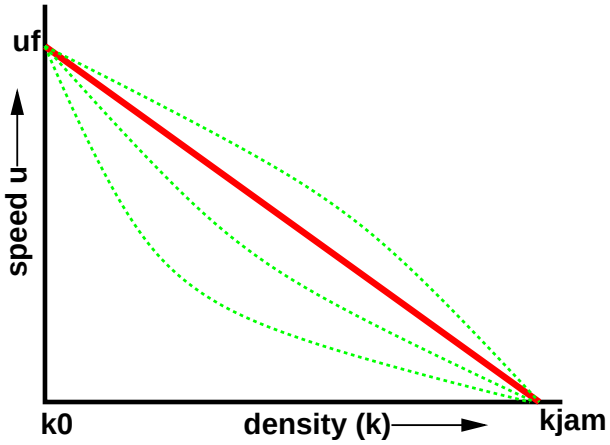


Figure 2.3: Speed-density diagram

It is also possible to have non-linear relationships as shown by the dotted lines. These will be discussed later.

2.8.2 Speed flow relation

The relationship between the speed and flow can be prostulated as:

- (a) the flow is zero either because there is no vehicles or there are too many vehicles so that they cannot move.
- (b) at maximum flow, the speed will be in between zero and free flow speed.

This relationship is shown in figure 2.4 The maximum flow q_max occurs at speed u. It is possible to have two different speeds for a given flow.

2.9 Combined diagrams

The diagrams shown in the relationship between speed-flow, speed-density, and flow-density are called the fundamental diagrams of traffic flow. These are as shown in figure 2.5

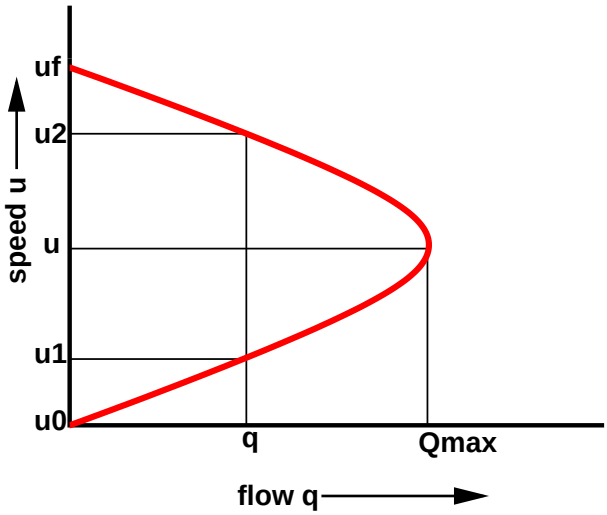


Figure 2.4: Speed-flow diagram

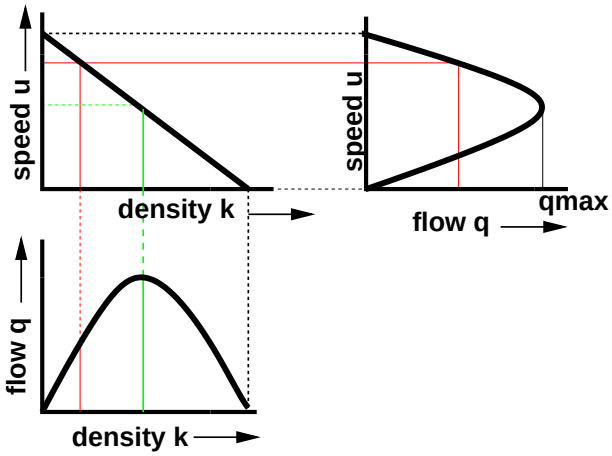


Figure 2.5: Fundamental diagram of traffic flow

2.10 Summary

2.11 Problems

Chapter 3

Traffic data collection

3.1 Overview

Unlike many other disciplines of the engineering, the situations that are interesting to a traffic engineer cannot be reproduced in a laboratory. Even if road and vehicles could be set up in large laboratories,it is impossible to simulate the behaviour of drivers in the laboratory. Therefore, traffic stream characteristics need to be collected from the field only. There are several methods of data collection depending on the need of the study and some important ones are described in this chapter.

3.2 Data requirements

the most important traffic characteristics to be collected from the field includes sped, travel time, flow and density. Some cargo spacing and headway are directly measured. In addition, th occupancy, ie percentage of time a point on the road is occupied by vehicles is also of interest.

The measurement procedures can be classified based on he geographical extent of the survey into five categories: (a)measurement at point on the road (b) measurement over a short section of the road (about kilometers) (c) measurement over a length of the road (about 0.5 km) (d) wide area samples obtained from number of locations, and (e) the use of an observer moving in the traffic stream.

In each category, numerous data collection are there. However, important and basic methods will be discussed.

3.2.1 Measurements at a point

The most important point measurement is the vehicle volume count. Data can be collected manually or automatically. In manual method, an observer standing at the point of interest and count vehicles with the help of hand tallies. Normally, data will be collected for short interval of 5 minutes or 15 minutes etc.and for each types of vehicles like cars, two wheelers, three wheelers, LCV, HCV, multi axle trucks, nonmotorised traffic like bullock cart, hand cart etc.

From the flow data, headway can be derived.

Modern methods include the use of inductive loop detector, video camera, and many other technologies. These methods helps to collect accurate information for long duration. In video cameras, data is collected from the field and is then analysed in the lab for obtaining results. Radars and microwave detectors are used to obtain the speed of a vehicle at a point. Since no length is involved, density cannot be obtained by measuring at a point.

3.2.2 Measurements over short section

The main objective of this study is to find the spot speed of vehicles. Manual methods include the use of enoscope. In this method a base length of about 30-90 metres is marked on the road. Enoscope is placed at one end and observer will stand at the other end. He could see the vehicle passing the farther end through enoscope and starts the stop watch. Then he stops the stop watch when it passes in front of him.

An alternative method is to use pressure contact tube which gives a pressure signal when vehicle moves at either end. Another most widely used method is Inductive loop detector which works on the principle of magnetic inductance. Roads will be cut and a small magnetic loop is placed. When the metallic content in the vehicle passes over it, a signal will be generated and the count of the vehicle can be found automatically if it is connected to a computer. The advantage of this detector is that the counts can be obtained throughout the

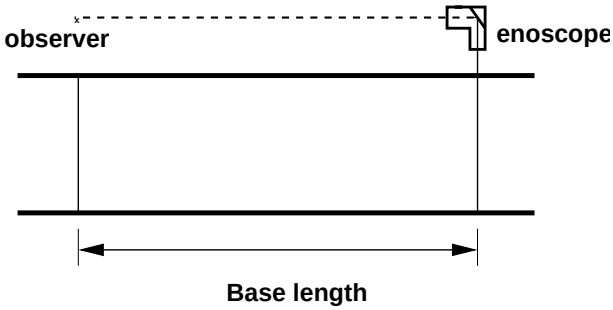


Figure 3.1: Illustration of measurement over short section using enoscope

life time of the road. But chances of errors are possible, because signals may be generated due to the impact when heavy vehicle passes through the next lane. When dual loops are used and if the spacing between them is known then speed also can be calculated.

3.2.3 Measurements over long section

This is normally used to obtain variations in speed. Usually the stretch will be having a length more than 500 metres. We can also get density. The most traditional method is using aerial photography. From the single frames, density can be measured, but not speed or volumes. In time lapse photography, several frames are available . If several frames are used, over short time intervals, speeds can be measured over a distance approximating the entire section length over which densities have been calculated.

3.2.4 Measurement over an area

This is mainly used for traffic control purposes and has the best application in Intelligent transportation systems or ITS.

3.2.5 Moving Observer Method for Stream Measurement

Determination of any of the two parameters of the traffic flow will provide the third one by the equation $q = u.k$. The most practically used method to get the relationship of stream is called the moving observer method, which provides simultaneous measurements of stream variables.

Consider a stream which is in the north bound direction. Two different cases of motion are considered. The first case considers the traffic stream to be moving and observer to be stationary. If n_o is the number of vehicles overtaking the observer during a period, t, then flow q is $\frac{n_0}{t}$, or

$$n_0 = q \times t \tag{3.1}$$

The second case assumes that the stream is stationary but the observer moves with speed v_o, if n_p is the number of vehicles overtaken by observer over a length l, then by definition, density k is $\frac{n_p}{l}$, or

$$n_p = k \times l \tag{3.2}$$

or

$$n_p = k.v_o.t \tag{3.3}$$

where $v - 0$ is the speed of the observer and t is the time taken. Now consider the case when the observer is moving within the stream. In that case m_o vehicles will overtake the observer and m_p vehicles will be overtaken by the observer in the test vehicle. Let the difference m is given by m_0 - m_p, then from equation 3.10 and equation 3.12,

$$m = q.t - k.v_o.t \tag{3.4}$$

This equation is the basic equation of moving observer method, which relates q, k to the counts $m, tandv_o$ that can be obtained from the test. However, we have two unknowns, $qandk$, but only one equation. For generating another equation, the test vehicle is run twice once with the traffic stream and another one against traffic stream, i.e.

$$\begin{aligned} m_w &= q.t_w + k.v_w && (3.5)\\ &= q.t_w + k.l && (3.6)\\ m_a &= qt_a - k.v_a && (3.7)\\ &= q.t_a - k.l && (3.8)\\ & && (3.9) \end{aligned}$$

where, a, w denotes against and with traffic flow. It may be noted that the sign of equation 3.8is negative, because test vehicle moving in the opposite direction can be considered as a case when the test vehicle in moving in the stream with negative velocity. Further, in this case, all the vehicles will be overtaking, since it is moving with negative speed. In other words, when the test vehicle moves in the opposite direction, the observer simply counts the number of vehicle s in the opposite direction. To solve for q, adding equation 3.6 and 3.8, will give,

$$q = \frac{m_w + m_a}{t_w + t_a} \tag{3.10}$$

Now to calculate space mean speed from equation 3.6

$$\begin{aligned}
\frac{m_w}{t_w} &= q - k.v_w \\
&= q - \frac{q}{u} v_w \\
&= q - \frac{q}{u}\left[\frac{l}{t_w}\right] \\
&= q(1 - \frac{l}{u} \times \frac{1}{t_w}) \\
&= q(1 - \frac{t_{avg}}{t_w})
\end{aligned}$$

If v_s is the mean stream speed, then average travel time is given by $t_{avg} = \frac{l}{v_s}$.Therefore,

$$\begin{aligned}
\frac{m_w}{q} &= t_w(1 - \frac{t_{avg}}{t_w}) = t_w - t_{avg} \\
t_{avg} &= t_w - \frac{m_w}{q} = \frac{l}{u}
\end{aligned}$$

$$v_s = \frac{l}{t_{avg}} = \frac{l}{t_w - \frac{m_w}{q}} \tag{3.11}$$

Thus two parameters of the stream can be determined. Knowing the two parameters the third parameter density can be found out as

$$k = \frac{q}{v_s} \tag{3.12}$$

For increase accuracy and reliability, the test is performed a number of times and the average results are taken.

Example 1 The length of the road stretch used for conducting the moving observer test is 0.5 km and the speed with which the test vehicle is moving is 20 km/hr. Given that the number of vehicles in the stream while the test vehicle was moving against the stream is 107, number of vehicles that overtake the test vehicle is 10, and the number of vehicles overtaken by the test vehicle is 74, find the stream flow, density and average speed of the stream.

Solution Time taken by the test vehicle to reach the other end of the stream while it is moving along with the traffic is $t_w = \frac{0.5}{20} = 0.025$ hr Time taken by the observer to reach the other end of the stream while it is moving against the traffic is $t_a = t_w = 0.025$ hr Flow of the stream is given by equation 3.10 $= \frac{107+(10-74)}{0.025+0.025}$ = 860 veh/hr Stream speed v_s can be found out from equation 3.11 $= \frac{0.5}{0.025-\frac{10.74}{860}} = 5$ km/hr Density (from equation 3.12) $= \frac{860}{5} = 172$ veh/km

Example 2 The data from four moving observer methods are shown in the table. Coulmn 1 gives the sample number, column 2 gives the number of vehicles moving against the stream, column 3 gives the number of vehicles that overtake the test vehicle, and last column gives the number of vehicles overtaken by the test vehicle. Find the three fundamental stream parameters for each set of data. Also plot the three fundamental diagrams of traffic flow.

Sample no.	1	2	3
1	107	10	74
2	113	25	41
3	30	15	5
4	79	18	9

Solution

Sample no.	M_a	M_o	M_p	$M(M_o - M_p)$	t_a	t_w	$q = \frac{M_a+M_w}{t_a+t_w}$	$u = \frac{l}{t_w - \frac{M_a}{q}}$	$k = \frac{q}{v}$
1	107	10	74	-64	0.025	0.025	860	5.03	171
2	113	25	41	-16	0.025	0.025	1940	15.04	129
3	30	15	5	10	0.025	0.025	800	40	20
4	79	18	9	9	0.025	0.025	1760	25.14	70

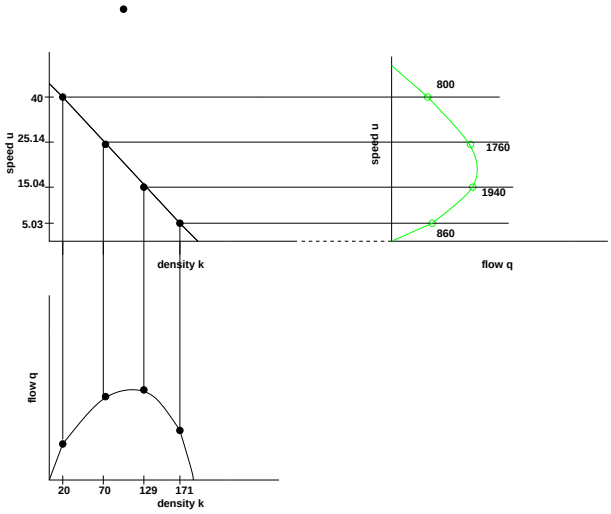


Figure 3.2: Fundamental diagrams of traffic flow

From the calculated values of flow, density and speed, the three fundamental diagrams can be plotted as shown in figure 3.2.

3.3 Summary

3.4 Problems

Chapter 4

Traffic stream models

4.1 Overview

4.2 Macroscopic stream models

4.3 Greenshield's model

Greenshield assumed a linear speed-density relationship as in figure 4.1 to derive the model.

The equation for this relationship is shown below.

$$v = v_f - (\frac{v_f}{k_j}).k \tag{4.1}$$

This equation (4.1) is often referred to as the Greenshields' model. It indicates that when density and flow becomes zero, speed approaches free flow speed (ie. $v \rightarrow v_f$ *when* $q \rightarrow 0$ *and* $k \rightarrow 0$). When the flow becomes maximum (ie. $q \rightarrow q_m$) speed and density reaches their optimum values (ie $u \rightarrow u_0$ *and* $k \rightarrow k_0$) and when density becomes maximum or otherwise called jam density speed and flow becomes zero (ie as $k \rightarrow k_0, u \rightarrow u_0$ *and* $q \rightarrow q_0$). The relationship between speed and flow is shown in figure 4.2.

Also, we know that

$$q = k.v \tag{4.2}$$

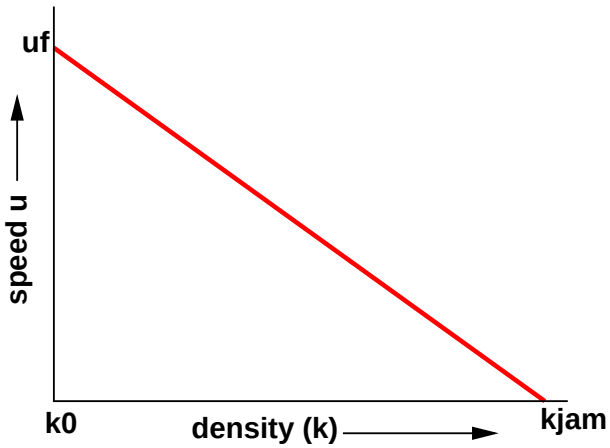


Figure 4.1: Relation between speed and density

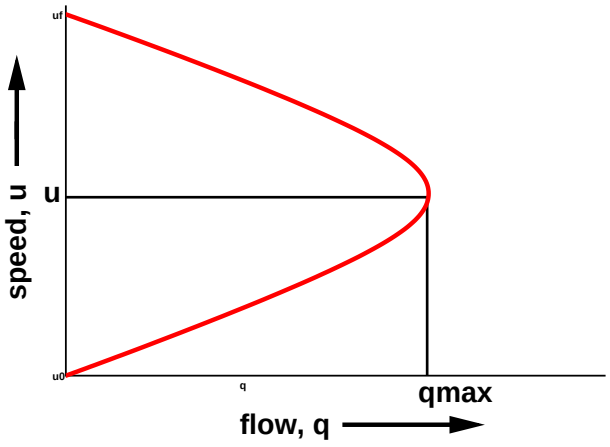


Figure 4.2: Relation between speed and flow

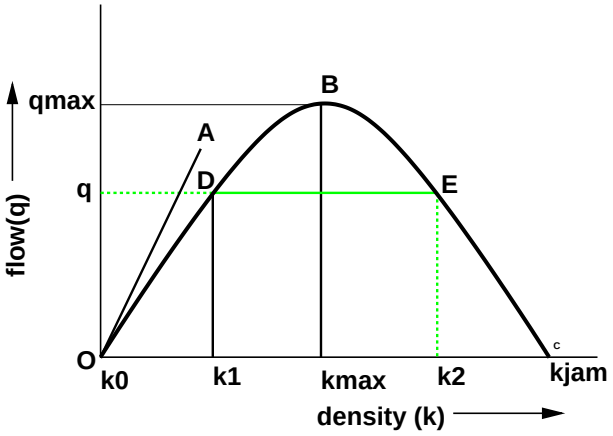


Figure 4.3: Relation between flow and density

The relation between flow and density is shown in figure 4.3.

Now substituting equation 4.1 in equation 4.2, we get

$$q = v_f.k - [\frac{v_f}{k_j}]k^2 \tag{4.3}$$

Put $k = \frac{q}{v}$ in equation 4.1 and solving, we get

$$q = k_j.v - [\frac{k_j}{v_f}]v^2 \tag{4.4}$$

To find density at maximum flow, differentiate equation 4.3 with respect to k and equate it to zero. ie.,

$$\begin{aligned} \frac{dq}{dk} &= 0 \\ v_f - \frac{v_f}{k_j}.2k &= 0 \\ k &= \frac{k_j}{2} \end{aligned}$$

But at maximum flow $k = k_0$, therefore

$$k_0 = \frac{k_j}{2} \tag{4.5}$$

Substituting equation 4.5 in equation 4.3

$$q_max = v_f.\frac{k_j}{2} - \frac{v_f}{k_j}.\frac{k_j}{2}^2$$

$$= \quad v_f.\frac{k_j}{2} - v_f.\frac{k_j}{4}$$

Thus we get,

$$q_max = \frac{v_f.k_j}{4} \tag{4.6}$$

To get the speed at maximum flow, substitute equation 4.5 in equation 4.1. We get,

$$v_0 \quad = \quad v_f - \frac{v_f}{k_j}.\frac{k_j}{2}$$

Solving, we get,

$$v_0 = \frac{v_f}{2} \tag{4.7}$$

4.4 Determination of speed flow relations from the field data

It is difficult to determine free flow speed and jam density directly from the field. Obtain density and speed for a number of observations from the field. Then try to fit a linear equation $y = a + bx$ such that y=k and x=v. Using linear regression method,

$$b \quad = \quad \frac{n.\Sigma xy - \Sigma x.\Sigma y}{n.\Sigma x^2 - \Sigma x^2} \tag{4.8}$$

$$a \quad = \quad \bar{y} - b\bar{x} \tag{4.9}$$

Alternate method of solving for b is,

$$b = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sum_{i=1}^{n}(x_i - \bar{x})^2} \tag{4.10}$$

Problem

For the following data on speed and density, determine the parameters of the Greenshields' model. Also find the density corresponding to a speed of 30 km/hr.

k	v
171	5
129	15
20	40
70	25

Solution

x(k)	y(v)	$(x_i - \bar{x})$	$(y_i - \bar{y})$	$(x_i - \bar{x})(y_i - \bar{y})$	$(x_i - \bar{x}^2)$
171	5	73.5	-16.3	-1198.1	5402.3
129	15	31.5	-6.3	-198.5	992.3
20	40	-77.5	18.7	-1449.3	6006.3
70	25	-27.5	3.7	-101.8	756.3
390	85			-2947.7	13157.2

$\bar{x} = \frac{\Sigma x}{n} = \frac{390}{4} = 97.5$ $\bar{y} = \frac{\Sigma y}{n} = \frac{85}{4} = 21.3$

From equation 4.10, b = $\frac{-2947.7}{13157.2}$ = -0.2 $a = y - b\bar{x}$ = 21.3 + 0.2×97.5 = 40.8

So the actual equation will be,

$$v = 40.8 - 0.2k \tag{4.11}$$

where $v_f = 40.8$ and $\frac{v_f}{k_j} = 0.2$ This implies, $k_j = \frac{40.8}{0.2} = 204$ veh/km From 4.6 $q_{max} = \frac{40.8 \times 204}{4} = 2080.8$ veh/hr

Density corresponding to the speed 30 km/hr can be found out by substituting $v = 30$ in equation 4.11. i.e, 30 = 40.8 - 0.2 × k

Therefore, k = $\frac{40.8-30}{0.2}$ = 54 veh/km

4.5 Other models

In Greenshield's model, linear relationship between speed and density was assumed. But in field we can hardly find such a relationship between speed and density. Therefore, the validity of Greenshields' model was questioned and many other models came up. Some of them will be discussed in this section.

