5.0 Introduction

Digital computers may be used to control urban road traffic in the modern day. The computers may be connected by data transmission lines to the traffic signal controllers at street junctions to form what are now called TMS. Thus it becomes possible to centrally co-ordinate the traffic signal timings over a wide area to check if the signals operate. Thus traffic may be diverted towards the free space and away from congested areas. It seems probable that in the foreseeable future, increases in the real cost of vehicle fuels, lost causes by accident, environmental impact and decreases in the cost of computer equipment will add further impetus to the development and use of TMS.

At present area coordinated signals are being set in fixed time cycles. A set of time determines when the signal should turn green and red within a cycle time, that is common to all signals in one area of a town. Typically the cycle time is between 40 and 120 seconds and any one set is operated for at least 15 minutes and up to several hours. Fixed time plans are pre calculated to suit the average conditions that the traffic controller expects to occur at different times of the day and days of the week. In most areas separate fixed time cycles are calculated for the morning and evening peak conditions and for the period between these peaks. Now it sense that fixed time cycles may not give the best standard of the control if the information on average flow is seriously erroneous, if there are large random variations in flow or if unexpected events, such as an accident occurs by chance.

In practice, the costs of collecting and analyzing traffic data are such that, in many towns the information on average flows within junctions is sparse and frequently many months or years out of date and is thus of low quality. Even if the traffic information is
accurate a poor standard of control may still result if the method of calculating fixed time is
defective. Now a days in our country fixed time cycles are calculated by manual means
for example by drawing time distance diagrams that depict the progression of a group of
vehicles through several adjacent signals. Because of the complexity of the traffic
movements, in most cases it is preferable to use computers to search in a systematic way
for signal timings that minimize total traffic delay, stops, fuel consumption. Also it is a
heavy burden for the traffic control staff who must periodically collect traffic data and
check their operation.

Furthermore, unless vehicle detectors are installed throughout the street network, the
computer has no information on the current traffic situation and so cannot be
programmed automatically to perform traffic management functions such as restricting
the number of vehicles that can enter the congested areas. Vehicle detectors may be
located on the approaches to all signalized junctions to collect data on traffic behaviour.
It is possible to use other types of vehicle detectors that provide similar information on
vehicle presence. The detectors may be located as far upstream as possible from the
signal stop line. The data from detectors on vehicle flow and occupancy are stored in the
computer in the form of cyclic profiles for each approach to a signal. The accuracy of the
profile depends upon the values assumed for turning flows, discharge from queues, and
effective green time and so on. On each section of street, cyclic profiles are stored and
the traffic model makes a prediction of current value of the queue of vehicles. The
computer controls the red and green signal time according to the queue.

Widespread congestion in a town can occur where the queues, which may start from
just one bottleneck, grow in length and extend backwards into upstream junctions. There
may then be a loss of capacity at the upstream junctions which causes further
congestion on other streets. Eventually, it is possible for the congestion to spread over
large areas of a town. To reduce the possibility of this happening it is desirable to control
traffic signals so that their associated queues do not extend into adjacent junctions. The
traffic model measures the proportion of the cycle time that the detector is occupied by a
queue. This information is used by the computer to alter the signal timings so as to reduce the likelihood of the queue blocking the upstream junctions.

The queues, number of stops and level of congestion depend upon many factors but of the particular importance is the number of vehicles that are attempting to travel through the area under control. In this chapter various mathematical models for traffic flow, area network control, and queuing analysis have been discussed with proper computer algorithm for calculating, estimating and predicting the queues and traffic flow. The control of traffic by using Neural Network and Artificial Intelligent support systems have been discussed.

The study of traffic flow was established only after motorized road vehicles began to appear in huge numbers. The fact that traffic volumes were about to reach the capacity of road infrastructure was the initiating factor for the scientific analysis of traffic flow. As early as 1934 Mr. Green Shields published a work named "A study of traffic capacity". It is remarkable how very early it was established that traffic flow must be a stochastic process. A historic document revealing this early cognition is a paper by Mr. Adams in 1936 en titled "Road traffic considered as a random series".