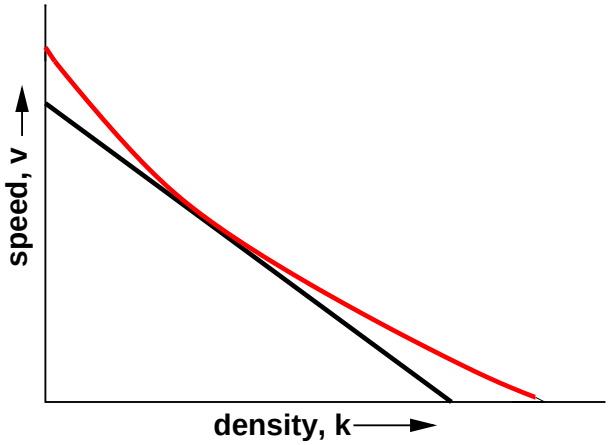


Figure 4.4: Greenberg's logarithmic model

4.5.1 Greenberg's logarithmic model

The second development in this area was the introduction of Greenberg's logarithmic model.

Using a hydrodynamic analogy , he combined the equations of motion and continuity for one-dimensional compressible flow and derived the following equation;

$$v = v_0 l_n \frac{k_j}{k} \tag{4.12}$$

One of the important derivation from this equation is that a relation between macroscopic and microscopic models can be established. However , one of the drawbacks of this model is that as density tends to zero, speed tends to infinity which implies that at low densities, the model has less capability of predicting speeds.

4.5.2 Underwood exponential model

Trying to overcome the limitation of Greenberg's model, Underwood put forward a new model as shown in equation 4.13.

$$v = v_f . e^{\frac{-k}{k_0}} \tag{4.13}$$

The model can be graphically expressed as shown in figure 4.5.

In this model, speed becomes zero only when density reaches infinity which is the drawback of this model. Hence this cannot be used for predicting speeds at high densities.

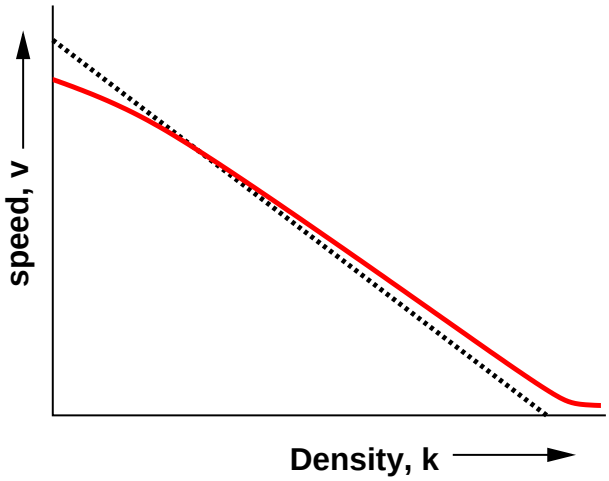


Figure 4.5: Underwood exponential model

4.5.3 Pipes'generalized model

Further development of these models were made with the introduction of a new parameter (n) to provide for a more generalised modelling approach. Pipes proposed a model which is shown by the following equation.

$$v = v_f[1 - (\frac{k}{k_j})^n] \tag{4.14}$$

When n is set equal to one, Greenshields' model is derived. Thus with varying values of n, a family of models can be developed.

4.5.4 Multiregime models

In single regime models as discussed above, an assumption is made that the same speed-density relation is valid for the entire range of densities seen in traffic streams. But human behaviour will be different at different densities. Therefore the speed-density relation will also be different in different zones of densities. Based on this concept, many models were proposed generally called multi-regime models.

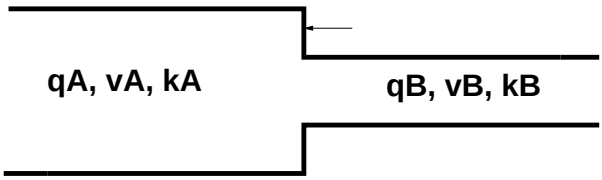


Figure 4.6: Shock wave: Stream characteristics

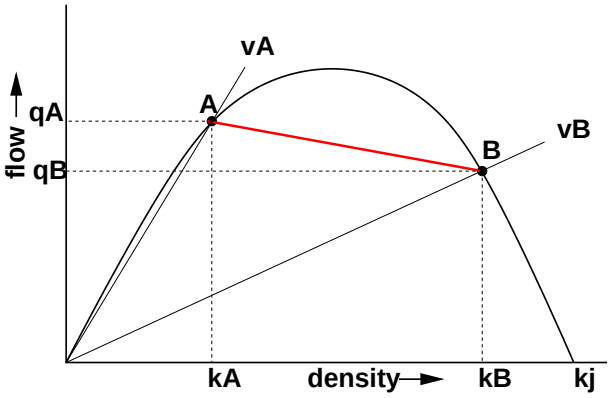


Figure 4.7: Shock wave: Flow-density curve

4.6 Shock waves

The flow of traffic along a stream can be considered similar to a fluid flow. Consider a stream of traffic flowing with steady state conditions, i.e., all the vehicles in the stream is moving with a constant speed, density and flow, say state A (refer figure 4.6.

The flow-density curve is shown in figure 4.7.

The speed of the vehicles at state A is given by the line joining the origin and point A in the graph. The time-distance graph of the traffic stream is also plotted in figure 4.8.

The lines are having the same slope which implies that they are moving with constant speed. Suddenly due to some obstructions in the stream (like an accident or traffic block) the steady state characteristics changes and they acquire another state of flow, say state B. The speed, density and flow of this state is denoted as $v_B, k_B, and q_B$. The sudden change in the characteristics of the stream leads to the formation of a shock wave. There will be a cascading effect of the vehicles in the upstream direction. Thus shock wave is basically the movement of the point that demarcates the two stream conditions. This is clearly marked in the figure 4.7.

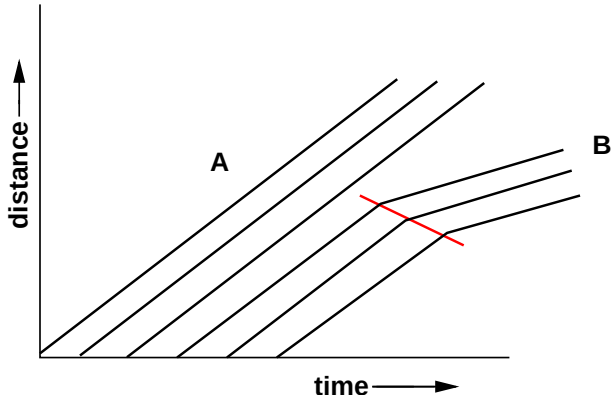


Figure 4.8: Shock wave : time-distance diagram

Thus the shock waves produced at state B are propagated in the backward direction. The speed of the vehicles at state B is the line joining the origin and point B of the flow-density curve. Slope of the line AB gives the speed of the shock wave (refer figure qfshock). If ω_{AB} is used to represent the speed of the shock wave, then

$$\omega_{AB} = \frac{q_A - q_B}{k_A - k_B} \tag{4.15}$$

There are possibilities for other types of shockwaves such as forward moving shockwaves and stationary shockwaves. The forward moving shockwaves are formed when a stream with higher density and higher flow meets a stream with relatively lesser density and flow. For eg, suddenly the width of the road increases, there are chances for a forward moving shockwave. Stationary shockwaves will occur when the streams meeting have the same flow value but different densities.

4.7 Macroscopic flow models

If one looks into traffic flow from a very long distance, the flow of fairly heavy traffic appears like a stream of a fluid. Therefore, a *macroscopic* theory of traffic can be developed with the help of hydrodynamic theory of fluids by considering traffic as an effectively one-dimensional compressible fluid. The behaviour of individual vehicle is ignored and one is concerned only with the behaviour of sizable aggregate of vehicles. The earliest traffic flow models began by writing the balance equation to address vehicle number conservation on a road. Infact, all traffic flow models and theories must satisfy the law of conservation of the number of vehicles on the

road. Assuming that the vehicles are flowing from left to right, the continuity equation can be written as

$$\frac{\partial k(x,t)}{\partial t} + \frac{\partial q(x,t)}{\partial x} = 0 \tag{4.16}$$

where x denotes the spatial coordinate in the direction of traffic flow, t is the time, k is the density and q denotes the flow. However, one cannot get two unknowns, namely $k(x,t)$ by and $q(x,t)$ by solving one equation. One possible solution is to write two equations from two regimes of the flow, say before and after a bottleneck. In this system the flow rate before and after will be same, or

$$k_1 v_1 = k_2 v_2 \tag{4.17}$$

From this the shockwave velocity can be derived as

$$v(t_o)_p = \frac{q_2 - q_1}{k_2 - k_1} \tag{4.18}$$

This is normally referred to as Stock's shockwave formula. An alternate possibility which Lighthill and Whitham adopted in their landmark study is to assume that the flow rate q is determined primarily by the local density k, so that flow q can be treated as a function of only density k. Therefore the number of unknown variables will be reduced to one. Essentially this assumption states that k(x,t) and q (x,t) are not independent of each other. Therefore the continuity equation takes the form

$$\frac{\partial k(x,t)}{\partial t} + \frac{\partial q(k(x,t))}{\partial x} = 0 \tag{4.19}$$

However, the functional relationship between flow q and density k cannot be calculated from fluid-dynamical theory. This has to be either taken as a phenomenological relation derived from the empirical observation or from microscopic theories. Therefore, the flow rate q is a function of the vehicular density k; $q = q(k)$. Thus, the balance equation takes the form

$$\frac{\partial k(x,t)}{\partial t} + \frac{\partial q(k(x,t))}{\partial x} = 0 \tag{4.20}$$

Now there is only one independent variable in the balance equation, the vehicle density k. If initial and boundary conditions are known, this can be solved. Solution to LWR models are kinematic waves moving with velocity

$$\frac{dq(k)}{dk} \tag{4.21}$$

This velocity v_k is positive when the flow rate increases with density, and it is negative when the flow rate decreases with density. In some cases, this function may shift from one regime to the other, and then a shock is said to be formed. This shockwave propagate at the velocity

$$v_s = \frac{q(k_2) - q(k_1)}{k_2 - k_1} \tag{4.22}$$

where $q(k_2)$ and $q(k_1)$ are the flow rates corresponding to the upstream density k_2 and downstream density k_1 of the shockwave. Unlike Stock's shockwave formula there is only one variable here.

4.8 Summary

4.9 Problems

Chapter 5

Microscopic traffic flow modelling

5.1 Overview

Macroscopic modelling looks at traffic flow from a global perspective, whereas microscopic modelling, as the term suggests, gives attention to the details of traffic flow and the interactions taking place within it. This section gives an overview of microscopic approach to modelling traffic and then elaborates on the various concepts associated with it.

A microscopic model of traffic flow attempts to analyse the flow of traffic by modelling driver-driver and driver-road interactions within a traffic stream which respectively analyses the interaction between a driver and another driver on road and of a single driver on the different features of a road. Many studies and researches were carried out on driver's behaviour in different situations - a case when he meets a static obstacle and another one when he meets a dynamic obstacle. More works are made on modelling driver behaviour in car-following situations and the entire body of work is often called as car following theories of vehicular traffic.

The car following theories have their base on mainly the Newtonian equations, which states that the acceleration may be regarded as a response of the particle to the stimulus it receives in the form of force which includes both the external force as well as those from the interaction of other particles of the system.

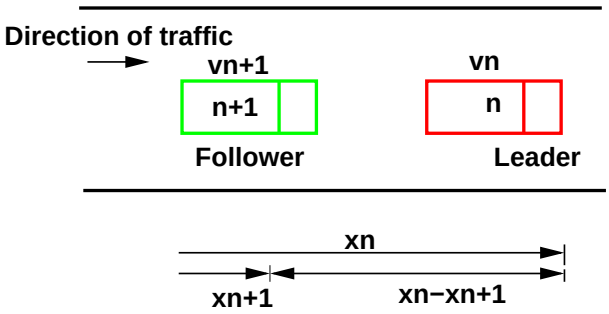


Figure 5.1: Notation for car following model

5.2 Notation

Longitudinal spacing of vehicles is of particular importance from the points of view of safety, capacity and level of service. The longitudinal space occupied by a vehicle depend on the physical dimensions of the vehicles as well as the gaps between vehicles. For measuring this longitudinal space, two microscopic measures are used-distance headway and distance gap. Distance headway is defined as the distance from a selected point (usually front bumper) on the lead vehicle to the corresponding point on the following vehicles. Hence, it includes the length of the lead vehicle and the gap length between the lead and the following vehicles. Before going in to the details, the various notations used in car-following models are discussed here. The leader vehicle is denoted by n and the following vehicle by $(n+1)$ Figure5.1. Two characteristics at an instant t are of importance-location and speed. Location and speed of the lead vehicle are represented by x_n^t and v_n^t respectively. Similarly, the location and speed of the follower are denoted by x_{n+1}^t and v_{n+1}^t respectively. The following vehicle is assumed to accelerate at time $t + \Delta t$ and not at t, where Δt is the interval of time required for a driver to react to a changing situation. The gap between the leader and the follower vehicle is therefore $x_n^t - x_{n+1}^t$.

5.3 Car following models

Car following theories describe how one vehicle follows another vehicle in an uninterrupted flow. Various theories were formulated to model how a driver reacts to the changes in the relative positions of the vehicle ahead. Many researchers worked on these theories and the studies by Pipes, Forbes, General Motors and Optimal velocity

model are worth discussing.

5.3.1 Pipe's Theory

The basic assumption of this theory is "A good rule for following another vehicle at a safe distance is to allow yourself at least the length of a car between your vehicle and the vehicle ahead for every ten miles per hour of speed at which you are travelling" According to Pipe's car-following theory, the minimum safe distance headway increases linearly with speed. A disadvantage of this theory is that at low speeds, the minimum headways proposed by the theory are considerably less than the corresponding field measurements.

5.3.2 Forbes' Theory

In this theory, the reaction time needed for the following vehicle to perceive the need to decelerate and apply the brakes is considered. That is, the time gap between the rear of the leader and the front of the follower should always be equal to or greater than the reaction time. Therefore the minimum time headway is equal to the reaction time(minimum time gap) and the time required for the lead vehicle to traverse a distance equivalent to its length. A disadvantage of this model is that similar to Pipe's model,there is a wide difference in the minimum distance headway at low and high speeds.

5.3.3 General Motors' Theory

The General Motors' theory is the most popular of the car-following theories because of the following:

1. Agreement with field data; The simulation models developed based on General motors' car following models shows good correlation to the field data.

2. Mathematical relation to macroscopic model; The Greenberg's logarithmic model for speed-density relationship can be derived from General motors car following model.

In car following theory, the motion of individual vehicle is governed by an equation, which is analogous to the Newton's Laws of motion. In Newtonian mechanics, acceleration can be regarded as the response of the particle

to *stimulus* it receives in the form of force which includes both the external force as well as those arising from the interaction with all other particles in the system.

5.3.4 Optimal Velocity Model

The concept of this model is that each driver tries to achieve an optimal velocity based on the distance to the preceding vehicle and the speed difference between the vehicles. This was an alternative possibility explored recently in car-following models. The formulation is based on the assumption that the desired speed $V_n^desired$ depends on the distance headway of the nth vehicle. i.e.$V^t_{n_{desired}} = V^{opt}(\Delta x^t_n)$ where v_{opt} is the optimal velocity function which is a function of the instantaneous distance headway Δx^t_n. Therefore a^t_n is given by

$$a^t_n = [1/\tau][V^{opt}(\Delta x^t_n) - v^t_n] \tag{5.1}$$

In short, the driving strategy of n^{th} vehicle is that, it tries to maintain a safe speed which inturn depends on the relative position, rather than relative speed

5.4 GM-CF model

5.4.1 Basic Philosophy

The basic philosophy of car following model is from Newtonian mechanics, where the acceleration may be regarded as the response of a matter to the stimulus it recieves in the form of the force it recieves from the interaction with other particles in the system. Hence, the basic philosophy of car-following theories can be summarised by the following equation

$$[\text{Response}]_n \ \alpha \ [\text{Stimulus}]_n \tag{5.2}$$

for the nth vehicle (n=1, 2, ...). Each driver can respond to the surrounding traffic conditions only by accelerating or decelerating the vehicle. Different theories on car-following have arisen because of the difference in views regarding the nature of the stimulus. The stimulus may be composed of the speed of the vehicle, relative speeds,

distance headway etc, and hence, it is not a single variable, but a function and can be represented as,

$$a_n^t = f_{sti}(v_n, \Delta x_n, \Delta v_n) \tag{5.3}$$

where f_{sti} is the stimulus function that depends on the speed of the current vehicle, relative position and speed with the front vehicle.

5.4.2 Follow-the-leader model

The car following model proposed by General motors is based on follow-the leader concept. This is based on two assumptions; (a) higher the speed of the vehicle, higher will be the spacing between the vehicles and (b) to avoid collision, driver must maintain a safe distance with the vehicle ahead.

Let Δx_{n+1}^t is the gap available for $(n+1)^{th}$ vehicle, and let Δx_{safe} is the safe distance, v_{n+1}^t and v_n^t are the velocities, the gap required is given by,

$$\Delta x_{n+1}^t = \Delta x_{safe} + \tau v_{n+1}^t \tag{5.4}$$

where τ is a sensitivity coefficient. The above equation can be written as

$$x_n - x_{n+1}^t = \Delta x_{safe} + \tau v_{n+1}^t \tag{5.5}$$

Differentiating equation 5.5 with respect to time, we get

$$v_n^t - v_{n+1}^t = \tau . a_{n+1}^t$$

$$a_{n+1}^t = \frac{1}{\tau}[v_n^t - v_{n+1}^t]$$

General Motors has proposed five generations of the sensitivity coefficient term, and the most general model has the form,

$$a_{n+1}^t = [\frac{\alpha_{l,m}(v_{n+1}^t)^m}{(x_n^t - x_{n+1}^t)^l}][v_n^t - v_{n+1}^t] \tag{5.6}$$

where α,l and m are parameters to be calibrated with the field data. In computer, for the implementation of simulation models, three things need to be remembered.

1. A driver will react to the change in speed of the front vehicle after a time gap called the reaction time during which the follower perceives the change in speed and react to it.

2. The vehicle position, speed and acceleration will be updated at certain time intervals depending on the accuracy required. Lower the time interval, higher the accuracy.

3. Vehicle position and speed is governed by Newton's laws of motion, and the acceleration is governed by the car following model.

Let ΔT is the reaction time, and Δt is the updation time, the governing equations can be written as,

$$v_n^t = v_n^{t-\Delta t} + a_n^{t-\Delta t} \times \Delta t \qquad (v = u + at) \tag{5.7}$$

$$x_n^t = x_n^{t-\Delta t} + v_n^{t-\Delta t} \times \Delta t + \frac{1}{2} a_n^{t-\Delta t} \Delta t^2 \qquad (s = ut + \frac{1}{2}at^2) \tag{5.8}$$

$$a_{n+1}^t = [\frac{\alpha_{l,m}(v_{n+1}^{t-\Delta T})}{(x_n^{t-\Delta T} - x_{n+1}^{t-\Delta T})}][v_n^{t-\Delta T} - v_{n+1}^{t-\Delta T}] \tag{5.9}$$

The equation 5.7 is a simulation version of the Newton's simple law of motion $v = u + at$ and equation 5.8 is the simulation version of the Newton's another equation $s = ut + \frac{1}{2}at^2$.

The acceleration of the follower vehicle depends upon the relative velocity of the leader and the follower vehicle, sensitivity coefficient and the gap between the vehicles.

Problem Let a leader vehicle is moving with zero acceleration for two seconds from time zero. Then he accelerates by 1 m/s^2 for 2 seconds, then decelerates by $1m/s^2$for 2 seconds. The initial speed is 16 m/s and initial location is 28 m from datum. A vehicle is following this vehicle with initial speed 16 m/s, and position zero. Simulate the behaviour of the following vehicle using General Motors' Car following model (acceleration, speed and position) for 7.5 seconds. Assume the parameters l=1, m=0 , sensitivity coefficient ($\alpha_{l,m}$) = 13, reaction time as 1 second and scan interval as 0.5 seconds.

t	$a(t)$	$v(t)$	$x(t)$	$a(t)$	$v(t)$	$x(t)$	dv	dx
0.00	0.00	16.00	28.00	0.00	16.00	0.00	0.00	28.00
0.50	0.00	16.00	36.00	0.00	16.00	8.00	0.00	28.00
1.00	0.00	16.00	44.00	0.00	16.00	16.00	0.00	28.00
1.50	0.00	16.00	52.00	0.00	16.00	24.00	0.00	28.00
2.00	1.00	16.00	60.00	0.00	16.00	32.00	0.00	28.00
2.50	1.00	16.50	68.13	0.00	16.00	40.00	0.50	28.13
3.00	1.00	17.00	76.50	0.00	16.00	48.00	1.00	28.50
3.50	1.00	17.50	85.13	0.23	16.00	56.00	1.50	29.13
4.00	-1.00	18.00	94.00	0.46	16.12	64.03	1.88	29.97
4.50	-1.00	17.50	102.88	0.67	16.34	72.14	1.16	30.73
5.00	-1.00	17.00	111.50	0.82	16.68	80.40	0.32	31.10
5.50	-1.00	16.50	119.88	0.49	17.09	88.84	-0.59	31.03
6.00	0.00	16.00	128.00	0.13	17.33	97.45	-1.33	30.55
6.50	0.00	16.00	136.00	-0.25	17.40	106.13	-1.40	29.87
7.00	0.00	16.00	144.00	-0.57	17.28	114.80	-1.28	29.20
7.50	0.00	16.00	152.00	-0.61	16.99	123.36	-0.99	28.64
8.00	0.00	16.00	160.00	-0.57	16.69	131.78	-0.69	28.22
8.50	0.00	16.00	168.00	-0.45	16.40	140.06	-0.40	27.94
9.00	0.00	16.00	176.00	-0.32	16.18	148.20	-0.18	27.80
9.50	0.00	16.00	184.00	-0.19	16.02	156.25	-0.02	27.75
10.00	0.00	16.00	192.00	-0.08	15.93	164.24	0.07	27.76
10.50	0.00	16.00	200.00	-0.01	15.88	172.19	0.12	27.81
11.00	0.00	16.00	208.00	0.03	15.88	180.13	0.12	27.87
11.50	0.00	16.00	216.00	0.05	15.90	188.08	0.10	27.92
12.00	0.00	16.00	224.00	0.06	15.92	196.03	0.08	27.97
12.50	0.00	16.00	232.00	0.05	15.95	204.00	0.05	28.00
13.00	0.00	16.00	240.00	0.04	15.98	211.98	0.02	28.02
13.50	0.00	16.00	248.00	0.02	15.99	219.98	0.01	28.02
14.00	0.00	16.00	256.00	0.01	16.00	227.98	0.00	28.02
14.50	0.00	16.00	264.00	0.00	16.01	235.98	-0.01	28.02

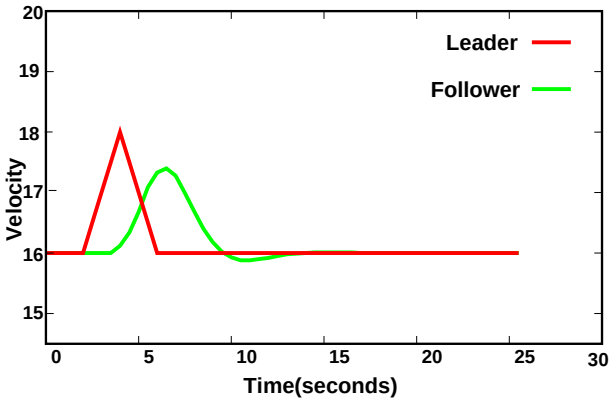


Figure 5.2: Velocity vz Time

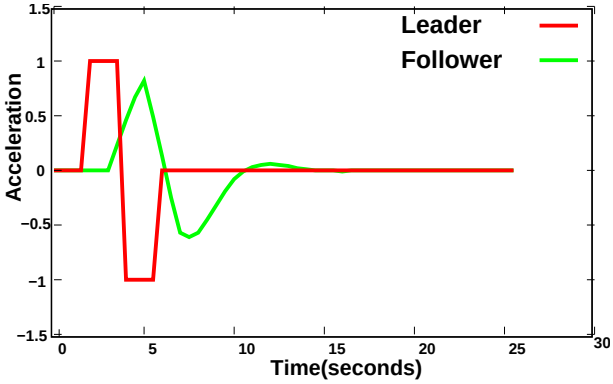


Figure 5.3: Acceleration vz Time

The earliest car-following models considered the difference in speeds between the leader and the follower as the stimulus. It was assumed that every driver tends to move with the same speed as that of the corresponding leading vehicle so that

$$a_n^t = [1/\tau]v_n^{t+1} - v_n^t \tag{5.10}$$

where τ is a parameter that sets the time scale of the model and $1/\tau$ can be considered as a measure of the sensitivity of the driver. According to such models, the driving strategy is to follow the leader and, therefore, such car-following models are collectively referred to as the follow the leader model. Efforts to develop this stimulus function led to five generations of car-following models,a nd the most general model is expressed

mathematically as follows

$$q_{n+1}^{t+\Delta t} = \frac{\alpha_{l,m}\ [v_{n+1}^{t+\Delta t}]^m}{[x_n^t - x_{n+1}^t]^l} \tag{5.11}$$

where l is a distance headway exponent and can take values from +4 to -1, m is a speed exponent and can take values from -2 to +2, and α is a sensitivity coefficient. These parameters are to be calibrated using field data.

5.5 Simulation Models

Simulation modelling is an increasingly popular and effective tool for analysing a wide variety of dynamical problems which are difficult to be studied by other means. Usually, these processes are characterized by the interaction of many system components or entities.

5.5.1 applications of simulation

Traffic simulations models can meet a wide range of requirements:

1. Evaluation of alternative treatments
2. Testing new designs
3. As an element of the design process
4. Embed in other tools
5. Training personnel
6. Safety Analysis

5.5.2 Need for simulation models

Simulation models are required in the following conditions

1. Mathematical treatment of a problem is infeasible or inadequate due to its temporal or spatial scale

2. The accuracy or applicability of the results of a mathematical formulation is doubtful, because of the assumptions underlying (e.g., a linear program) or an heuristic procedure (e.g., those in the Highway Capacity Manual)

3. The mathematical formulation represents the dynamic traffic/control environment as a simpler quasi steady state system.

4. There is a need to view vehicle animation displays to gain an understanding of how the system is behaving

5. Training personnel

6. Congested conditions persist over a significant time.

5.5.3 Classification of Simulation Model

Simulation models are classified based on many factors like

1. Continuity
   (a) Continuous model
   (b) Discrete model
2. Level of detail
   (a) Macroscopic models
   (b) Mesoscopic models
   (c) Microscopic models
3. Based on Processes
   (a) Deterministic
   (b) Stochastic

5.6 Summary

5.7 Problems

Chapter 6

Capacity and Level of service

6.1 Overview

Some of the issues addressed in traffic capacity analysis is a clear understanding of how much traffic a given transportation facility can accommodate. Capacity and Level of service are two related terms. Level of service tries to answer how good is the present traffic situation on a given facility. Thus it gives a qualitative measure of traffic.

6.2 Capacity

Capacity is defined as the maximum number of vehicles, passengers, or the like, per unit time, which can be accommodated under given conditions with a reasonable expectation of occurrence.

Some of the observations that are found from this definition can be now discussed. Capacity is independent of the demand. It speaks about the physical amount of vehicles and passengers a road can afford. It does not depend on the total number of vehicles demanding service. But it depends on traffic conditions, geometric design of the road etc. For example, a curved road has lesser capacity compared to a straight road. It is expressed in terms of units of some specific thing (car, people, etc.), so it does depend on the traffic composition. It depends on the environmental conditions also. Capacity is a probabilistic measure. It varies with respect to

time and position in the maximum number of units of demand it can accommodate by similar facilities. Hence it is not always possible to completely derive analytically the capacity. In most cases it is obtained through field observations.

6.3 Level of service

A term closely related to capacity and confused with it is service volume. But when capacity gives a quantitative measure of traffic, level of service or LOS tries to give a qualitative measure. A service volume is the maximum number of vehicles, passengers, or the like, which can be accommodated by a given facility or system under given conditions at a given level of service.

For a given road or facility, capacity could be constant. But actual flow will be different for different days and different times in a day itself. The intention of LOS is to relate the traffic service quality to given flow rate of traffic. It is a term that designates a range of operating conditions on a particular type of facility. Highway Capacity manual (HCM) has evolved some principles and divided the quality of traffic into six levels ranging form level A to level E. Level A represents the best quality of traffic where the driver has the freedom to drive with free flow speed and level E represents the worst quality of traffic. Level of service is defined based on the measure of effectiveness or (MOE). Typically three parameters are used under this and they are speed and travel time, density, and delay. One of the important measures of service quality is the amount of time spent in travel. Therefore speed and travel time are considered to be effective in defining LOS of a facility. Density is the proximity of other vehicles in the stream. Since it helps to calculate the ability of the driver to maneuver in the traffic stream, it is also used to describe LOS. Delay is a term that describes excess or unexpected time spent in travel. Many specific delay measures are defined and used as MOE's in the highway capacity manual.

6.4 Types of facilities

Most important classification of transportation facilities from the engineering perspective is as *uninterrupted flow* and *interrupted flow*. Uninterrupted flow is the flow of traffic in which there is no obstructions to the

movement of vehicles along the road. Freeway is one example for this type of facility. In a freeway, when a vehicle enters a freeway, there is no need for the vehicle to stop anywhere till it leaves the freeway. There are three sections in a freeway - basic unit, weaving section and ramps(on/off). Vehicles will be entering the freeway through ramps. Ramps used for entering the freeway is called on-ramps and those used for exiting the freeway are called off-ramps. Freeways generally have 4, 6, or 8 lane alignments. Multilanes also provide uninterrupted flow. In multilanes there will be signalized as well as unsignalized intersections. Uninterrupted flow is possible in sections of rural and suburban multilane highways between signalized intersections where signal spacing is sufficient to allow for uninterrupted flow. Two lane highways also provide uninterrupted flow facilities.

Interrupted flow refers to the condition when the traffic flow on the road is obstructed due to some reasons. This is experienced in signalized intersections, unsignalized intersections, arterials etc. At signalized intersections, there will be some kind of active control and the vehicle will have to stop or sometimes reduce its speed and the flow of traffic is interrupted and that capacity is defined in terms of control delay ie sec/veh. Arterials are roads of long stretches with many intersections in between and obviously there will be interruption to the flow of traffic. Here the capacity is expressed in terms of average travel speed. Some other facilities are facilities for pedestrians, bicycles, bus-transit facilities, rail-transit facilities etc. Example for pedestrian facility is a provision of subway exclusively for the use of pedestrians. Here the capacity may be expressed in terms of number of passengers. In bus transit system, the buses has to stop at the bus bays and also it has to share the road with the other vehicles. Hence the capacity will be affected by the control characteristics and the traffic conditions prevailing in the road. Since trains have exclusive right of way, the capacity is strictly governed by the control characteristics. It has two types of capacities - line capacity and station capacity. Line capacity is based on the number of tracks available between two stations. Station capacity refers to the facilities available in the platform of the station,and other facilities.

For uninterrupted flow of traffic, MOE is density in freeways. Speed also becomes important in two-lane highways and multilane highways. In the case of interrupted flow, MOE is delay. The delay of travel time becomes an important factor in calculating the capacity.

6.5 Highway capacity

Highway capacity is defined by the Highway Capacity Manual as follows: The maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or a uniform segment of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions. The highway capacity depends on certain conditions as listed below;

1. Traffic conditions: It refers to the traffic composition in the road such as the mix of cars, trucks, buses etc in the stream. It also include peaking characteristics, proportions of turning movements at intersections etc.

2. Road way characteristics: This points out to the geometric characteristics of the road. These include lane width, shoulder width, lane configuration, horizontal alignment and vertical alignment.

3. Control conditions: This primarily applies to surface facilities and often refer to the signals at intersections etc.

Again in the definition, it is defined for a *point or uniform section*. Capacity is estimated for segments having uniform conditions. Points where these conditions change represent the boundaries where separate analysis may be required. Capacity is the maximum flow rate that a facility can afford. This maximum flow rate is taken for the worst 15 minutes of the peak hours while finding out the capacity. Capacity is measured as a reasonably expected value and not the maximum flow rate ever observed in the facility. This is because the measured capacity at a single location will show significant variation from day to day. Further, local driving habits also produce variations in the observed capacity.

6.6 Factors affecting level of service

Level of service was introduced in Highway capacity manual(HCM) to denote the level of service one can derive from a road under different operating characteristics and traffic volumes. The factors affecting Level of service (LOS) can be listed as follows:

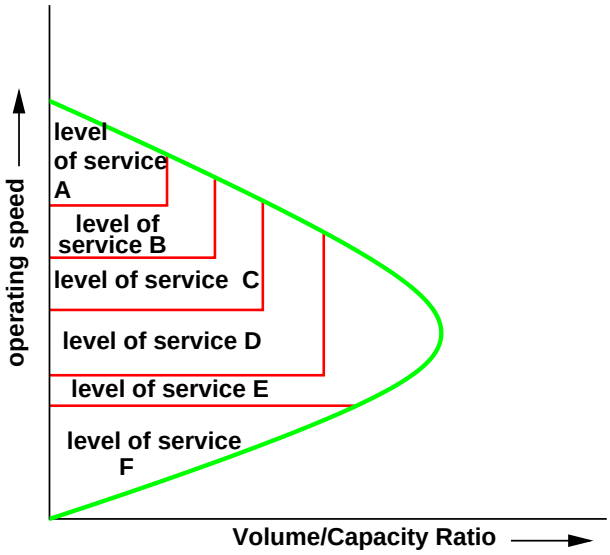


Figure 6.1: Graph between operating speed and v/c ratio

1. Speed and travel time
2. Traffic interruptions/restrictions
3. Freedom to travel with desired speed
4. Driver comfort and convenience
5. operating cost.

Highway Capacity Manual(HCM) used travel speed and volume by capacity ratio (v/c ratio) to distinguish between various levels of service. The value of v/c ratio can vary between 0 and 1. Depending upon the travel speed and v/c ratio, HCM has defined six levels of service, level A to level F. A graph between operating speed and v/c ratio is shown in the figure 6.1. Level of service A represents the zone of free flow. Here the traffic volume will be less, traffic will be experiencing free flow also. The drivers will be having the complete freedom to choose their desired speed. Even at maximum density, for this LOS the average spacing between vehicles is 167 m. Lane changes within the traffic stream, as well as merging and diverging movements, are made relatively easy. The effect of minor incidents and point breakdowns are easily aborted at this level. Level of service B represents zone of reasonably free flow. Free flow speeds are still maintained at this level of service. The drivers

freedom to choose their desired speed is only slightly restricted The lowest average spacing between vehicles is about 100 m. The effects of minor incidents and point breakdowns are still easily aborted. At level of service C, the presence of other vehicles begins to restrict the maneuverability within the traffic stream. Average speeds remain at or near the freeflow speed level, but significant increase in driver vigilance is required at this level. Minimum average spacing between the vehicles is in the range of 67 m. Queues may be expected to form behind any significant blockage. At level of service D, the average speeds begin to decline with increasing flows. Freedom to maneuver within the traffic stream is noticeably restricted. At this level, density deteriorates more quickly with flow. The spacing between the vehicles is about 50 m. As the traffic stream has little space to absorb disruptions, minor incidents can lead to queuing of vehicles. Level of service E define operation at capacity. At this level, the stream reaches it's maximum density limit. There will be no usable gaps in the stream and even slight disruptions will cause a breakdown, with queues forming rapidly behind the disruption. Maneuvering within the traffic stream becomes extremely difficult. Level of service F describes conditions in a queue that has formed behind a point of breakdown or disruption. As vehicles shuffle through the queue, there may be periods when they move quickly, and others when they are stopped completely. Thus this level of service is used to describe the point of breakdown as well, eventhough operations downstream of such a breakdown may appear good.

Level of service E represents the region of forced flow, having low speed .

6.7 Summary

6.8 Problems

Chapter 7

Traffic signs

7.1 Overview

Traffic control devices is the media used for communicating between traffic engineer and road users. Unlike other modes of transportation, there is no control on the drivers using the road. Here traffic control devices comes to the help of the traffic engineer. The types of traffic control used are- traffic signs, road markings , traffic signals and parking control.

7.2 Principles of traffic control devices

1. The control device should fulfill a need : Each device must have a specific purpose for the safe and efficient operation of traffic flow. The superfluous devices should not be used.

2. It should command attention from the road users: This affects the design of signs. For commanding attention, proper visibility should be there. Also the sign should be distinctive and clear. The sign should be placed in such a way that the driver requires no extra effort to see the sign.

3. It should convey a clear, simple meaning: Clarity and simplicity of message is essential for the driver to properly understand the meaning in short time. The use of colour, shape and legend as codes becomes

important in this regard. The legend should be kept short and simple so that even a less educated driver could understand the message in less time.

4. Road users must respect the signs: Respect is commanded only when the drivers are conditioned to expect that all devices carry meaningful and important messages. Overuse, misuse and confusing messages of devices tends the drivers to ignore them.

5. The control device should provide adequate time for proper response from the road users: This is again related to the design aspect of traffic control devices. The sign boards should be placed at a distance such that the driver could see it and gets sufficient time to respond to the situation. For example, the STOP sign which is always placed at the stop line of the intersection should be visible for atleast one safe stopping sight distance away from the stop line.

7.3 Communication tools

A number of mechanisms are used by the traffic engineer to communicate with the road user. These mechanisms recognize certain human limitations, particularly eyesight. Messages are conveyed through the following elements.

1. colour: It is the first and most easily noticed characteristics of a device. Usage of different colours for different signs are important. The most commonly used colours are red, green, yellow, black, blue, and brown . These are used to code certain devices and to reinforce specific messages. Consistent use of colours helps the drivers to identify the presence of sign board ahead.

2. shape : It is the second element discerned by the driver next to the colour of the device. The categories of shapes normally used are circular, triangular, rectangular, and diamond shape. Two exceptional shapes used in traffic signs are octagonal shape for STOP sign and use of inverted triangle for GIVE WAY (YIELD) sign. Diamond shape signs are not generally used in India.

3. legend : This is the last element of a device that the drive comprehends. This is an important aspect in

the case of traffic signs. For the easy understanding by the driver, the legend should be short, simple and specific so that it does not divert the attention of the driver. Symbols are normally used as legends so that even a person unable to read the language will be able to understand that. There is no need of it in the case of traffic sifnals and road markings.

4. pattern: It is normally used in the application of road markings, complementing traffic signs. Generally solid, double solid and dotted lines are used. Each pattern conveys different type of meaning. The frequent and consistent use of pattern to convey information is recommended so that the drivers get accustomed to the different types of markings and can instantly recognize them.

7.4 Types of traffic signs

There are several hundreds of traffic signs available covering wide variety of traffic situations.They can be classified into three main categories.

1. Regulatory signs
2. Warning signs
3. Informative signs

In addition special type of traffic sign namely *Work zone signs* are also available. These type of signs are used to give warning to the road users when some construction work is going on the road. They are placed only for short duration and will be removed soon after the work is over and when the road is brought back to its normal condition. The first three signs will be discussed in detail now.

7.4.1 Regulatory signs

These signs are also called mandatory signs because it is mandatory that the drivers must obey these signs. If the driver fails to obey them, the control agency has the right to take legal action against the driver. These signs are primarily meant for the safety of other road users. These signs have generally black legend on a white background. They are circular in shape with red borders. The regulatory signs can be further classified into :

1. Right of way series: These include two unique signs that assign the right of way to the selected approaches of an intersection. They are the STOP sign and GIVE WAY sign For example, when one minor road and major road meets at an intersection, preference should be given to the vehicles passing through the major road. Hence the give way sign board will be placed on the minor road to inform the driver on the minor road that he should give way for the vehicles on the major road. In case two major roads are meeting, then the traffic engineer decides based on the traffic on which approach the sign board has to be placed. Stop sign is another example of regulatory signs that comes in right of way series which requires the driver to stop the vehicle at the stop line.