It was a considerably more significant approach to base the deterministic description of traffic flow in dense traffic on the movement of single vehicle. This idea, first published in 1950 by Reuschel, was initiated by an American team. They carried out a multitude of experiments with car drivers, looked at the distance behaviour between following vehicles and tried to model the observed behaviour using the so-called car following - equations. A significant new development was undertaken in Germany a wide manner in 1974, who developed an approach to simulate car following behaviour in a more realistic way.

In whatever way traffic flow will be modeled or described, the fundamental relationship between traffic volume, traffic density and mean speed will always be valid. Vehicular traffic theory can be broadly separated into two branches. Traffic Flow Theory and Car Following Theory. Traffic flow theory is concerned with finding relations
between three fundamental variables of traffic flow which are velocity \( v \), density \( p \) and flow \( q \). Only two of these variables are independent since they are related through \( q = pv \).

6.1 Traffic studies

The basic traffic studies are necessary to gather facts on traffic conditions. They must be set up and carried out so that the information is timely, reasonably accurate and unbiased. Studies may be classified as administrative which is the assembly of data already available in office which involves existing condition.

Inventories

An inventory is the accounting, tabulation, listing information, and describing existing the conditions. Some inventories such as traffic, parking facilities and transit route may require frequent updating.

Traffic Generators

Schools
Parks
stadium
Shopping centres
office complex

The use of automated data processing systems will facilitate accessibility to most inventories and data files especially in larger agencies. The details of intersections or street and high way sections should be readily available.
Volume studies

Many traffic analysis such as those relating to capacity, design, channelization and delay are most specifically involved with peak hour conditions. Many situations can be adequately described by counts that are taken during the single heaviest hour of morning traffic and of evening traffic. Hourly variation graphics show the present daily traffic.

Statistical distribution of traffic characteristics

Statistical distributions are useful in predicting events where events occur randomly. An event is said to occur randomly when each small increment of time or space is equally likely to contain an event. The event may be the arrival of vehicle at a left turn lane in a rural intersection. As long as the flow rate $q$ is constant each half second interval is as likely as every other half second interval to contain a vehicle arrival. As a further example, consider the distribution of occupied parking spaces in a parking garage. The event (a parked vehicle) would be random if every space had the same opportunity of being occupied. This would probably not be a random event because the spaces near the pedestrian exit and on the lower levels are more likely to be occupied than spaces more distant from the pedestrian exist. Statistical distribution can be classified into two general categories.

1. Counting or discrete distribution
2. Interval or gap distribution
Counting Distribution

Counting of the number of events that occur in a given time period is relatively easy and has been a useful tool of the traffic controller. Four counting distributions are discussed below.

1. Poisson distribution
2. Binomial Distribution
3. Negative Binomial distribution
4. Generalized Poisson distribution

Poisson Distribution

Poisson distribution is used to describe discrete events that are truly random and was the first distribution to be applied to an analysis of vehicle flow. The distribution is stated as

\[ P(x) = \frac{e^{-\lambda t} \lambda^x}{x!} \text{ for traffic counting} \]

where

- \( P(x) \) = probability that \( x \) vehicles will arrive during a counting interval \( t \)
- \( \lambda \) = average rate of arrival veh/s = flow rate
- \( t \) = duration of each counting interval
- \( m = \bar{\lambda} t \) average no. of vehicles during a period of duration \( t \)
- \( e \) = natural base of log.

The only parameter that must be estimated is the arrival rate \( \lambda \). Consider the 1 hour flow of 120 vehicles. The average minutes count in this case is 2 vehicle per minute since \( t = 1 \). Substitute 2 for \( \lambda \) and \( t = 1 \). The equation becomes

\[ P(x) = \frac{2^x e^{-2}}{x!} \text{ since } x \text{ is varying from } 0,1,2,3 \]

For each value of \( x \) a \( P(x) \) is determined. Knowing the number of counts per study period the \( P(x) \) value can be used to calculate the \( f(x) \) value, which is the number of minutes
expected to have a flow of exactly $x$ vehicles. The procedures are followed and the results are tabulated in the table.

The test data has been taken from Ernakulam city.