2. Speed series: Number of speed signs may be used to limit the speed of the vehicle on the road. They include typical speed limit signs, truck speed, minimum speed signs etc. Speed limit signs are placed to limit the speed of the vehicle to a paricular speed for many reasons. Separate truck speed limits are applied on high speed roadways where heavy commercial vehicles must be limited to slower speeds than passenger cars for safety reasons. Minimum speed limits are applied on high speed roads like expressways, freeways etc. where safety is again a predominant reason. Very slow vehicles may present hazard to themselves and other vehicles also.

3. Movement series: They contain a number of signs that affect specific vehicle maneuvers. These include turn signs, alignment signs, exclusion signs, oneway signs etc. Turn signs include turn prohibitions and lane use control signs. Lane use signs make use of arrows to specify the movements which all vehicles in the lane must take. Turn signs are used to safely accommodate turns in unsignalized intersections.

4. Parking series: They include parking signs which indicate not only parking prohibitions or restrictions, but also indicate places where parking is permitted, the type of vehicle to be parked, duration for parking etc.

5. Pedestrian series: They include both legend and symbol signs. These signs are meant for the safety of pedestrians and include signs indicating pedestrian only roads, pedestrian crossing sites etc.

6. Miscellaneous: Wide variety of signs that are included in this category are: a "KEEP OF MEDIAN"

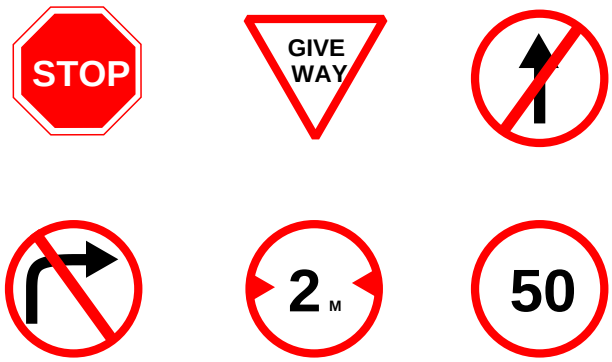


Figure 7.1: Regulatory signs

sign, signs indicating road closures, signs restricting vehicles carrying hazardous cargo or substances, signs indicating vehicle weight limitations etc.

Some examples of the regulatory signs are shown in figure 7.1. They include a stop sign, give way sign, signs for no entry, sign indicating prohibition for right turn, speed limit sign, vehicle width limit sign etc.

7.4.2 Warning signs

Warning signs or cautionary signs give information to the driver about the impending road condition. They advice the driver to obey the rules. These signs are meant for the own safety of drivers. They call for extra vigilance from the part of drivers.

The colour convention used for this type of signs is the legend will be black in colour with a white background. The shape used is upward traingular or diamond shape with red borders. Some of the examples for this type of signs are given in fig 7.2 and includes right hand curve sign board, signs for narrow road, sign indicating railway track ahead etc.

7.4.3 Informative signs

Informative signs also called guide signs, are provided to assist the drivers to reach their desired destinations. These are predominantly meant for the drivers who are unfamiliar to the place. The guide signs are redundant for the users who are accustomed to the location.

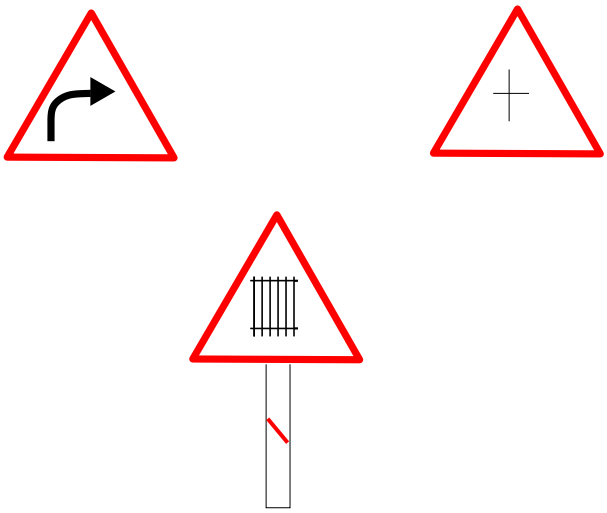


Figure 7.2: Cautionary signs

Some of the examples for these type of signs are route markers, destination signs, mile posts, service information, recreational and cultural interest area signing etc. Route markers are used to identify numbered highways. They have designs that are distinctive and unique. They are written black letters on yellow background. Destination signs are used to indicate the direction to the critical destination points, and to mark important intersections. Distance in kilometers are sometimes marked to the right side of the destination. They are, in general, rectangular with the long dimension in the horizontal direction. They are colour coded as white letters with green background.

Mile posts are provided to inform the driver about the progress along a route to reach his destination. Service guide signs give information to the driver regarding various services such as food, fuel, medical assistance etc. They are written with white letters on blue background. Information on historic, recreational and other cultural area is given on white letters with brown background. In the figure 7.3 we can see some examples for informative signs which include route markers, destination signs, mile posts, service centre information etc..

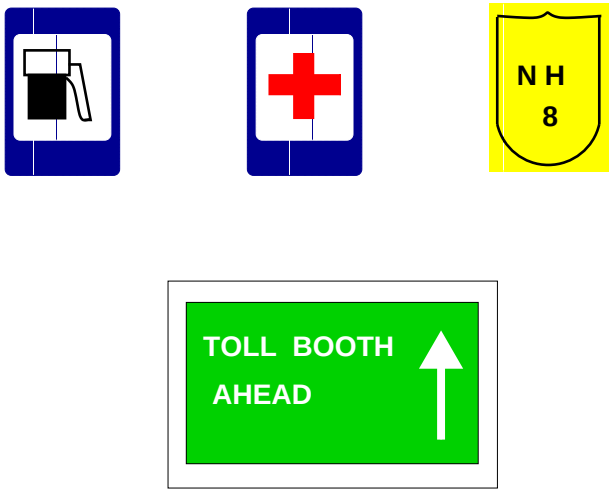


Figure 7.3: Informative signs

7.5 Summary

7.6 Problems

Chapter 8

Road markings

8.1 Overview

The essential purpose of road markings is to guide and control traffic on a highway. They supplement the function of traffic signs. The markings serve as a psycological barrier and signify the delineation of traffic path and its lateral clearence from traffic hazards for the safe movement of traffic. Hence they are very important to ensure the safe, smooth and harmonious flow of traffic.

8.2 Classification of road markings

The road markings are defined as lines, patterns, words or other devices, except signs, set into applied or attached to the carriageway or kerbs or to objects within or adjacent to the carriageway, for controlling, warning, guiding and informing the users. The road markings are classified as longitudinal markings, transverse markings, object markings, word messages, marking for parkings, marking at hazardous locations etc.

8.3 Longitudinal markings

They are marked along the direction of traffic on the roadway surface, for the purpose of indicating to the driver, his proper position on the roadway. Some of the guiding principles in longitudinal markings may also be discussed.

White is the predominant colour used in the case of longitudinal markings for separating traffic flow in the same direction. Yellow colour is used to separate the traffic flow in opposite direction and also to separate the edges. The lines can be either broken, solid or double solid. Broken lines are permissive in character and allows crossing with discretion, if traffic permits. Solid lines are restrictive in character and does not allow crossing except for entry or exit fraom a side road or premises or to avoid a stationary obstruction. Double solid lines indicate severity in restrictions and should not be crossed except in case of emergency. There can also be a combination of solid and broken lines. In such a case, a solid line may be crossed with discretion, if the broken line of the combination is nearer to the direction of travel. Vehicles from the opposite directions are not permitted to cross the line. Different types of longitudianl markings are centre line, traffic lanes, no passing zone, warning lines, border or edge lines, bus lane markings, cycle lane markings.

8.3.1 Centre line

Centre line separates the opposing streams of traffic and facilitates their movements. Usually no centre line is provided for roads having width less than 5 m and for roads having more than four lanes. The centre line may be marked with either single solid line, single broken line, double broken line or double solid line depending upon the road and traffic requirements. On urban roads with less than four lanes, the centre line may be single broken line 150 mm wide of 3 m segments and 4.5 m gaps. This is shownin figure 8.1

On curves and near intesections, gap shall be reduced to 3 metres. On undivided urban roads with atleast two traffic lanes in each direction, the centre line marking may be a single solid line 150 mm wide as in figure 8.2, or double solid line 100 mm wide separated by a space of 100 mm as in figure 8.3.

The centre barrier line marking for four lane road is shown in figur 8.4.

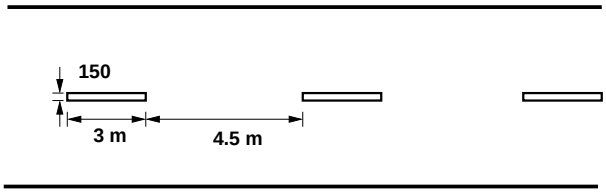


Figure 8.1: Centre line marking for a two lane road

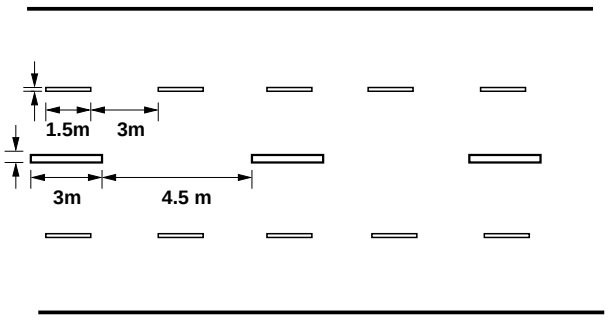


Figure 8.2: Centre line and lane marking for a four lane road

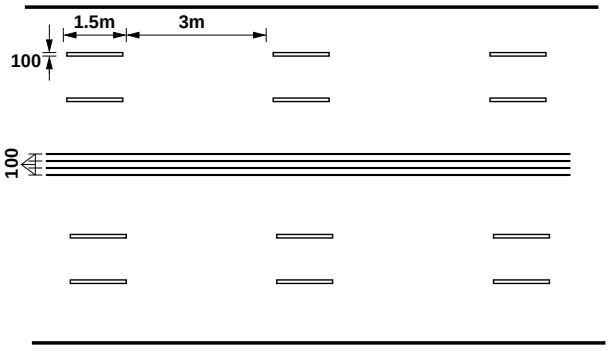


Figure 8.3: Double solid line for a two lane road

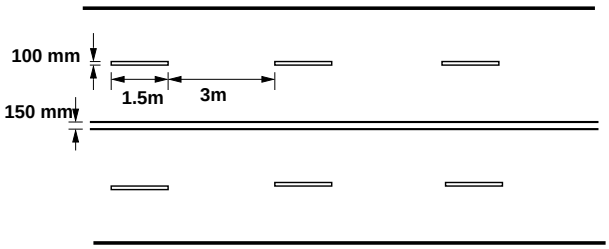


Figure 8.4: Centre barrier line marking for four lane road

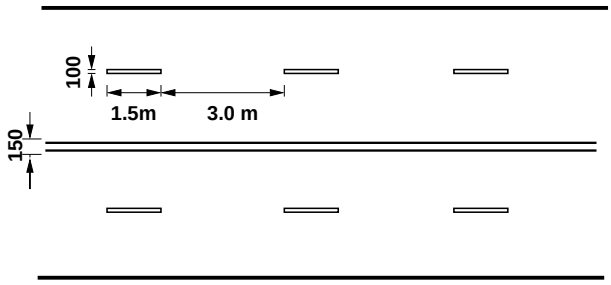


Figure 8.5: Lane marking for a four lane road

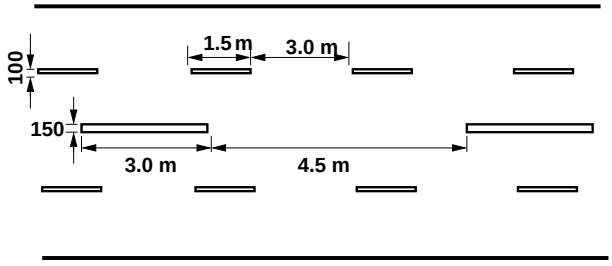


Figure 8.6: Traffic lane line for a four lane road

8.3.2 Traffic lane lines

The subdivision of wide carriageways into separate lanes on either side of the carriage way helps the driver to go straight and also curbs the meandering tendency of the driver. At intersections etc, these traffic lane lines will eleminate confusion and facilitates turning movements. Thus they help in incresing the capacity of the road. Also they ensure more safety. The traffic lane lines are normally single broken lines of 100 mm width. Some examples are shown in figure 8.5 and figure 8.6.

8.3.3 No passing zones

No passing zones are established on summit curves, horizontal curves, and on two lane and three lane highways where overtaking maneuvers are prohibited because of low sightdistance. It may be marked by a solid yellow line along the centre or a double yellow line. In the case of a double yellow line, the left hand element may be a solid barrier line, the right hand may be a either a broken line or a solid line . These solid lines are also called barrier lines. When a solid line is to the right of the broken line, the passing restriction shall apply only to the

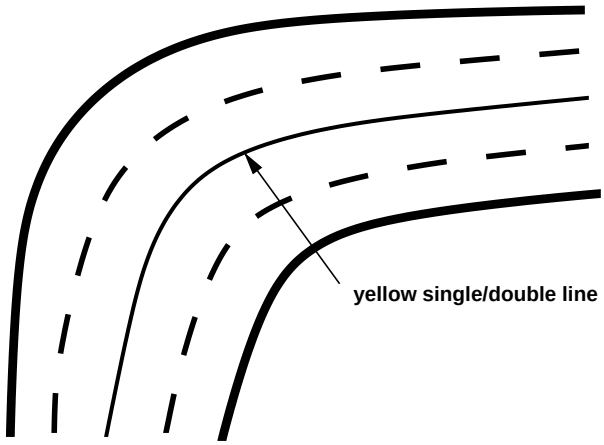


Figure 8.7: Barrier line marking for a four lane road

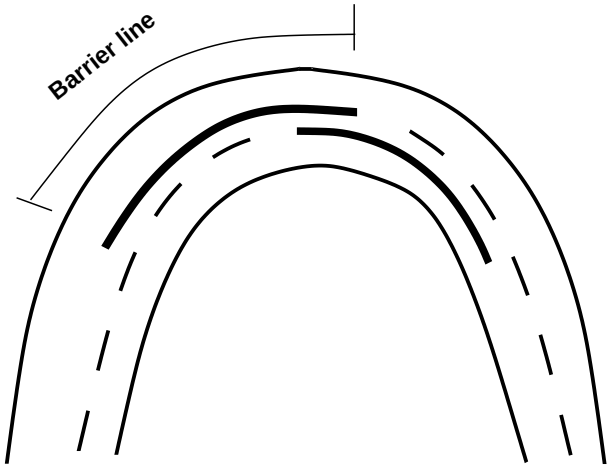


Figure 8.8: No passing zone marking at horizontal curves

opposing traffic. Some typical examples are shown in figure 8.7 and figure 8.8.

8.3.4 Warning lines

They are broken lines with line segments as long as the normal gaps and vice-versa. They warn the drivers about the obstruction approaches. They are marked on horizontal and vertical curves where the visibility is greater than prohibitory criteria specified for no overtaking zones. A minimum of seven line segements should be there. An example is shown in figure 8.9

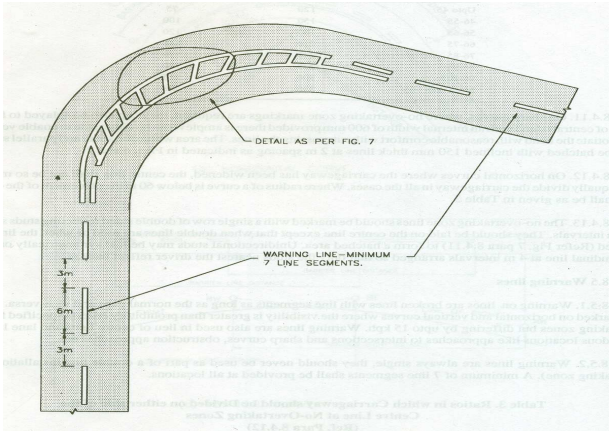


Figure 8.9: Warning line marking for a two lane road

8.3.5 Edge lines

These indicate edges of rural roads which have no kerbs to delineate the limits upto which the driver can safely venture. They should be atleast 150 mm from the actual edge of the pavement. They are painted in yellow or white.

All the lines should be light reflective, so that they will be visible during night also. Improved night visibility may also be obtained by the use of minute glass beads embedded in the pavement marking materials to produce a retroreflective surface.

8.4 Transverse markings

Transverse markings are marked across the direction of traffic. They are marked at intersections etc. The site conditions play a very important role. The type of road marking for a particular intersection depends on several variables such as speed characteristics of traffic, availability of space etc. Stop line markings, markings for pedestrian crossing, direction arrows, etc. are some of the markings on approaches to intersections.

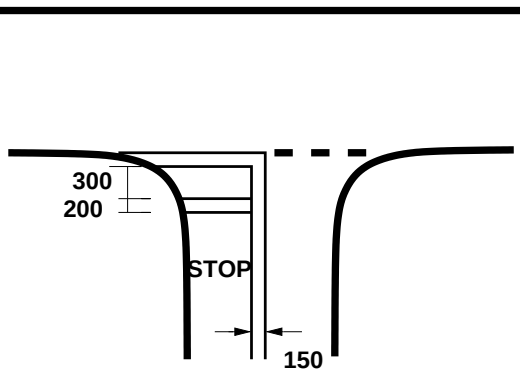


Figure 8.10: Stop line marking near an intersection

8.4.1 Stop line

Stop line indicates the position beyond which the vehicles should not proceed when required to stop by control devices like signals or by traffic police. They should be placed either parallel to the intersecting roadway or at right angles to the direction of approaching vehicles. An example for a stop line marking is shown in figure 8.10.

8.4.2 Pedestrian crossings

Pedestraian crossings are provided at places where the conflict between vehicular and pedestrian traffic is severe. The site should be selected that there is less inconvenience to teh pedestrians and also the vehicles are not interrupted too much. At intersections, the pedestrain crossings should be preceeded by a stop line at a distance of 2 to 3m for unsignalized intersections and at a distance of 1m for signalised intersections. Most commonly used pattern for pedestrian crossing is Zebra crossing consisting of equally spaced white strips of 500 mmm wide. A typical example of an intersection illustrating pedestrian crossings is shown in figure 8.11.

8.4.3 Directional arrows

In addition to the warning lines on approaching lanes, directional arrows should be used to guide the drivers in advance over the correct lane to be taken while approaching busy intersections. Because of the low angle at which the markings are viewed by the drivers, the arrows should be very much elongated in the direction of

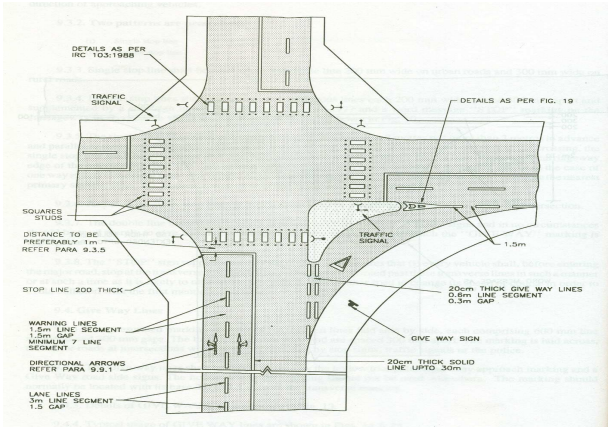


Figure 8.11: Pedestrian marking near an intersection

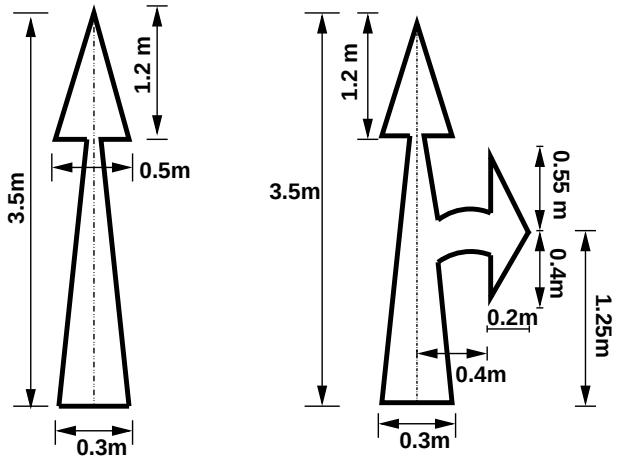


Figure 8.12: Directional arrow marking

traffic for adequate visibility. The dimensions of these arrows are also very important. A typical example of a directional arrow is shown in figure 8.12.

8.5 Object marking

Physical obstructions in a carriageway like traffic island or obstructions near carriageway like signal posts, pier etc. cause serious hazard to the flow of traffic and should be adequately marked. They may be marked on the objects adjacent to the carriageway.

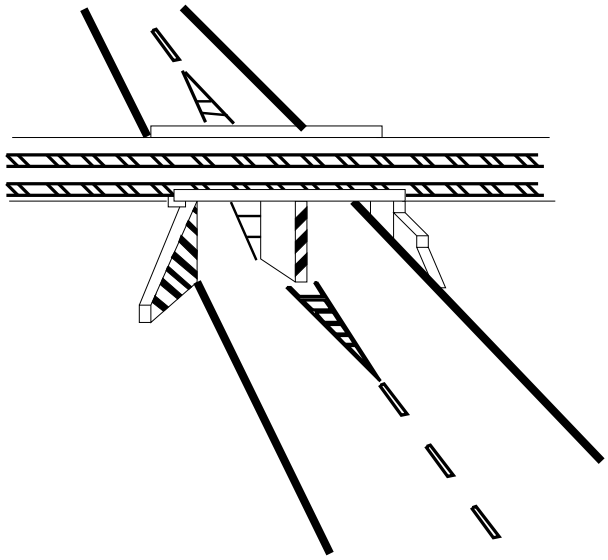


Figure 8.13: Marking for objects adjacent to the road way

8.5.1 Objects within the carriageway

The obstructions within the carriageway be marked by not less than five alternate black and yellow stripes. The stripes should slope forward at an angle of 45° towards the ide of the obstruction on which traffic passes. These stripes shall be uniform and not less than 100 m wide to provide sufficient visibility.

8.5.2 Objects adjacent to carriageway

Sometimes objects adjacent to the carriageway may pose some obstructions to the flow of traffic. Objects such as subway piers and abutments, culvert head walls etc. are some examples for such obstructions. They should be marked with alternate black and white stripes at an angle of 45 ° towards the side of obstruction on which the traffic passes. Poles close to the carriageway should be painted in alternate balck and white upto a height of 1.25 m above the road level. Other objects such as guard stones, drums, guard rails etc. where chances of hitting vehicles are only when the vehicle runs off the carriageway should be painted in solid white. Kerbs of all islands located in the line of traffic flow shall be painted with either alternating black and white stripes 500 m wide or chequered black and white stripes of same width. They are illustrated in figure 8.13.

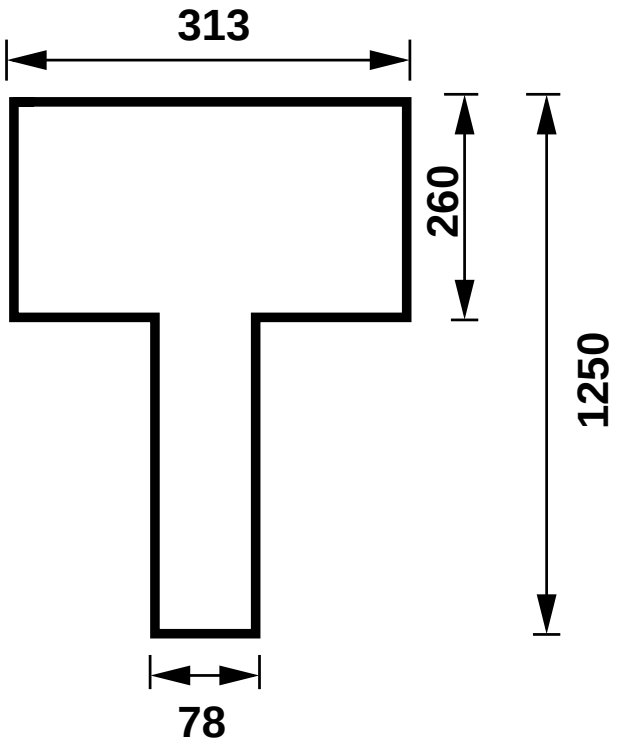


Figure 8.14: Dimension of legends used in road marking

8.6 Word messages

Information to guide, regulate or warn the road user may also be conveyed by inscription of word message on road surface. The basic characters for word messages are the capitals. The legends should be as brief as possible and shall not consist of more than three words for any message. Some of the examples for word messages are STOP, SLOW, SCHOOL, RIGHT TUN ONLY etc. The dimensioning of a typical alphabet is shown in figure 8.14.

8.7 Parking

The marking of the parking space limits on urban roads promotes more efficient use of the parking spaces and tends to prevent enroachment on places like bus stops, fire hydrant zones etc. where parking is undesirable. Such parking space limitations should be indicated with markings that are solid white lines 100 mm wide.

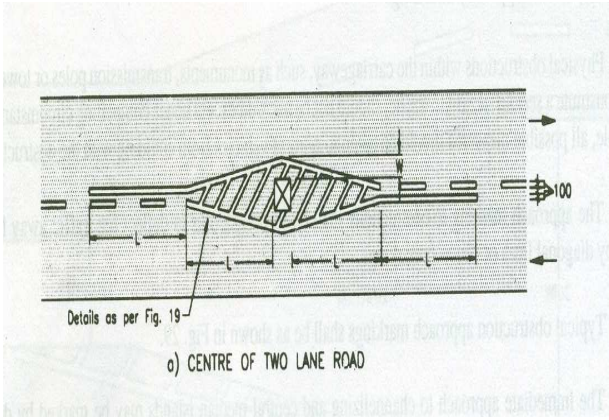


Figure 8.15: Approach marking for obstructions on the road way

Words TAXI, CARS,SCOOTERS etc. may also be written if the parking area is specific for any particular type of vehicle. To indicate parking restriction, kerb or carriage way marking of continuous yellow line 100 mm wide covering the top of kerb or carriageway close to it may be used.

8.8 Hazardous location

Wherever there is a change i the width of the road, the driver should be warned about this situation with the help of suitable road markings. Line markings at carriage way width transition shall be 100 mm wide and of standard centre line or lane line design. Converging lines shall be 150 mm wide and shall have a taper length of not less than twenty times the off-set distance. Typical carriageway markings showing transition from wider to narrower sections and vice-versa is shown in figure 8.15.

8.9 Summary

8.10 Problems

Chapter 9

Parking

9.1 Overview

Parking is one of the major problems that is created by the increasing road traffic. It is an impact of transport development. The availability of less space in urban areas has increased the demand for parking space especially in areas like Central business district. This affects the mode choice also. This has a great economical impact.

9.2 Parking studies

Before taking any measures for the betterment of conditions, data regarding availablity of parking space, extent of its usage and parking demand is essential. It is also required to estimate the parking fares also. Parking surveys are intended to provide all these information.

9.2.1 Parking statistics

Parking accumulation: It is defined as the number of vehicles parked at a given instant of time. Normally this is expressed by accumulation curve. Accumulation curve is the graph obtained by plotting the number of bays occupied with respect to time.

Parking volume: Parking volume is the total number of vehicles parked at a given duration of time.

Parking load : Parking load gives tthe area under the accumulation curve. It can also be obtained by simply multiplying the number of vehicles with the time interval. It is expressed as vehicle hours.

Average parking duration: It is the ratio of total vehicle hours to the number of vehicles parked.

Parking turnover: It is the ratio of number of vehicles parked in a duration to the number of parking bays available.

Parking index: Parking index is also called occupancy or efficieny. It is defined as the ratio of number of bays occupied in a time duration to the total space available. It gives an aggregate measure of how effectively the parking space is utilized. Parking index can be found out as follows:

$$parking\ index = \frac{parking\ load}{parking\ capacity} \times 100 \tag{9.1}$$

9.3 Parking surveys

There are three major types of parking surveys. They are

1. **In-out survey**: In this survey, the occupancy count in the selected parking lot is taken at the beginning. Then the number of vehicles that enter the parking lot for a particular time interval is counted. The number of vehicles that leave the parking lot is also taken. The final occupancy in the parking lot is also taken. Here the labour required is very less. Only one person may be enough. But we wont get any data regarding the time duration for which a particular vehicle used that parking lot. Parking duration and turn over is not obtained. Hence we cannot estimate the parking fare from this survey.

2. **Fixed period sampling**: This is almost similar to in-out survey. All vehicles are counted at the beginning of the survey. Then after a fixed time interval that may vary between 15 minutes to i hour, the count is again taken. Here there are chances of missing the number of vehicles that were parked for a short duration.

3. **License plate method of survey**: This results in the most accurate and realistic data. In this case of survey, every parking stall is monitored at a continuous interval of 15 minutes or so and the license plate number is noted down. This will give the data regarding the duration for which a particular vehicle was using the parking bay. This will help in calculating the fare because fare is estimated based on the duration for which the vehicle was parked. If the time intervalis shorter, then there are less chances of missing short-term parkers. But this method is very labour intensive.

9.4 Ill effects of parking

Parking has got some ill-effects like congestion, accidents, obstruction to fire-fighting operations etc.

Congestion: By parking, there will be loss in the street space which leads to the lowering of the road capacity. Hence speed will be reduced, journey time and delay will also subsequently increase. The operational cost of the vehicle increases leading to great economical loss to the community.

Accidents: Careless maneuvering of parking and unparking leads to accidents which are referred to as parking accidents. Common type of parking accidents occur while driving out a car from the parking area, careless opening of the doors of parked cars, and while bringing in the vehicle to the parking lot for parking.

Obstruction to fire fighting operations: Parked vehicles may obstruct the movement of firefighting vehciles. Sometimes they block access to hydrants and acess to buildings.

Environmental pollution: They also cause pollution to the environment because stopping and starting of vehicles while parking and unparking results in noise and fumes. They also affect the aesthetic beauty of the buidings because cars parked at every available space creates a feeling that buliding rises from a plinth of cars.

9.5 Parking requirements

There are some minimum parking requirements for different types of building. For residentail plot area from 101 to 300 sq.m only community parking sapce is required. For residential plot area from 500 to 1000 sq.m, minimum one-fourth of the open area should be reserved for parking. For offices, atleast one space for every

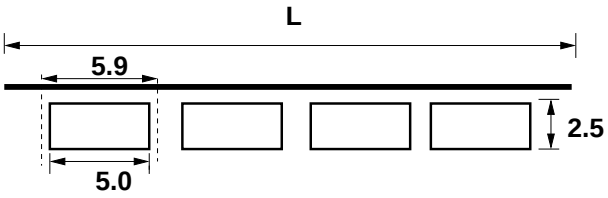


Figure 9.1: Illustration of parallel parking

70 sq.m should be kept aside for parking facility of vehicles. One parking space is enough for 10 seats in a restaurant where as theatres and cinema halls need keep only 1 parking space for 20 seats. Thus the parking requirements are different for different land use zones.

9.6 On street parking

On street parking means the vehicles are parked on the sides of the street itself. This will be usually controlled by government agencies itself. Common types of on-street parking are as listed below. This classification is based on the angle in which the vehicles are parked with respect to the road alignment. As per IRC the standard dimensions of a car is taken as 5× 2.5 and that for a truck is 3.75× 7.5 .

1. Parallel parking: The vehicles are parked along the length of the road. Here there is no backward movement involved while parking or unparking the vehicle. Hence it is the most safest parking from the accident perspective. But it consumes the maximum curb length and therefore only a minimum number of vehicles can be parked for a given kerb length. Since it consumes least width of the road it produces least obstruction to the on-going traffic on the road. Parallel parking of cars is shown in figure 9.1. The number of vehicles that can be parked , N = $\frac{L}{5.9}$

2. 30° parking: Here more vehicles can be parked compared to parallel parking. Also there is better maneuverablity. It causes minimum delay to traffic. An example is shown in figure 9.2. From the figure, it is clear that the number of vehicles that can be parked, N = $\frac{L-1.25}{5}$

3. 45° parking: As the angle of parking increases, more number of vehicles can be parked. Hence compared to parallel and thirty degree parking, more number of vehicles can ne accommodated in this type of parking.

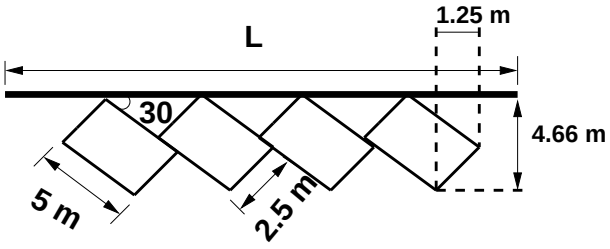


Figure 9.2: Illustration of 30° parking

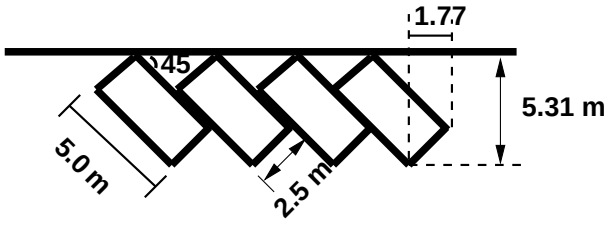


Figure 9.3: Illustration of 45° parking

From figure 9.3, number of vehicles that can be parked ,N = $\frac{L-1.77}{3.54}$

4. 60° parking: Here also more number of vehicles can be accomodated. From the figure 9.4, number of vehicles that can be parked, N = $\frac{L-2.16}{2.89}$

5. Right angle parking: Here the vehicles are parked perpendicular to the direction of the road. Hence it consumes maximum width. Curb length required is very little. Hence there are chances of severe accidents. Also it causes obstruction to the road traffic. But it can accommodate maximum number of vehicles. An

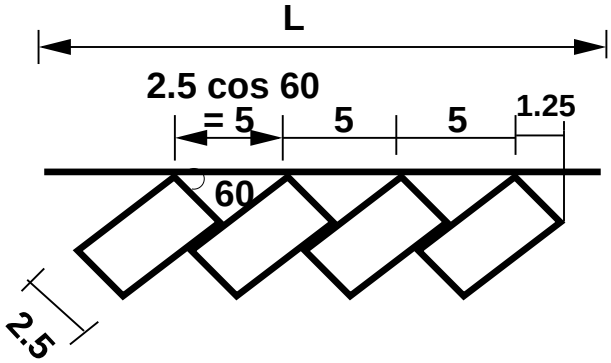


Figure 9.4: Illustration of 60° parking

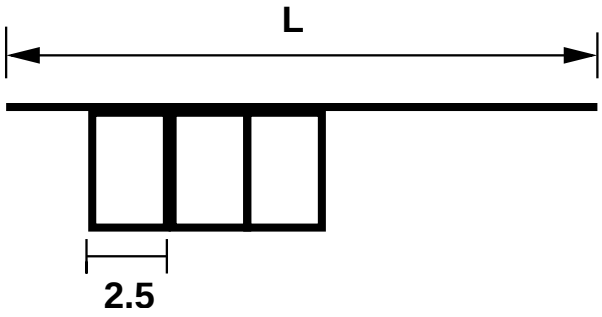


Figure 9.5: Illustration of 90° parking

example is shown in figure 9.5.

9.7 Off street parking

There will be some area exclusively allotted for parking which will be at some distance away from the main stream of traffic. Such a parking is referred to as off-street parking. They may be operated by either public agencies or private firms. A typical example of an off-street parking is shown in figure **??**.

figureeeeeeeeeeeeeeeeee

Example 1

From an In-out survey conducted for a parking area consisting of 40 bays, the initial count was found to be 25. The number of vehicles coming in and out of the parking lot for a time interval of 5 minutes is as shown in the table. Find the accumulation, total parking load, average occupancy and efficiency of the parking lot.

Time	In	Out
5	3	2
10	2	4
15	4	2
20	5	4
25	7	3
30	8	2
35	2	7
40	4	2
45	6	4
50	4	1
55	3	3
60	2	5

Solution

Time (1)	In (2)	Out (3)	Accumulation (4)	Occupancy (5)	Parking load (6)
5	3	2	26	65	130
10	2	4	24	60	120
15	4	2	26	65	130
20	5	4	27	67.5	135
25	7	3	31	77.5	155
30	8	2	37	92.5	185
35	2	7	32	80	160
40	4	2	34	85	170
45	6	4	36	90	180
50	4	1	39	97.5	195
55	3	3	39	97.5	195
60	2	5	36	90	180

- Accumulation can be found out as initial count plus number of vehicles that entered the parking lot till that time minus the number of vehicles that just exited for that particular time interval. For the first time interval of 5 minutes, accumulation can be found out as 25+3-2 = 26. It is being tabulated in column 4.
- Occupancy is given by equation 9.1. For the first time interval of five minutes, $Parking\ index = \frac{26}{40} \times 100$ = 65%. The occupancy for the remaining time slot is similarly calculated and is tabulated in column 5.

  Average occupancy is the average of the occupancy values for each time interval. Thus it is the average of all values given in column 5 and the value is 80.63%.
- Parking load is tabulated in column 6. It is obtained by multiplying accumulation with the time interval. For the first time interval, Parking load = 26 × 5 = 130 vehicle minutes.
- Total parking load is the summation of all the values in column 5 which is equal to 1935 vehicle minutes or 32.25 vehicle hours

Example 2

The parking survey data collected from a parking lot by license plate method is s shown in the table below. Find the average occupancy, average turn over, parking load, parking capacity and efficiencyof the parking lot.

Bay	Time			
	0-15	15-30	30-45	45-60
1	1456	9813	-	5678
2	1945	1945	1945	1945
3	3473	5463	5463	5463
4	3741	3741	9758	4825
5	1884	1884	-	7594
6	-	7357	-	7893
7	-	4895	4895	4895
8	8932	8932	8932	-
9	7653	7653	8998	4821
10	7321	-	2789	2789
11	1213	1213	3212	4778
12	5678	6678	7778	8888

Solution

1	2	3	4	5	6	7	8	9	10
Bay	Time				Time				
	15	30	45	60	15	30	45	60	Turn over
1	1456	9813	-	5678	1	1	0	1	3
2	1945	1945	1945	1945	1	1	1	1	1
3	3473	5463	5463	5463	1	1	1	1	2
4	3741	3741	9758	4825	1	1	1	1	3
5	1884	1884	-	7594	1	1	0	1	2
6	-	7357	-	7893	0	1	0	1	2
7	-	4895	4895	4895	0	1	1	1	1
8	8932	8932	8932	-	1	1	1	0	1
9	7653	7653	8998	4821	1	1	1	1	3
10	7321	-	2789	2789	1	0	1	1	2
11	1213	1213	3212	4778	1	1	1	1	3
12	5678	6678	7778	8888	1	1	1	1	4
	Accumulation				10	11	9	11	
	Occupancy				0.83	0.92	0.75	0.92	2.25

Solution Code the given data. If a vehicle occupies that bay for that time interval, then it has a code 1. This is shown in columns 6, 7, 8 and 9 of the tables corresponding to the time intervals 15,30,45 and 60 seconds. Turn over is computed as the nunber of vehicles present in that bay for that particular hour. For the first time interval of 15 minutes, it is calculated as 1+1+0+1 = 3. This is being tabulated in column 10 of the table.

Parking volume = Sum of the turn over in all the bays = 27 veh/sec

Average duration is the average time for which the parking lot was used by the vehicles It can be calculated as sum of the accumulation for each time interval × time interval divided by the parking volume $=\frac{(10+11+9+11)\times 15}{27} = 22.78$

Accumulation for a time interval is the total of number of vehicles in the bays 1 to 12 for that time interval. Accumulation for first time interval of 15 minutes = 1+1+1+1+1+0+0+1+1+1+1+1 = 10

Occupancy for that time interval is accumulation in that particular interval divided by total number of bays. For first time interval of 15 minutes, occupancy = (10×100)/12 = 83%

Average occupancy is found out as the average of total number of vehicles occupying the bay for each time interval. It is expressed in percentage. Averge occupancy = $\frac{0.83+0.92+0.75+0.92}{4} \times 100 = 85.42\%$.

Average turn over = $\frac{\text{Sum of turn}-\text{over}}{\text{Total number of bays}} = 2.25$

Parking capacity = number of bays × number of hours = 12× 1 = 12 vehicle hours

Parking load = total number of vehicles accumulated at the end of each time interval × time = $\frac{(10+11+9+11)\times 60}{15}$ = 10.25 vehicle hours

Efficiency = $\frac{\text{Parking load}}{\text{Total number of bays}} = \frac{10.25}{12} = 85.42\%$.