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<th>$x$</th>
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<th>$P(x)$</th>
<th>Theoretical frequency</th>
<th>$f_x$</th>
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If the number of vehicles ($x$) counted in intervals of time ($t = 1$ minute) and the observed frequency of each interval is taken, the values of $P(x)$ are determined; the theoretical frequency is $\sum f(x) \cdot P(x)$.

The above equation can be rewritten with $x$ values of 0, 1, 2, 3,

$$P(0) = \frac{\lambda^0 e^{-\lambda t}}{0} = e^{\lambda t}$$

$$P(1) = \frac{\lambda e^{-\lambda t}}{1!} = \frac{\lambda t}{1} [P(x = 1)]$$

$$P(2) = \frac{\lambda^2 e^{-\lambda t}}{2!} = \frac{m^2 e^{-m}}{2} = \frac{m}{2} [P(x = 2)]$$

$$P(3) = \frac{\lambda^3 e^{-\lambda t}}{3!} = \frac{m^3 e^{-m}}{3} = \frac{m}{3} [P(x = 3)]$$

So $P(x)$ can be calculated as follows

$$P(x) = \frac{m}{x} [P(x - 1)]$$
Generalized Poisson Distribution

The generalized Poisson Distribution is given by

\[ P(x) = \sum_{j=0}^{x-1} \frac{(r+1)^{j-1} e^{-r} r^j}{j!} \]

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\[ \sum f = 244 \quad \sum fx = 1822 \quad \sum fx^2 = 15622 \]

\[ \bar{x} = 7.46 \quad s^2 = 8.299 \quad k = 66 \quad p = 0.0898 \quad q = 0.102 \]

\[ p(0) = p^k = 0.898^{66} = 0.00082 \]

5.2 Interval Distributions

If the vehicles arrive in some pattern by the counting distribution it follows that there is also a distribution of intervals or gaps between the arrivals of successive vehicles. These intervals will be in time units and are continuous variables as opposed to discrete variables obtained from counting distributions.
Negative Binomial Distribution

If the mean flow changes during the counting period, giving a mean / variance ratio which is substantially less than 1.0 we use negative binomial distribution. The negative binomial distribution follows from the binomial distribution and gives the probability that x failures occur in n trials before getting k events. Consider a traffic stream made up of a mixture of cars and trucks. The passage of each vehicle is a trial. The passage of the passenger car is will be considered a successful event. The negative binomial distribution may be used to give a probability that six passenger cars will be observed (x = 6) before the third truck arrives (k = 3). The total no of trials n = x + k, (6 + 3) = 9

\[ P(x) = \frac{(x + k - 1)!}{x!(k - 1)!} p^x q^{x-k} \quad x = 0, 1, 2, 3 \]

calculations may be simplified by noting that

\[ P(0) = p^k \quad \text{and} \quad P(x) = \frac{x + k - 1}{x} q^{x-1} \quad q = 1 - p \]

The mean value of x is

\[ x = \frac{kq}{p} \quad \text{and the variance of x is} \quad kq/p^2 \]

Assume that 10% of the vehicles in a traffic stream are trucks (p = 0.10, q = 0.90). Then the probability that six passenger cars (x = 6) will be observed before the third truck (k = 3) is observed can be derived by the equation

\[ P(6) = \frac{6+3-1}{6! (3-1)!} (0.1)^3 (0.9)^6 = 0.0149 \]

The values of p and k are estimated as follows.

\[ p = \frac{x}{x^2} \quad k = \frac{x}{x^2 - x^2} \]
Binomial Distribution

As traffic flow becomes congested, the flow becomes more uniform, so that the variance of the number of vehicles per interval is decreased and the ratio of mean/variance is greater than one. Binomial distribution is fit for the case.

\[ P(x) = n \cdot p^x \cdot q^{n-x} \]

The two parameters of the binomial distribution are estimated as follows

\[ p = \frac{\bar{x} - s^2}{\bar{x}} \quad \text{and} \quad n = \frac{\bar{x}}{\bar{x} - s^2} \]

where \( \bar{x} = \text{mean number of events per n second} \)