9.8 Summary

9.9 Problems

Chapter 10

Traffic intersections

10.1 Overview

It is the most complex location on any highway. Conflicts are common at the intersections. This is because vehicles moving in different direction wan tto occupy same space at the same time. In addition, the pedestrians also seek same space for crossing. Intersection is an area shared by two or more roads. It is some area designated for the vehicles to turn to different directions to reach their desired destinations. Its main function is to provide channelization of route direction. Drivers have to make split second decision at an intersection by considering his route. intersection geomery, other vehicles, their speed, direction etc. A small error in judjement can cause severe accidents. It also causes delay and it depends on type, geometry and type of control. Overall traffic flow depends on the performance of intersection. It also affects the capacity of the road. Especially in the case of an urban scenario, both from the accident perspective and the capacity perspective, the study of intersectionis very important for the traffic engineers.

10.2 Conflicts at an intersection

Conflicts in an intersection are different for different types of intersection. Consider a typical four-legged intersection as shown in figure. The conflicts for competing through movements are 4, those between right

turn and competing through movements are 8. The conflicts between right turn traffics are 4, and between left turn and merging traffic is 4. The conflicts created by pedestrians come to 8 taking into account all the four approaches. Diverging traffic also produces about4 conflicts. Summing up all the conflicts in the intersection, it comes to 32.

The essence of the intersection control is to resolve these conflicts at the intersection for the safe and efficient movement of both vehicular traffic and pedestrians. Two methods of intersection controls are there. They are the time sharing approach and the space sharing approach. The type of intersection control that has to be adopted depends on the traffic volume, road geometry, cost involved, importance of the road etc.

10.3 Levels of intersection control

The control of an intersection can be exercised at different levels. They can be either passive control, semi control, or active control. In passive control, there is no such direct strict rules on the driver . In semi control, some amount of control on the driver is there from the traffic agency. Active control means the movement of the traffic is fully controlled by the traffic agency and the drivers cannot simply maneuver the intersection according to his choice.

10.3.1 Passive control

Some of the intersection control that are classified under passive control are as follows:

1. No control: If the traffic coming to an intersection is low, then by applying the basic rules of the road like driver on the left side of the road must yield and that through movements will have priority than turning movements, the driver itself can manage to traverse the intersection.

2. Traffic signs: With the help of warning signs, guide signs etc. it is able to provide some level of control at an intersection.

3. Traffic signs plus marking: In addition to the traffic signs, road markings also complement the traffic control at intersections. Some of the examples are stop line marking, yield lines, arrow marking etc.

4. GIVE WAY control: This control requires the driver in the minor road to slow down to a minimum speed and allow the vehicle on the major road to proceed.

5. Two way stop control: In this case, the vehicle drivers on the minor streets should see that the conflicts are avoioded.

6. All-way stop control: This is usually used when it is difficult to differentiate between the major and minor roads in an intersection. In such a case, STOP sign is placed on all the approaches to the intersection and the driver on all the approaches are required to stop the vehicle. The vehicle at the right side will get priority over the left approach.

10.3.2 Semi control

Channelization and traffic rotaries come under semi control.

1. Channelization: The traffic is separated to flow through defenite paths by raising a portion of the road in the middle usually called as islands distinguished by road markings. The conflicts in traffic movements are reduced to a great extent in such a case.

2. Traffic rotaries: It is a form of intersection control in which the traffic is made to flow along one direction round a traffic island. This completely avoids through-conflicts.

10.3.3 Active control

Active control means the road user will be forced to follow the path suggested by the traffic control agencies. He cannot maneuver according to his wish. Traffic siganls and grade separated intersections come under this classification.

1. Traffic signals: This control is based on time sharing approach. At a time with the help of appropriate signals, certain traffic movements are restricted where as certain other movements are permitted. Two or more phase signals may be provided depending upon the traffic conditions of the intersection.

2. Grade separated intersections: It is an intersection where crossing movements at different levels is permitted. It is very expensive and is usually used on high speed facilities like expressways, freeways etc.

10.4 Types of intersection control

The intersections are of two types. They are at-grade intersections and grade-separated intersections. In at-grade intersections, all roadways join or cross at the same vertical level. Grade separated intersections allows the traffic to cross at different vertical levels. Sometimes the topography itself may be helpful in constructing such intersections. Otherwise the initial construction cost required will be very high. But it increase the road capacity because vehicles can flow with high speed and accident potential is also reduced by such vertical separtion of traffic.

10.4.1 At grade intersection control

: Different types of intersection control provided at 'at-grade' are as follows.

1. Uncontrolled or passive control: The traffic control at 'at-grade' intersection may be uncontrolled in cases of low traffic. Here the road users are required to obey the basic rules of the road. Passive control like traffic signs, road markings etc. are used to complement the intersection control.

2. Semi controlled or channelized intersections: In channalized intersections, as the name suggests, the traffic is directed to flow through different channels and this physical separation is made possible with the help of some barriers in the road like traffic islands, road markings etc.

3. Rotary/Round about: It is a form of 'at-grade' intersection laid out for the movement of traffic such that no through conflicts are there. Free-left turn is permitted where as through traffic and right-turn traffic is forced to move around the central island in a clock-wise direction in an orderly manner. Merging, weaving and diverging operations reduces the conflicting movements at the rotary.

4. Signalized control: When the vehicles traversing the intersection is very large, then the control is done with the help of signals. The phases provided for the signal may be either 2 or more than 2 . If more than

two phases are provided, then it is called multiphase signal.

The signals can operate in several modes. Most common are fixed time signals and vehicle actuated signals. In fixed time signals, the cycle time, phases and interval of each signal is fixed. Each cycle of the signal will be exactly like another. But they cannot cater to the needs of the fluctuating traffic. i vehicle actuated signals, vehicle detectors will be placed on the streets approaching the intersection, and when the vehicle arrives, the detector will detect the vehicle and pass the information to the controller. The controller then sets the cycle time and adjusts the phase lengths according to the prevailing traffic requirements.

10.4.2 Grade separated intersection control

As we discussed earlier, grade-separated intesections are provided to separate the traffic in the vertical grade. But the traffic need not be those pertaining to road only. When a railway line crosses a road, then also grade separators are used. Different types of grade-separators are flyovers and interchange. Flyovers itself are subdivided into overpass and underpass. When two roads cross at a point, if the road having major traffic is elevated to a higher grade for further movement of traffic, then such structures are called overpass. Otherwise, if the major road is depressed to a lower level to cross another by means of an under bridge or tunnel, it is called under-pass.

10.4.3 Interchange

Interchange is a system where traffic between two or more roadways flows at different levelsin teh grade separated junctions. Different types are

1. Diamond interchange: It is the most popular form of four-leg interchange found in the urban locations where major and minor roads crosses. The important feature of this interchange is that it can be designed even if the major road is relatively narrow.

2. Clover leaf interchange: It is also a four leg interchange and is used when two highways of high volume

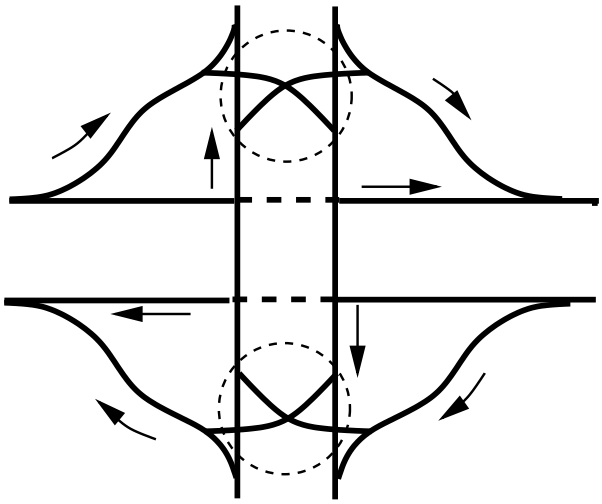


Figure 10.1: Diamond interchange

and speed intersect each other. The main advantage od cloverleaf intersection is that it provides complete separation of traffic. Also high speed at intersections can be achieved. But the disadvantage is that large area of land is required.

3. Trumpet interchange: It is a three leg interchange. If one of the legs of the interchange meets a highway at some angle but does not croos it, then the interchange is called trumpet interchange.

10.5 Channalized intersection

The vehicles approaching the intersection are directed to definite paths by islands, marking etc. and this process is called channelization. There can be channalized or unchannalized intersection but channalized intersection provides more safety and efficiency. It reduces the number of possible conflicts by reducing the area of conflicts available in the carriageway. If no channalizing is provided, the driver will have less tendency to reduce the speed while entering the intersection from the carriageway. The presence of traffic islands, markings etc. makes the driver to reduce the speed and becomes more cautious while maneuvering the intersection. A channalizing island serves as a refuge for pedestrians and makes pedestrian crossing safer. Channalization of traffic through a three-legged intersection and a four-legged intersection is shown in the figure.

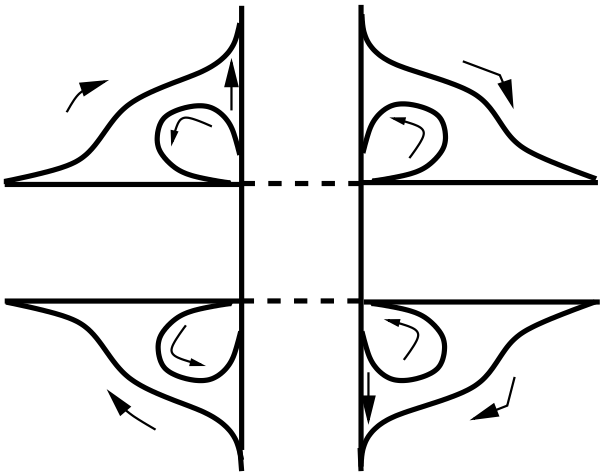


Figure 10.2: Cloverleaf interchange

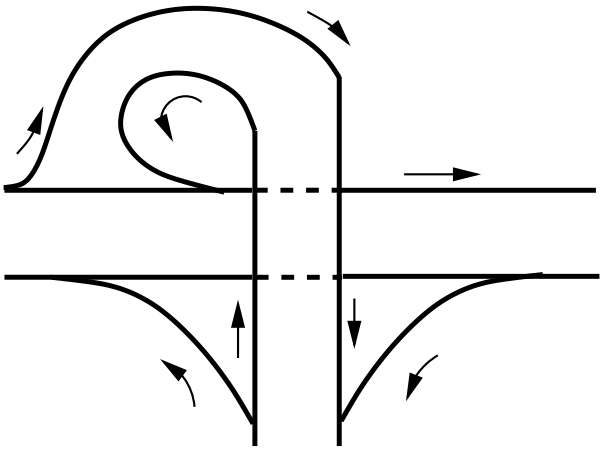


Figure 10.3: Trumpet interchange

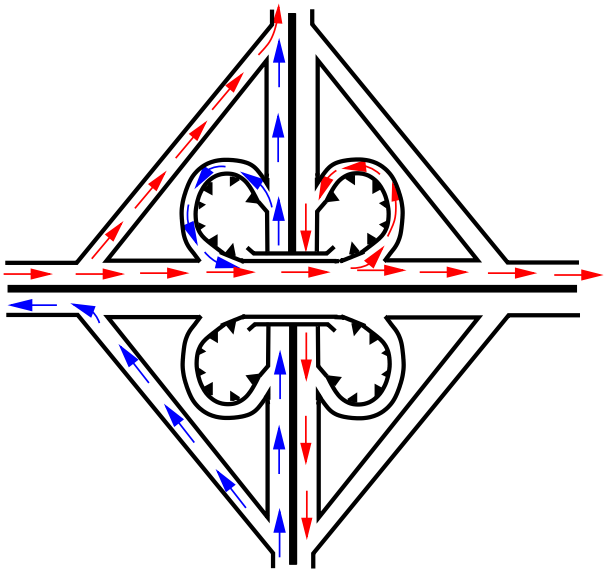


Figure 10.4: Cloverleaf interchange

10.6 Clover-leaf intersection

It is also a four leg interchange and is used when two highways of high volume and speed intersect each other. The main advantage od cloverleaf intersection is that it provides complete separation of traffic. Also high speed at intersections can be achieved. But the disadvantage is that large area of land is required.

10.7 Summary

10.8 Problems

Chapter 11

Traffic rotaries

11.1 Overview

Rotary intersections or round abouts are special form of at-grade intersections laid out for the movement of traffic in one direction round a central traffic island. Essentially all the major conflicts at an intersection namely the collision between through and right-turn movements are converted into milder conflicts namely merging and diverging. The vehicles entering the rotary are gently forced to move in a clockwise direction in orderly fashion. They then weave out of the rotary to the desired direction. The benefits, design principles, capacity of rotary etc. will be discussed in this chapter.

11.2 Advantages and disadvantages of rotary

The key advantages of the rotary intersection are listed below:

1. Traffic flow is regulated to only one direction of movement, thus eliminating severe conflicts between crossing movements.

2. All the vehicles entering the rotary are gently forced to reduce the speed and continue to move at slower speed. Thus, more of the vehicles need to be stopped.

3. Because of lower speed of negotiation and elimination of severe conflicts, accidents and their severity are much less in rotaries.

4. Rotaries are self governing and do not need practically any control by police or traffic signals.

5. They are ideally suited for moderate traffic, especially with irregular geometry, or intersections with more than three or four approaches.

Although rotaries offer some distinct advantages, there are few specific limitations for rotaries which are listed below.

1. All the vehicles are forced to slow down and negotiate the intersection.Therefore the cumulative delay will be much higher than channelized intersection.

2. Even when there is relatively low traffic, the vehicles are forced to reduce their speed.

3. Rotaries require large area of relatively flat land making them costly at urban areas.

4. Since the vehicles are not stopping, and the vehicles accelerate at rotary exits, they are not suitable when there is high pedestrian movements.

11.3 Guidelines for the selection of rotaries

Rotaries are not suitable for every location. There are few guidelines that help in deciding the suitability of a rotary. They are:

1. Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.

2. A total volume of about 3000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit.

3. A rotary is very beneficial when the proportion of the right-turn traffic is very high; typically it is suitable for more than 30 percent.

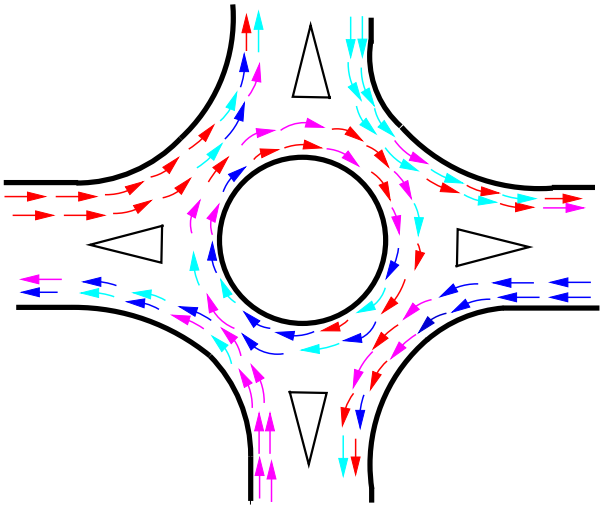


Figure 11.1: Traffic operations in a rotary

4. Rotaries are suitable when there are more than four approaches, or if there is no suitable lanes available for right-turn traffic.

11.4 Traffic operations in a rotary

As noted earlier, the traffic operations at a rotary are three; diverging, merging and weaving. All the other conflicts are converted into these three less severe conflicts.

1. Diverging: It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.

2. Merging: Merging is the opposite of diverging, when traffic streams coming from various places and going to a common destination are joined together into a single stream it is referred to as merging.

3. Weaving: Weaving is the combined movement of both the merging and diverging movements in the same direction.

These movements are shown in figure 11.1.

It can be observed that movements from each direction split into three; left, straight, and right turn. The major design aspect of a rotary is the design of the length and width of the weaving section.

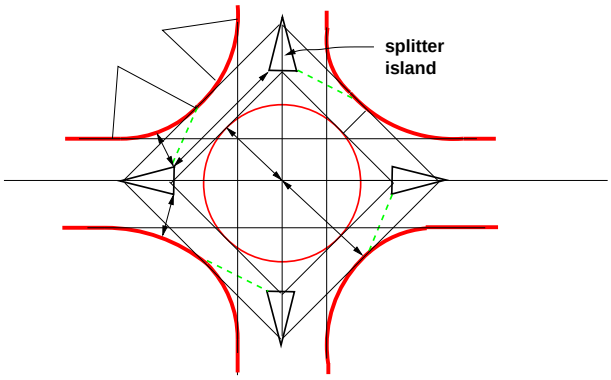


Figure 11.2: Design of a rotary

11.4.1 Design elements

The design elements include design speed, radius at entry, exit and the central island, weaving length and width, entry and exit widths. In addition the capacity of the rotary can also be determined by using some empirical formulae. A typical intersection is shown in figure 11.2

11.4.2 Design speed

All the vehicles are required to reduce their speed at a rotary. Therefore, the design speed of a rotary will be much lower than the roads leading to it. Although it is possible to design roundabout without much speed reduction, the geometry may lead to large size incurring huge cost of construction. The normal practice is to keep the design speed as 30 and 40 kmph for urban and rural areas respectively.

11.4.3 Entry, exit and island radius

The radius at the entry depends on various factors like design speed, superelevation, and coefficient of friction. The entry to the rotary is not straight, but a small curvature is introduced. This will force the driver to reduce the speed. The speed range of about 20 kmph and 25 kmph is ideal for an urban and rural design respectively.

The exit radius should be higher than the entry radius and the radius of the rotary island so that the vehicles will discharge from the rotary at a higher rate. A general practice is to keep the exit radius as 1.5 to 2 times the entry radius. However, if pedestrian movement is higher at the exit approach, then the exit radius could be

set as same as that of the entry radius.

The radius of the central island is governed by the design speed, and the radius of the entry curve. The radius of the central island, in practice, is given a slightly higher reading so that the movement of the traffic already in the rotary will have priority of movement. The radius of the central island which is about 1.3 times that of the entry curve is adequate for all practical purposes.

11.4.4 Width of the rotary

The entry width and exit width of the rotary is governed by the traffic entering and leaving the intersection and the width of the approaching road. The width of the carriageway at entry and exit will be lower than the width of the carriageway at teh approaches to enable reduction of speed. IRC suggests that a two lane road of 7m width should be kept as 7m for urban roads and 6.5m for rural roads. Further, a three lane road of 10.5m is to be reduced to 7 and 7.5m respectively for urban and rural roads.

The width of the weaving section should be higher than the width at entry and exit. Normally this will be one lane more than the average entry and exit width. Thus weaving width is given as,

$$w_{weaving} = (\frac{e_1 + e_2}{2}) + 3.5m \tag{11.1}$$

where e_1 is the width of the carriageway at the entry and e_2 is the carriageway width at exit.

Weaving length determines how smoothly the traffic can merge nad diverge. It is decided based on many factors such as weaving width, proportion of weaving traffic to the non-weaving traffic etc. This can be best achieved by making the ratio of weaving length to the weaving width very high. A ratio of 4 is the minimum value suggested by IRC. Very large weaving length is also dangerous, as it may encourage over-speeding.

11.5 Capacity

The capacity of rotary is determined by the capacity of each weaving section. Transportation road research lab proposed the following empirical formula to find the capacity of the weaving section.

$$Q_w = \frac{280w[1 + \frac{e}{w}][1 - \frac{p}{3}]}{1 + \frac{w}{l}} \tag{11.2}$$

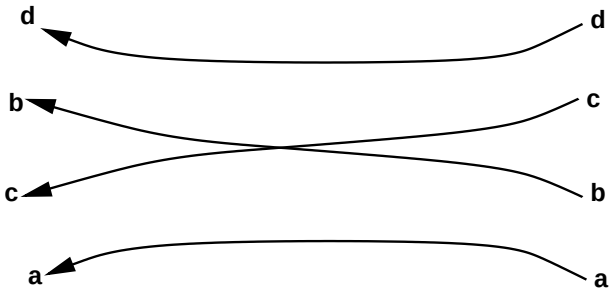


Figure 11.3: Weaving operation in a rotary

where e is the average entry and exit width, i.e, $\frac{(e_1+e_2)}{2}$, w is the weaving width, l is the length of weaving, and p is the proportion of weaving traffic to the non-weaving traffic. Figure 11.3 shows four types of movements at a weaving section, a and d are the non-weaving traffic and b and c are the weaving traffic. Therefore,

$$p = \frac{b+c}{a+b+c+d} \tag{11.3}$$

This formula is valid only if the following conditions are satisfied.

1. Weaving width at the rotary is in between 6 and 18 metres.
2. The ratio of average width of the carriage way at entry and exit to the weaving width is in the range of 0.4 to 1.
3. The ratio of weaving width to weaving length of the roundabout is in between 0.12 and 0.4.
4. The proportion of weaving traffic to non-weaving traffic in the rotary is in the range of 0.4 and 1.
5. The weaving length available at the intersection is in between 18 and 90.