\( s^2 = \text{variance in the no. of events} \)

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</tbody>
</table>

\[ \Sigma \quad 262 \quad 2343 \quad 23077 \]

\[ x = 8.94 \quad s^2 = 8.13 \quad n = 98 \quad p = 0.09 \quad q = 0.91 \]

\[ p(0) = q^n = 0.91^{98} = 0.0000968 \]
The interval distributions are
1. Negative exponential Distribution.
2. Shifted Exponential Distribution
3. Erlang Distribution

Negative Exponential Distribution

Negative exponential distribution is the interval distribution directly from the Poisson distribution. If there is no vehicle arrived at a time interval $t$ there will be a headway $h$ of at least $t$ seconds between the last previous arrival and the next arrival.

$$P(0) = P(h \geq t) = e^{-\lambda t}$$

But $\lambda = \frac{1}{\bar{h}}$ where $\bar{h}$ is the mean headway. So we may express in $p(h \geq t) = e^{\lambda t}$ The cumulative distribution function of the negative exponential may be written as

$$P(h \leq t) = 1 - e^{\lambda t} = 1 - e^{\lambda t}$$

The probability density function of the negative exponential distribution is

$$f(t) = \lambda e^{-\lambda t}$$

with mean and variance $\bar{h} = 1/\lambda$, $s^2 = \frac{1}{\lambda^2}$

$\lambda t = 7.46$

$\lambda = 0.1243$

$p(h \leq 0s) = 1 - e^{\lambda t} = 1 - e^{-1243(0)} = 0.000$

$p(h \leq 1s) = 1 - e^{\lambda t} = 1 - e^{-1243(1)} = 0.1168$

$p(h \leq 2s) = 1 - e^{\lambda t} = 1 - e^{-1243(2)} = 0.2201$

$p(h \leq 3s) = 1 - e^{\lambda t} = 1 - e^{-1243(3)} = 0.3112$

$p(h \leq 4s) = 1 - e^{\lambda t} = 1 - e^{-1243(4)} = 0.3917$

$p(h \leq 5s) = 1 - e^{\lambda t} = 1 - e^{-1243(5)} = 0.4628$

$p(h \leq 6s) = 1 - e^{\lambda t} = 1 - e^{-1243(6)} = 0.5256$

$p(h \leq 7s) = 1 - e^{\lambda t} = 1 - e^{-1243(7)} = 0.5810$

$p(h \leq 8s) = 1 - e^{\lambda t} = 1 - e^{-1243(8)} = 0.6300$

$p(h \leq 9s) = 1 - e^{\lambda t} = 1 - e^{-1243(9)} = 0.6733$
\[ p(h \leq 10s) = 1 - e^{-10h} = 1 - e^{-1213[10]} = 0.7115 \]

and so on. The probability between the head way interval is

\begin{align*}
(0 & \& 1) \text{ is } 0.1168 \\
(1 & \& 2) \text{ is } 0.2201 - 0.1168 = 0.1033 \\
(2 & \& 3) \text{ is } 0.3112 - 0.2201 = 0.0911 \\
(3 & \& 4) \text{ is } 0.3917 - 0.3112 = 0.0805 \\
(4 & \& 5) \text{ is } 0.4628 - 0.3917 = 0.0711 \\
(5 & \& 6) \text{ is } 0.5256 - 0.4628 = 0.0628 \\
(6 & \& 7) \text{ is } 0.5810 - 0.5256 = 0.0554 \\
(7 & \& 8) \text{ is } 0.6300 - 0.5810 = 0.0490 \\
(8 & \& 9) \text{ is } 0.6733 - 0.6300 = 0.0433 \\
(9 & \& 10) \text{ is } 0.7115 - 0.6733 = 0.0382
\end{align*}

The corresponding graph is plotted below to compare the probability for various distributions.

![Negative Exponential Distribution](image)

**Shifted Negative exponential Distribution**

Small time head ways are very unlikely to occur in vehicles observed in a single traffic lane, but the negative exponential distribution predicts the highest probabilities for short time head ways. One approach is to introduce a minimum allowable headway. A region in which head ways are prohibited. This can be accomplished by shifting the negative
exponential distribution to the right to a distance $c$. For the shifted negative exponential function the cumulative distribution is

$$P(h \leq t) = 1 - e^{-(t - c)}$$

for $t \geq c$

The probability density function is

$$f(t) = \begin{cases} 0 & \text{for } t < c \\ \frac{1}{t - c} \frac{t - c}{e^{t - c}} & \text{for } t \geq c \end{cases}$$

mean and variance $t = \frac{1}{\lambda}$, $s^2 = (t - c)^2$

The mean headway $\bar{t}$ can be calculated from observed frequency and the shifted parameter $c$ is assumed.

### Erlang Distribution

The shifted negative exponential distribution makes a probability of a headway less than $c$ equal to zero. A more desirable distribution, one that would have a very low but not zero, probability of a small headway is the Erlang Distribution

$$f(t) = \lambda \left( \frac{\lambda}{t - c} \right)^{k-1} e^{-\lambda(t - c)}$$

![Fig. 5.2.2](image-url)
The mean and variance are
\[ \mu = \frac{k}{\lambda}, \quad \sigma^2 = \frac{k}{\lambda^2} \]
The cumulative distribution function of Erlang distribution is
\[ P(h \leq t) = 1 - e^{-\lambda t} \sum_{n=0}^{k-1} \frac{(\lambda t)^n}{n!} \]
For \( k = 1 \) this reduces to
\[ 1 - e^{-\lambda t} \text{ the negative exponential distribution.} \]
For \( k = 2 \)
\[ P(h \leq t) = 1 - e^{-\lambda t} [1 + \lambda t] \]
For \( k = 3 \)
\[ P(h \leq t) = 1 - e^{-\lambda t} [1 + \lambda t + \lambda^2 t^2/2] \]
For \( k = 4 \)
\[ P(h \leq t) = 1 - e^{-\lambda t} [1 + \lambda t + \lambda^2 t^2/2 + \lambda^3 t^3/3!] \text{ and so on} \]

5.3 Vehicular Speed

Speed is a fundamental measurement of the traffic performance on the highway system. Most analytical and simulation model models of traffic predict speed as the measure of performance given the design, demand, and control of the highway system. Speed is also used as an indication of level of service, accident analysis and traffic noise and so on.

The wide spread availability of radar, nearly all speed checks may be conducted with such an electronic equipment. The radar meter operates on the principle that a radio wave reflected from a moving target undergoes a frequency change proportional to the speed of the target. Graphic records may be available to provide a permanent record.
Consider standing at a point along a highway facility during a relatively short period of time under uninterrupted flow conditions that is a location away from intersections. The speeds of the individual vehicles are measured and recorded. The sample mean and sample variance of these un-grouped speed observation would be

\[ \bar{x} = \frac{\sum_{i=1}^{n} x_i}{n} \]

\[ s^2 = \frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n - 1} \]

where
- \( \bar{x} \) = sample mean speed
- \( x_i \) = speed of the \( i^{th} \) vehicle
- \( s^2 \) = sample variance

In most cases the speed observations are grouped. The frequencies of each speed level or speed interval are determined from the series of individual vehicular speeds. Observed spot data has been collected from various stations of Ernakulam city and calculations are given below.

\[ \bar{x} = \frac{\sum_{i=1}^{g} f_i x_i}{n} \]

\[ s^2 = \frac{\sum_{i=1}^{g} f_i (x_i - \bar{x})^2}{n - 1} \]

where
- \( g \) = no. of speed groups
- \( i \) = speed group \( i \)
- \( f_i \) = no. of observation in speed group \( i \)
- \( x_i \) = mid point speed group \( i \)
- \( N \) = total no. of speed observations
<table>
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<th>x</th>
<th>f</th>
<th>cum</th>
<th>%</th>
<th>fx</th>
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</table>

\[ \Sigma \quad 368 \quad 12918 \quad 466880 \]

\[ \bar{x} = 35.10 \quad s^2 = 36.55 \]