Example

The width of a carriage way approaching an intersection is given as 15m. The entry and exit width at the rotary is 10m. The traffic approaching the intersection from the four sides is shown in the figure 11.4 below.

Find the capacity of the rotary using the given data. The traffic from the four approaches negotiating through the roundabout is illustrated in figure 11.5.

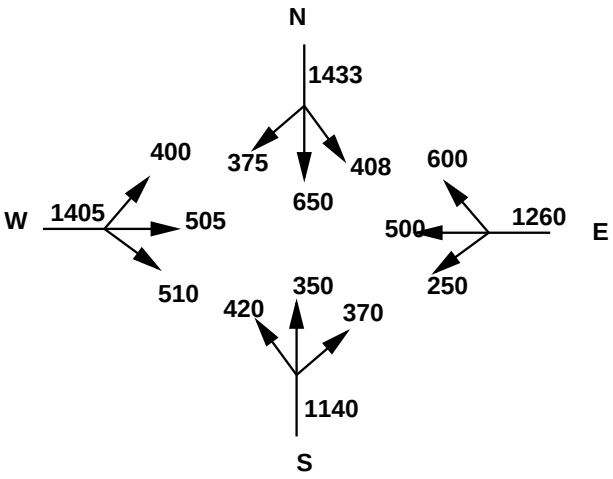


Figure 11.4: Traffic approaching the rotary

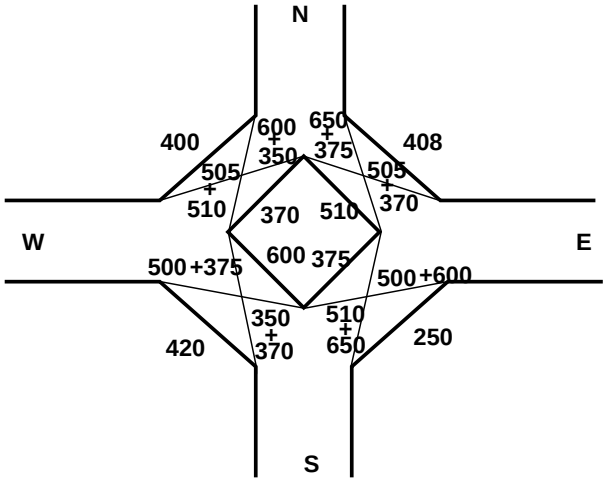


Figure 11.5: Traffic negotiating a rotary

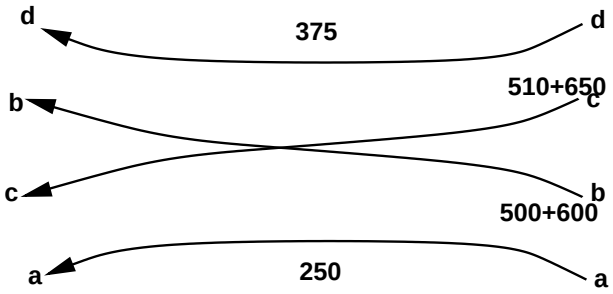


Figure 11.6: Traffic weaving in East-South direction

Weaving width is calculated as, w = $[\frac{e_1+e_2}{2}] + 3.5 = 13.5$m

Weaving length ,l = 4×w = 54m

The proportion of weaving traffic to the non-weaving traffic in all the four approaches is found out first. It is clear from equation 11.2 that the highest proportion of weaving traffic to non-weaving traffic will give the minimum capacity. Let the proportion of weaving traffic to the non-weaving traffic in West-North direction be denoted as p_{WN},in North-East direction as p_{NE}, in the East-South direction as p_{ES},and finally in the South-West direction as p_{SW}. Then,

$p_{WN} = \frac{505+510+350+600}{505+510+350+600+400+370} = \frac{1965}{2735} = 0.718$

$p_{NE} = \frac{650+375+505+370}{650+375+505+370+510+408} = \frac{1900}{2818} = 0.674$

$p_{ES} = \frac{510+650+500+600}{510+650+500+600+250+375} = \frac{2260}{2885} = 0.783$

$p_{SW} = \frac{350+370+500+375}{350+370+500+375+420+600} = \frac{1595}{2615} = 0.6099$

Thus the proportion of weaving traffic to non-weaving traffic is more in the East-South direction.

The weaving traffic movements in the East-South direction is shown in figure **??**.

Therefore, the capacity also need to be found only for that section.

From equation 11.2, $Q_{ES} = \frac{280\times13.5[1+\frac{10}{13.5}][1-\frac{0.783}{3}]}{1+\frac{13.5}{54}} = 2161.164$ veh/hr.

11.6 Summary

11.7 Problems

Chapter 12

Traffic signal design

12.1 Overview

The conflicts arising from movements of traffic in different directions is solved by time sharing of the facility. The advantages of traffic signal includes

- an orderly movement of traffic
- an increased capacity of the intersection
- and requires only simple geometric design.

However the disadvantages of the signalized intersection are it affects larger stopped delays, and the design requires complex considerstions. Although the overall delay may be lesser than a rotary for a high volume, a user is more concerned about the stopped delay.

12.2 Defintions and notations

A number of definitions and notations are there to be understood in signal design. They are discussed below:

1. Cycle: A signal cycle is one complete rotation through all of the indications provided.

2. Cycle length: Cycle length is the time in seconds that it takes a signal to complete one full cycle of indications. It indicates the time interval between the starting of of green for one approach till the next time the green starts. It is given by the symbol "C".

3. Interval: Thus it indicates the change from one stage to another. There are two types of intervals - change interval and clearance interval. *Change interval* which is also called the yellow time indicates the interval between the green and red signal indications for an approach. *Clearance interval* which is also called "all red" is included after each yellow interval indicating a period during which all signal faces show red and is used for clearing off the vehicles in the intesection.

4. Green interval: It is the green indication for a particular movement or set of movements and is denoted by G_i.

5. Red interval: It is the red indication for a particular movement or set of movements and is denoted by R_i.

6. Phase: A phase is the green interval plus the change and clearance intervals that follow it. Thus it is the assigning of conflicting movements into separate groups. It allows a set of movements to flow and safely halt the flow before another set of movements.

7. Lost time: It indicates the time during which the intersection is not effectively utilized for any movement.

For eg, when the signal for an aproach turns from red to green, the driver of the vehicle which is in the front of the queuee, will take some time to percieve the signal (usually called as reaction time) and some time will be lost here before he moves. That reaction time is referred to as the lost time.

12.3 Signal design procedure

The signal design procedure involves six major steps. They include the phase design, determination of amber time and clearance time, determination of cycle length, apportioning of green time, pedestrian crossing requirements, and the performance evaluation of the above design.

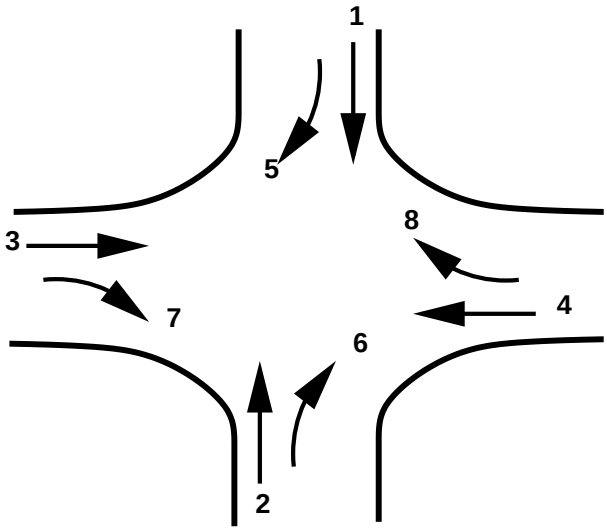


Figure 12.1: Four legged intersection

The objective of phase design is to separate the conflicting movements in an intersection into various phases, so that movements in a phase should have no conflicts. If all the movements are to be separated with no conflicts, then a large number of phases are required. In such a situation, the objective is to design phases with minimum conflicts or with less severe conflicts.

There is no precise methodology for the design of phases. This is often guided by the geometry of the intersection, flow pattern especially the turning movements, the relative magnitudes of flow. Therefore, a trial and error procedure is often adopted. However, phase design is very important because it affects the further design steps. Further, it is easier to change the cycle time and green time when flow pattern changes, where as a drastic change in the flow pattern may cause considerable confusion to thedrivers. To illustrate various phase plan options, consider a four legged intersection with through traffic and right turns. Left turn is ignored. See figure 12.1

The first issue is to decide how many phases are required. It is possible to have two, three, four or even more number of phases.

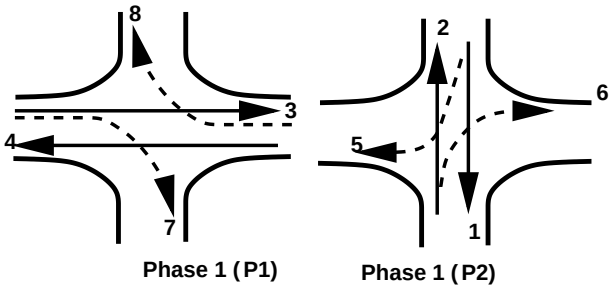


Figure 12.2: Two phase signal

12.3.1 Two phase signals

Two phase system is usually adopted if through traffic is significant compared to the turning movements. For example in figure 12.2, non-conflicting through traffic 3 and 4 are grouped in a single phase and non-conflicting through traffic 1 and 2 are grouped in the second phase. However, in the first phase flow 7 and 8 offer some conflicts and are called permitted right turns. Needless to say that such phasing is possible only if the turning mmovements are relatively low. On the other hand, if the turning movements are significant ,then a four phase system is usually adopted.

12.3.2 Four phase signals

There are atleast three possible phasing options. For example, figure 12.3 shows the most simple and trivial phase plan.

Here, flow from each approach is put into a single phase avoiding all conflicts. This type of phase plan is ideally suited in urban areas where the turning movements are comparable with through movements and when through traffic and turning traffic need to share same lane. This phase plan could be very inefficient when turning movements are relatively low.

Figure 12.4 shows a second possible phase plan option where opposing through traffic are put into same phase.

The non-conflicting right turn flows 7 and 8 are grouped into a third phase. Similarly flows 5 and 6 are grouped into fourth phase. This type of phasing is very efficient when the intersection geometry permits to have

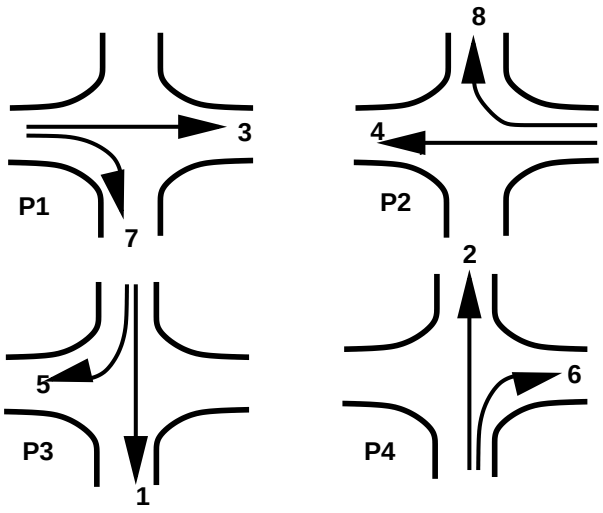


Figure 12.3: One way of providing four phase signals

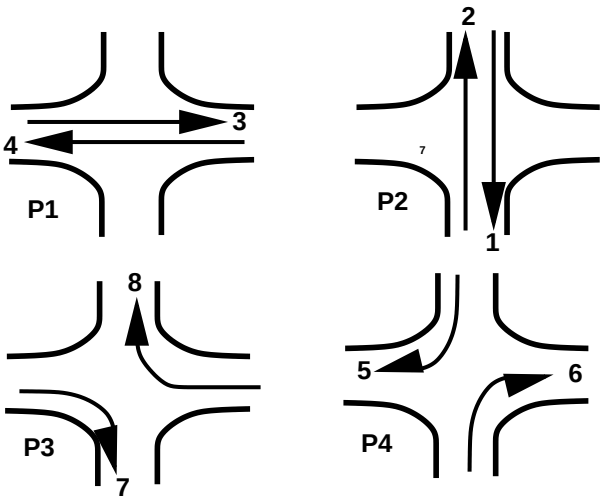


Figure 12.4: Second possible way of providing a four phase signal

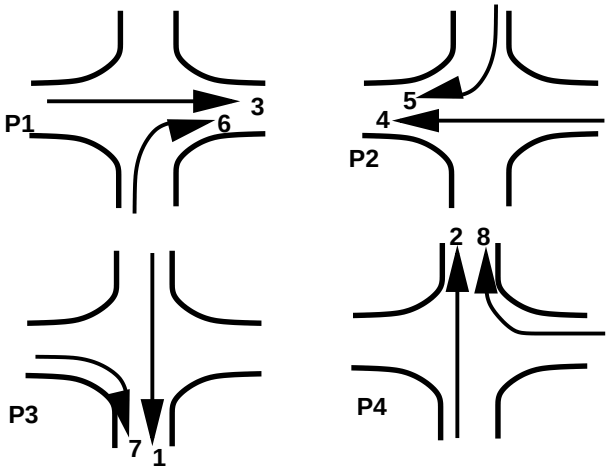


Figure 12.5: Third possible way of providing a four-phase signal

atleast one lane for each movement, and the through traffic volume is significantly high. Figure 12.5 shows yet another phase plan.

There are five phase signals, six phase signals etc. But they are used in the case of adaptive siganls, that is the signals which are highly responsive to the real time traffic. They include the combination of basic four phases.

12.4 Interval design

There are two intervals, namely the change interval and clearance interval normally provided in a traffic signal. The change interval or yellow time is provided after green time for movement. The purpose is to warn a driver approaching the intersection during the end of a green time about the coming of a red signal. They normally have a value of 3 to 6 seconds.

The design consideration is that a driver approaching the intersection with design speed should be able to stop at the stop line of the intersection before the start of red time. ITE's recommended methodology for computing the appropriate length of change interval is as follows:

$$y = t + \frac{v_{85}}{2a + 19.6g} \tag{12.1}$$

where y is the length of yellow interval in seconds, t is the reaction time of the driver, v_{85} is the 85^{th} percentile

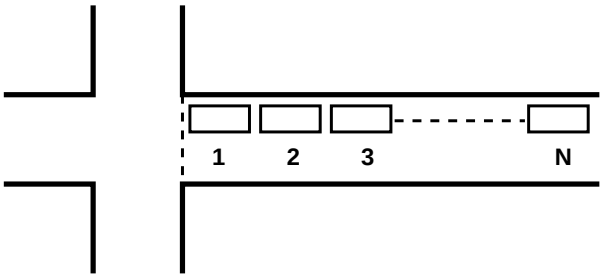


Figure 12.6: Group of vehicles at a signalised intersection waiting for green signal

speed of approaching vehicles in m/s, a is the decceleration rate of vehicles in m/s^2, g is the grade of approach expressed as a decimal. Change interval can also be approximately computed as $y = \frac{SSD}{v}$, where SSD is the stopping sight distance and v is the speed of the vehicle. The clearance interval is provided after yellow interval and as mentioned earlier, it is used to clear off the vehicles in the intersection. Clearance interval is optional in a signal design. It depends on the geometry of the intersection. If the intersection is small, then there is no need of clearance interval whereas for very large intersections, it may be provided.

12.5 Cycle time

Cycle time is the time taken by a signal to complete one full cycle of iterations. i.e. one complete rotation through all signal indications. It is denoted by the letter C. The way in which the vehicles depart from an intersection when the green signal is initiated will be discussed now. Figure 12.6 illustrates a group of N vehicles at a signalised intersection, waiting for the green signal. As the signal is initiated, the time interval between two vehicles, referred as headway crossing the curb line is noted. The first headway is the time interval between the initiation of the green signal and the vehicle crossing the curb line whereas the second headway is the time interval between the first and the second vehicle crossing the curb line. Successive headways are then plotted as in figure 12.7.

The first headway will be relatively longer since it includes the reaction time of the driver and the time necessary to accelerate. The second headway will be comparitively lower because the second driver ca overlap his/her reaction time with that of the first driver's. Finally we can see that the headways level out. The level

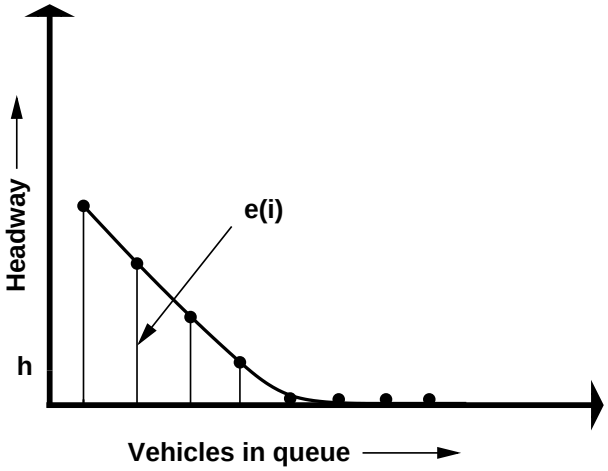


Figure 12.7: Headways departing signal

headway which characterises all headways begining with the fourth or fifth vehicle, is defined as the saturation headway, and is denoted as h in the figure. This is the headway that can be achieved by a stable moving platoon of vehicles passing through a green indication. If every vehicles require h seconds of green time, and if the signal were always green, then s vehicles/per hour would enter the intersection. This s is called saturation flow rate. Thus,

$$s = \frac{3600}{h} \tag{12.2}$$

where s is the saturation flow rate in vphgl, h is the saturation headway in seconds. The unit of saturation flow rate is vehicles per hour of green time per lane. But we know that the signal wont be green always. Also the headway will be more than h paricularly for the first 4 or 5 vehicles. This difference between the actual headway and h is denoted as e(i) as shown in figure 12.7. These differences can be added for the first few vehicles,

$$l_1 = \sum_{i=1}^{n} e(i) \tag{12.3}$$

The green time required to clear N vehicles can be found out as,

$$T = l_1 + h(N) \tag{12.4}$$

where T is the time required to clear N vehicles through signal, l_1 is the start-up lost time, h is the saturation headway in seconds.

12.5.1 Effective green time

There is another concept to find the amount of green time available. This is called effective green time. It is the sum of actual green time (G_i) plus the yellow and all red times (Y_i) minus the applicable lost times. This lost time is the sum of start-up lost time (l_1) and clearance lost time (l_2). Thus effective green time can be written as,

$$g_i = G_i + Y_i - t_L \tag{12.5}$$

12.5.2 Lane capacity

The ratio of effective green time to the cycle length ($\frac{g_i)}{C}$)is defined as green ratio. We know that saturation flow rate is the number of vehicles that can be moved in one lane in one hour assuming the signal to be green always. Then the capacity of the lane can be computed as,

$$c_i = s_i \frac{g_i}{C} \tag{12.6}$$

where c_i is the capacity of lane in vehicle per hour, s_i is the saturation flow rate in vehicle per hour per lane, C is the cycle time in seconds.

Problem

For a movement at a signalised intersection green time is given as 27 seconds, 3 seconds for yellow plus all red out of a 60 seconds cycle. If the satutration headway is 2.4 seconds/vehicle, the start-up lost time is 2 seconds/phase and the clearance lost time is 1 second/phase, find the capacity of the movement per lane?

Solution

Lost time, t_L = 2+1 = 3 From equation 12.5 effective green time, g_i = 27+3-3 = 27 seconds

From equation 12.2 saturation flow rate, $s_i = \frac{3600}{h} = \frac{3600}{2.4}$ = 1500 veh/hr.

Capacity of the movement per lane can be found out from equation 12.6 as $C_i = 1500 \times \frac{27}{60} = 675$ veh/hr/lane.

12.5.3 Critical lane

During any green signal phase, several lanes on one or more approaches are permitted to move. One of these will have the most intense traffic. Thus it requires more time than any other lane moving at the same time. If sufficient time is allocated for this lane, then all other lanes will also be well accommodated. There will be one and only one critical lane in each signal phase. The volume of this critical lane is called critical lane volume.

12.6 Determination of cycle length

The cycle length or cycle time is the time taken for complete indication of signals in a cycle. Fixing the cycle length is one of the crucial steps involved in signal design.

If t_{Li} is the start-up lost time for a phase i, then the total start-up lost time per cycle, $L = \sum_{i=1}^{N} t_L$, where N is the number of phases.

If start-up lost time is same for all phases, then the total start-up lost time is $L = Nt_L$.