![Cumulative Frequency Graph](Fig. 5.3.1)
The graph is a cumulative percentile distribution in which the vertical scale represent the percent of vehicles travelling at or less than the indicated speed group and the horizontal scale is speed in miles per hour. From the graph it is very clear that, it is fairly bell shaped distribution which is a normal distribution. The probability density function of normal distribution is

\[ f(x) = \frac{1}{\sigma \sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \]

5.4 Mathematical models On Flow density Speed

In this we shall establish the relationships between speed density, flow and travel time for uninterrupted and interrupted traffic flows. The difference between arrival flows measured upstream of queuing section and the departure flows measured at a reference point along the road is emphasized. The former is related to demand while the latter is related to capacity. The difference is of particular importance in over-saturated (congested) conditions where demand exceeds the capacity. The speed measured at a reference point along the road under congested
condition is known as moving queue speed. This speed is associated with departure flow which cannot exceed the capacity flow. On the other hand, the average speed based on travel time through a road section including the travel distance upstream of the queuing section is associated with demand flow rate exceed the capacity.

As a starting point three basic variables describing the movement of a vehicle as observed at a reference point along the road are headway, spacing, speed. Headway (h) is the time between the passage of the front ends of two successive vehicles. Spacing (L_h) is the distance corresponding to the headway, i.e., the distance between the front end of the leading vehicle and the front end of the following vehicle. Speed (v) is the distance travelled per unit time.
The relationship between the headway, spacing and speed is

\[ V = \frac{L_v}{h} \]

where

- \( h \) = headway (sec)
- \( L_h \) = spacing (m/veh)
- \( v \) = vehicle speed (m/sec)

Other variables shown are the vehicle length, space(gap) length and the corresponding vehicle passage time and gap time. The space (gap) length \( L_{sp} \) is the distance between two successive vehicles as measured between the back end of the leading vehicle and the front end of the following vehicle, and is equivalent to spacing less vehicle length.

Vehicle passage time \( t_v \) corresponds to vehicle length and is the time between the passage of the front and back ends of a vehicle. Gap time \( t_g \) is the time between the passage of the back end of the leading vehicle and the front end of following vehicle and equivalent to headway time minus vehicle passage time. Thus

\[
L_a = L_{sp} - L_v
\]

\[
t_v = \frac{L_v}{v}, \quad t_g = h - t_v = h - \frac{L_v}{v} = \frac{L_a}{v}
\]

where

- \( h \) = headway
- \( t_v \) = vehicle passage time (sec)
- \( t_g \) = gap time (sec)
- \( L_a \) = vehicle length (m/veh)

In the calculations relating to average traffic conditions the vehicle length should represent the actual traffic composition where the traffic stream is represented as a mixture of light vehicles (LVS) and heavy vehicles (HVs), the average vehicle length can be calculated as

\[
L_v = (1-P_{hv}) L_{avm} + P_{hv} L_{avlv}
\]

\[
P_{hv} = \text{proposition of heavy vehicles in the traffic stream}
\]

\[
L_{avm} = \text{average vehicle length for light vehicles / passage car units}
\]

\[
L_{avlv} = \text{average vehicle length for heavy vehicles}
\]
Flow rate

Flow rate (veh/sec) is the number of vehicles per unit passing (arriving or departing) a given reference point and can be related to a headway.

\[ h = \frac{1}{q} \]

Considering the difference between congested and not congested traffic operations it is important to distinguish between the arrival (demand) flow rate and the departure flow rate for a given traffic facility. For example, at a signalized intersection approach lane, the departure flow rate measured at the stop line is the queue discharge flow rate during the saturated portion of the green period, \( q = q_a \) (departure from queue) and the arrival flow rate after queue has cleared, \( q = q_a \) (not queued vehicles). The departure flow rate after queue clearance corresponds to the arrival flow rate measured under uninterrupted conditions at a point upstream of the back of the queue \( q_a = q_a \).