Number of cycles per hour = $\frac{3600}{C}$

The total lost time per hour = $\frac{3600}{C}.L$

Substituting as $L = Nt_L$, total lost time per hour can be written as = $\frac{3600.N.t_l}{C}$

The total effective green time available for the movement in a hour will be one hour minus the total lost time in an hour. It is given by, $T_g = 3600 - \frac{3600.N.t_L}{C}$

Total number of critical lane volume that can be accommodated per hour is given by V_c as, $V_c = \frac{T_g}{h}$

Substituting for T_g, the maximum sum of critical lane volumes that can be accommodated within the hour is,

$$V_c = \frac{3600}{h}[1 - \frac{N.t_L}{C}] \tag{12.7}$$

$$= S_i[1 - \frac{N.t_L}{C}] \tag{12.8}$$

$$C = \frac{N.t_L}{1 - \frac{V_c}{S}} \tag{12.9}$$

$$\tag{12.10}$$

It is assumed that there will be uniform flow of traffic in an hour. To account for the variation of volume

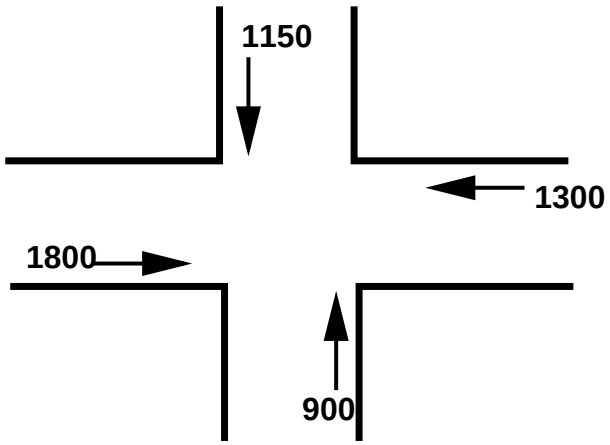


Figure 12.8: Traffic flow in the intersection

in an hour, a factor called peak hour factor which is the ratio of hourly volume to the maximum flow rate is introduced. Another ratio called v/c ratio indicating the quality of service is also included in the equation. Incorporating these two factors in the equation for cycle length, the final expression will be,

$$C = \frac{N.t_L}{1 - \frac{V_c}{S_i \times PHF \times \frac{v}{c}}} \tag{12.11}$$

Highway capacity manual (HCM) has given an equation for determining the cycle length which is as follows:

$$C = \frac{N.L.X_C}{X_C - \Sigma(\frac{V}{S})_i} \tag{12.12}$$

where N is the number of phases, L is the lost time per phase, $(\frac{V}{S})_i$ is the ratio of volume to saturation flow for phase i, X_C is the quality factor called critical $\frac{V}{C}$ratio where V is the volume and C is the capacity.

Problem

The traffic flow in an intersection is shown in the figure 12.8 below.

Given start-up lost time is 3 seconds, saturation head way is 2.3 seconds, compute the cycle length for the signal in that intersection.

Solution

If we assign the phases as shown below figure 12.9, the critical volume for the first phase which is the maximum of the flows in that phase = 1150 vph

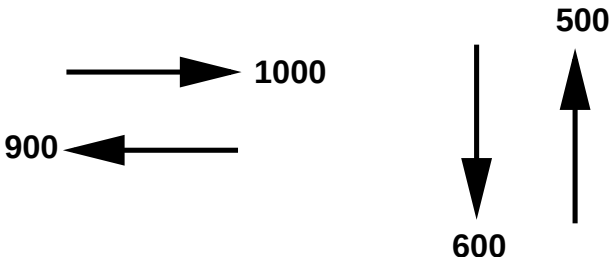


Figure 12.9: One way of providing phases

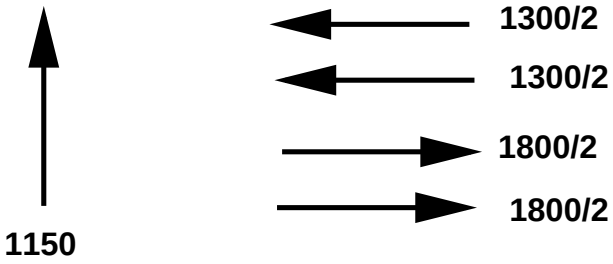


Figure 12.10: second way of providing phases

Similarly critical volume for the second phase = 1800 vph

Total critical volume for the two signal phases = 1150+1800 = 2950 vph

Saturation flow rate for the intersection can be found out from the equation 12.2 as $S_i = \frac{3600}{2.3} = 1565.2$ vph

The intersection can handle only 1565.2 vph, but the critical volume is 2950 vph .

Therefore the total of the critical lane volumes should be reduced by breaking them as shown below. So assigning the phases as shown below (12.10),

Here we are dividing the lanes in East-West direction into two, the critical volume in the first phase is 1150vph and in the second phase it is 900vph. The total critical volume for the signal phases is 2050 vph which is again greater than the saturation flow rate and hence we have to again reduce the lane volumes.

Assigning three lanes in East-West direction, as shown in figure 12.11, the critical volume in the first phase is 575 vph and that in the second phase is 600 vph, total critical lane volume = 575+600 = 1175 vph which is lesser than 1565.2 vph.

Now the cycle time for the signal phases can be computed from equation 12.10 as,

$C = \frac{2 \times 3}{1 - \frac{1175}{1565.2}} = 24$ seconds

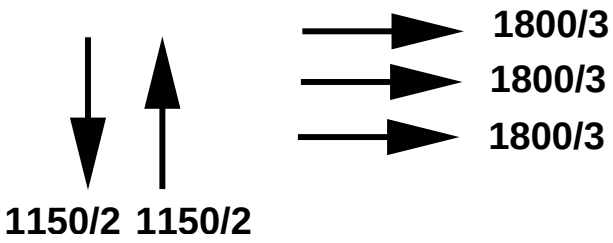


Figure 12.11: Third way of providing phases

12.7 Problems

Chapter 13

Traffic signal design

13.1 Overview

13.2 Green splitting

This is also called apportioning of green time. Some time will be lost as the satr-up lost time and clearance time. Thus green splitting is the proportioning of effective green time in the signal phase. The green splitting is given by,

$$g_i = \left[\frac{V_c i}{\sum_{i=1}^{n} V_{ci}}\right] \times T_G \tag{13.1}$$

where $V_c i$ is the critical lane volume and T_G is the effective green time available. Actual greentime can be now found out as,

$$G_i = g_i - y_i + t_{Li} \tag{13.2}$$

where G_i is the actual green time, g_i is the effective green time available, y_i is the amber time, and L_i is the lost time for phase i.

Problem

The phase diagram of an intersection with two phases are as shown in figure 13.7.

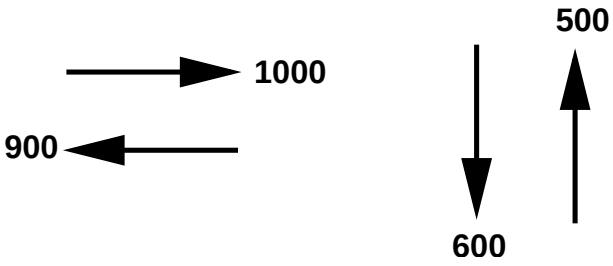


Figure 13.1: Phase diagram for an intersection

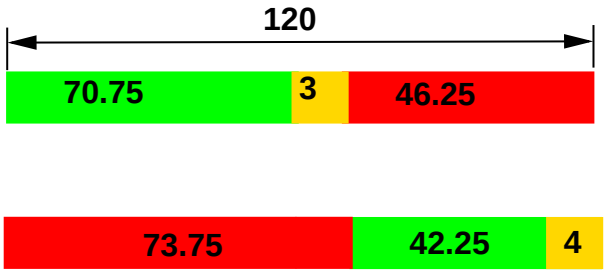


Figure 13.2: Timing diagram

Th lost time and yellow time for the first phase is 2.5 and 3 respectively and that for the second phase are 3.5 and 4 respectively. If the cycle time is 120 seconds, find the green time allocated for the two phases.

Solution Critical lane volume for the first phase = 1000 vph

Critical lane volume for the second phase = 600 vph

The sum of the critical lane volumes = 1000+600 = 1600 vph

Effective green time can be found out from equation 12.5 as t_g= 120-(2.5-3.5)= 114 seconds

Green splitting for the first phase, g_1 can be found out from equation 13.1 as $g_1 = \frac{1000}{1600} \times 114 = 71.25$ seconds

Green splitting for the second phase, g_2 can be found out from equation 13.1 as $g_2 = \frac{600}{1600} \times 114 = 42.75$ seconds

Actual green time can be found out from equation 13.2. Thus actual green time for the first phase, G_1 = 71.25-3+2.5 = 70.75 seconds

Actual green time for the second phase, G_2 = 42.75-3+2.5 = 42.25 seconds

The phase diagram is as shown in figure 13.2

13.3 Pedestrian crossing requirements

There are two wqys pedestrian crossing requirements has to be taken care i.e, either by proper phase design or by providing an exclusive pedestrian phase. It is possible in some cases to allocate time for the pedestrians without providing an exclusive phase for them. For eg, consider an intersesction in which the traffic moves from north to south and also from east to west. If we are providing a phase which allows the traffic to flow only in north-south direction, then the pedestrians can cross in east-west direction. But in some cases, it may be necessary to provide an exclusive pedestrian phase. In such cases, the procedure involves computation of time duration of allocation of pedestrian phase. If a minimum start-up time of 4 to 7 seconds is assumed, then the minimum time needed for pedestrians to safely cross a street is:

$$G_p = t_s + \frac{dx}{u_P} \tag{13.3}$$

where G_p is the minimum safe time required for the pedestrians to cross, often referred to as the "pedestrian green time", t_s is the start-up lost time, dx is th ecrossing distance in metres, and u_p is the walking speed of pedestrians which is the 15th percentile speed and usually taken as 1.2 m/s.

13.4 Performance measures

It is a measure used to evaluate the effectiveness of the design. There are many parameters involved to evaluate the effectiveness of the design and most common of these include delay, queuing and stops. Delay is a measure that most directly relates the driver's experience. It describes the amount of time that is consumed while traversing the intersection. The figure 13.3 shows a plot of distance versus time for the progress of one vehicle.

The desired path of the vehicle as well as the actual progress of the vehicle is shown. There are three types of delay as shown in the figure. They are stopped delay, approach delay and controlled delay. *Stopped time delay* includes the time at which the vehicle is actually stopped waiting at the red signal. It starts when the vehicle reaches a full stop, and when the vehicle begins to accelerate. *Approach delay* includes the stopped time as well as the time lost due to acceleration and deceleration. —t is measured as the time scale differential between the actual path of the vehicle, and its actual path extended. *Travel time delay* is measured as the time scale

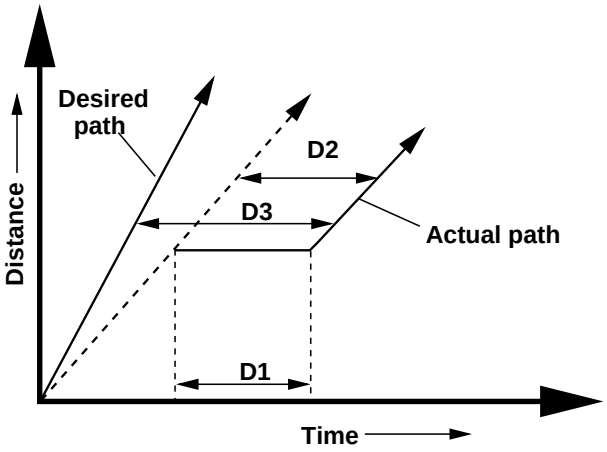


Figure 13.3: Illustration of delay measures

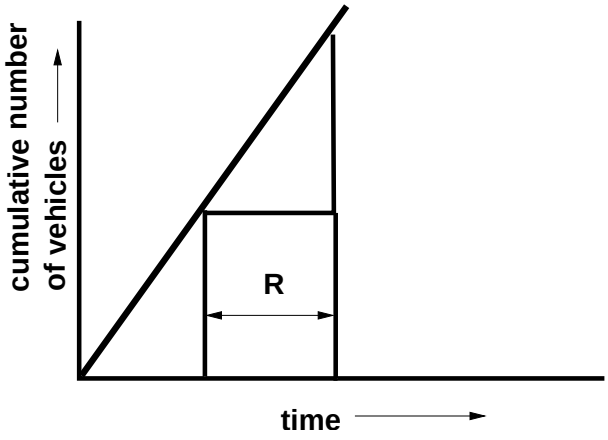


Figure 13.4: Graph between time and cumulative number of vehicles at an intersection

difference between the driver's desired time from the origin, and the actual time. For a signalized intersection, it is measured at the stop-line as the vehicle enters the intersection.

Vehicles are not coming uniformly to an intersection. i.e., they are not approaching the intersection at constant time intervals. They come in a random manner. The modelling of signalised intersection delay is complex. Most fundamental of the delay models is Webster's delay model. It assumes that the vehicles are arriving at a uniform rate. Plotting a graph with time along the x-axis and cumulative vehicles along the y-axis we get a graph as shown in figure 13.4.

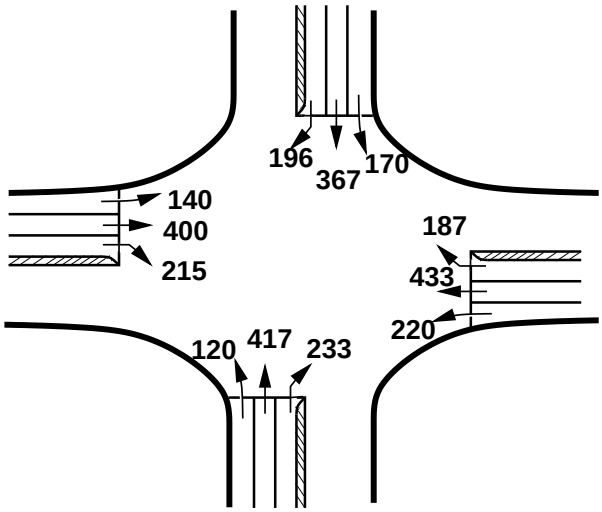


Figure 13.5: Traffic flow for a typical four-legged intersection

Webster derived an expression for delay per cycle based on this, which is as follows.

$$d_i = \frac{\frac{C}{2}[1 - \frac{g_i}{C}]^2}{1 - \frac{V_i}{S}} \tag{13.4}$$

where g_i is the effective green time, C is the cycle length, V_i is the critical flow for that phase, and S is the saturation flow.

Delay is the most frequently used parameter of effectiveness for intersections. Other measures like length of queue at any given time (Q_T) and number of stops are also useful. Length of queue is used to determine when a given intersection will impede the discharge from an adjacent upstream intersection. The number of stops made is an important input parameter in air quality models.

Problem The traffic flow for a four-legged intersection is as shown in figure 13.5.

Given that the lost time per phase is 2.4 seconds, saturation headway is 2.2 seconds, amber time is 3 seconds per phase, find the cycle length, green time and performance measure(delay per cycle). Assume critical V/C ratio as 0.9.

Solution The phase plan is as shown in figure 13.6.

Sum of critical lane volumes is the sum of maximum lane volumes in each phase, ΣV_{Ci} = 433+417+233+215 = 1298 vph.

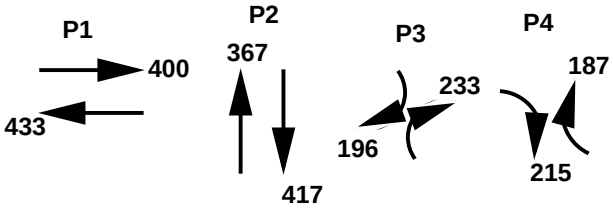


Figure 13.6: Phase plan

Saturation flow rate, S_i from equation 12.2 = $\frac{3600}{2.2}$ = 1637 vph.

$\frac{V_c}{S_i} = \frac{433}{1637} + \frac{417}{1637} + \frac{233}{1637} + \frac{1298}{1637} = 0.793$

Cycle length can be found out from the equation 12.12 as C =$\frac{4\times2.4\times0.9}{0.9-\frac{1298}{1637}}$ = 80.68 seconds ≈ 80 seconds

The effective green time can be found out as $G_i = \frac{V_{Ci}}{V_C} \times (C - L)$ = 80-(4×2.4)= 70.4 seconds, where L is the lost time for that phase = 4× 2.4

Green splitting for the phase 1 can be found out from 13.1 as $g_1 = 70.4 \times [\frac{483}{1637}] = 22.88$ seconds

Similarly green splitting for the phase 2,$g_2 = 70.4 \times [\frac{417}{1637}] = 22.02$ seconds

Similarly green splitting for the phase 3,$g_3 = 70.4 \times [\frac{233}{1637}] = 12.04$ seconds

Similarly green splitting for the phase 4,$g_4 = 70.4 \times [\frac{215}{1637}] = 11.66$ seconds

The actual green time for phase 1 from equation 13.2 as G_1 = 22.88-3+2.4 ≈ 23 seconds

Similarly actual green time for phase 2, G_2 = 22.02-3+2.4 ≈ 23 seconds

Similarly actual green time for phase 3, G_3 = 12.04-3+2.4 ≈ 13 seconds

Similarly actual green time for phase 4, G_4 = 11.66-3+2.4 ≈ 12 seconds

Pedestrian time can be found out from 13.3 as $G_p = 4 + \frac{6\times3.5}{1.2}$ = 21.5 seconds

The phase diagram is shown in figure 13.7

Delay at the intersection in the west-east direction can be found out from equation 13.4 as d_{EW} = $\frac{\frac{104.5}{2}[1-\frac{23-2.4+3}{104.5}]^2}{1-\frac{433}{1637}}$=42.57 sec/cycle

Delay in 1 hour = 42.57×$\frac{3600}{104.5}$ = 1466.6 sec/hr

Delay at the intersection in the north-south direction can be found out from equation 13.4 as d_{WE} = $\frac{\frac{104.5}{2}[1-\frac{23-2.4+3}{104.5}]^2}{1-\frac{400}{1637}}$=41.44 sec/cycle

Delay in 1 hour = 41.44×$\frac{3600}{104.5}$ = 1427.59 sec/hr

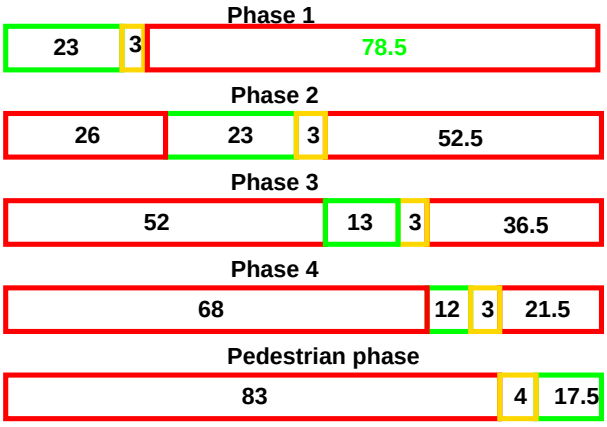


Figure 13.7: Timing diagram

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{NS} = \frac{\frac{104.5}{2}[1-\frac{23-2.4+3}{104.5}]^2}{1-\frac{367}{1637}}$=40.36 sec/cycle

Delay in 1 hour = $40.36 \times \frac{3600}{104.5}$ = 1390.39 sec/hr

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{SN} = \frac{\frac{104.5}{2}[1-\frac{23-2.4+3}{104.5}]^2}{1-\frac{417}{1637}}$=42.018 sec/cycle

Delay in 1 hour = $42.018 \times \frac{3600}{104.5}$ = 1447.51 sec/hr

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{SE} = \frac{\frac{104.5}{2}[1-\frac{13-2.4+3}{104.5}]^2}{1-\frac{233}{1637}}$=46.096 sec/cycle

Delay in 1 hour = $46.096 \times \frac{3600}{104.5}$ = 1587.996 sec/hr

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{NW} = \frac{\frac{104.5}{2}[1-\frac{13-2.4+3}{104.5}]^2}{1-\frac{196}{1637}}$=44.912 sec/cycle

Delay in 1 hour = $44.912 \times \frac{3600}{104.5}$ = 1547.21 sec/hr

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{WS} = \frac{\frac{104.5}{2}[1-\frac{12-2.4+3}{104.5}]^2}{1-\frac{215}{1637}}$=46.52 sec/cycle

Delay in 1 hour = $46.52 \times \frac{3600}{104.5}$ = 1602.60 sec/hr

Delay at the intersection in the west-east direction can be found out from equation 13.4 as $d_{EN} = \frac{\frac{104.5}{2}[1-\frac{12-2.4+3}{104.5}]^2}{1-\frac{187}{1637}}$=45.62 sec/cycle

Delay in 1 hour = $45.62 \times \frac{3600}{104.5} = 1571.598$ sec/hr

13.5 Summary

13.6 Problems