Density

Density is the number of vehicles per unit distance and is related to average spacing through

\[ k = \frac{1}{L_h} \text{ where } L_h \text{ is in meters and } k \text{ is in veh/m} \]

Since \( L_h = \frac{v}{q} \), the density is related to flow rate and speed as \( k = \frac{q}{v} \). The average spacing in a stationary queue \( L_{hj} \) (jam spacing) is the sum of the vehicle length \( L_v \) and the jam space length \( L_{sj} \):  

\[ L_{hj} = L_v + L_{sj} \]

\[ L_v = \text{vehicle length (m / veh)} \]
Average space length in a stationary queue measured from the back of the leading vehicle to the front of the following vehicle.

The jam density, i.e., the number of vehicles per unit distance in a stationary queue, can be calculated from the average spacing in queue.

\[ k_j = \frac{1000}{L_{sj}} \] where \( L_{sj} \) is in m/veh and \( k_j \) is veh/km.

Typical jam space length of 2m. Hence jam spacing of \( L_{sj} = 6 \) m per car and 12 m per heavy vehicle. So \( L_v = 4.3 \) m the jam spacing \( L_{sj} = 4.3 + 2.0 = 6.3 \) m/veh and the corresponding jam density is \( k_j = \frac{1000}{6.3} = 159 \) veh/km. Similarly, density at maximum flow is \( k_n = \frac{1000}{L_{sm}} \) where the spacing at maximum flow \( L_{sm} = 1000 \) v/h, \( q_n \) veh/h, \( v_n \) km/h in veh/km.

**Speed - Density - Flow Relationship**

As the vehicles speed up from a stationary queue, the space length between vehicles increases gradually and therefore the spacing increases and the density decreases. The corresponding flow rate increases to a maximum flow \( q_n \) and then decreases as the speed increases towards the free flow speed \( v_f \). The relationship between speed, density, and flow is known as the fundamental relation in traffic flow theory.

\[ q = vk \]

where \( q \) is the flow rate (veh/h or veh/sec), \( v \) is the speed km/h, m/sec, and \( k \) is the density (veh/km or veh/m). For uninterrupted traffic [Fig 5.4.3] the maximum flow rate \( q_n \) is the capacity (Q...
Region A represents under saturated conditions with arrival flows below capacity \((q = q_a \leq Q)\) which are associated with uninterrupted speeds \((v_f \geq v_a \geq v_n)\) where \(v_f\) is the free flow speed \(v_n\) is the speed at maximum flow. Region B as observed at a reference point along the road represents over saturated (congested) conditions with flow rates below the maximum flow \((q = q_c \leq q_m)\) which are associated with reduced speeds \((v_c \leq v_n)\).

Changes in condition from region A to region B through the maximum flow point represents queue formation (e.g., due to two lanes of traffic merging into one lane, or traffic stopping at traffic signals). On the other hand, changes in conditions from region B to region A through the maximum flow points represents queue discharge (e.g., one lane of traffic diverging into two lanes, or traffic departing from a queue at traffic signals).

Region C for uninterrupted flow represents arrival flows above capacity \((q_a > q_m)\) associated with average speeds based on travel time through the section. In this case the flow represents the demand flow rate which can exceed the capacity value.

For interrupted traffic [Fig. 5.4.4] capacity is given by \(Q = sg/e\) where \(s\) is the average queue discharge (saturation) flow rate, \(g\) is the effective green time and \(e\) is the cycle time. The average saturation flow rate is smaller than the maximum queue discharge rate \((s < q_n)\) because of lower discharge rate at the start of the green period and the capacity is the average saturation flow reduced by the available green time ratio \(g/e\).

The flow rate for the congested flow region (B) e.g., at a signalized intersection stop line is the rate departure from the queue. This corresponds to the instantaneous queue discharge flow rate during the green period \((q_i)\) that increases from zero to steady maximum queue discharge flow rate \((q_m)\) while the queue discharge speed increases from zero to steady queue speed \((v_n)\) corresponding to the maximum flow.
The free flow speed for uninterrupted flow ($v_f$) is the average speed that occurs under zero flow conditions. The corresponding zero flow speed for interrupted flow ($v_d$) includes the free flow travel time for uninterrupted flow plus total minimum (zero flow) delay at traffic interruptions.

A speed flow model can be used as a starting point. For region B the following model from derived using exponential queue discharge flow and speed models can be used.

$$v_s = v_n \left[ 1 - \left(1 - \frac{q_s}{q_m}\right) \frac{k_n}{k_j}\right]$$

where
- $v_s$, $q_s$ = speed (km/h) and flow rate (veh/h) in region B
- $v_n$ = speed at maximum flow (km/h)
- $q_m$ = maximum flow rate (veh/h)
- $k_n$ = density at maximum flow (veh/km)
- $k_j$ = jam density

if the speed $v_s$ is known the flow rate in region B ($q_s$) can be estimated from

$$q_s = q_m \left[ 1 - \left(1 - \frac{q_s}{q_m}\right) \frac{k_n}{k_j}\right]$$

The time dependent travel time function model for the region A & C of the speed flow relationship for uninterrupted or interrupted flow condition is

$$v = v_{of} \left[ 1 + 0.25v_{of} T_p \left( z + \sqrt{z^2 + m_{cf}/Q_{fp}} \right) \right]$$

where $v$ = travel speed in km/h ($v = v_u$ for uninterrupted flow, $v = v_d$ for interrupted flow)
\( v_{of} = \) zero flow travel speed in \( \text{km/h} \) (\( v_{of} = v_f \) for uninterrupted flow)

\( T_p = \) peak flow (analysis) period in hours

\( Q = \) capacity in vehicle per hour

\( z = x - 1 \)

\( x = q_d/Q \) (\( q_d \) is the demand flow rate)

\( m_e = \) a delay parameter

The slope of the speed-flow curve in region A and C is determined by the delay parameter \( m_e \). This slope indicates the rate of change of delay, i.e., the difference between the zero flow travel and travel time at a given flow rate. For interrupted conditions, this delay is due to vehicle interactions within the traffic stream. A speed flow function for interrupted traffic flow can be constructed from uninterrupted speed flow function by calculating the zero flow and speed at capacity \( (v_{of}, v_Q) \) from

\[
v_{of} = v_f \left[ 1 + d_m v_f/3600 \right] \quad v_Q = -v_{of} \left[ 1 + d_Q v_{of}/3600 \right]
\]

where

\( v_f = \) uninterrupted zero flow speed

\( v_{of} = \) uninterrupted traffic speed when the demand flow equals traffic capacity

\( d_m = \) minimum delay per unit distance

\( d_Q = \) delay per unit distance at capacity \( (q_d = Q) \)

### 5.5 Queuing Process in Traffic Flow

Queuing theory which was originally developed by A. K. Erlang in 1909 has found widespread application in the problems of high way traffic flow. In any highway traffic situation, it is necessary to know the distribution of vehicles arriving into the queuing system; whether the source of vehicles arrival is finite or infinite. The application of queuing theory to traffic control has been mainly developed around the regular and random distributions. When vehicles arrive at random, the number of vehicles arriving in successive intervals of time can be represented by Poisson distribution and depart with an exponentially distributed service rate.

Consider a traffic queue where \( P(n, t + dt) \) is the probability that the queue contains \( n \) vehicles \((n > 0)\) at time \( t + dt \). There are three ways in which the system could have reached
this state if it is assumed that \( dt \) is so small that only one vehicle could have arrived or departed.

1. A vehicle did not arrive or depart in time \( t \) to \( t+dt \)
2. The queue contained \( n-1 \) vehicles at time \( t \) and one arrived in \( dt \)
3. The queue contained \( n+1 \) vehicles at time \( t \) and one departed in time \( dt \)

Now with Poisson distributed arrivals

\[
P(n) = (\lambda t)^n e^{-\lambda t} / n!
\]

where \( P(n) \) is the probability of \( n \) vehicles arriving in time \( t \) when the mean rate of vehicle arrival is \( \lambda \).

\[
P(0) = (\lambda dt)^0 e^{-\lambda dt} / 0! = e^{-\lambda dt}
\]

where \( P(0) \) is the probability of zero arrivals in \( t \) to \( t+dt \)

\[
P(1) = (\lambda dt)^1 e^{-\lambda dt} / 1! = \lambda dt e^{-\lambda dt}
\]

where \( P(1) \) is the probability of one arrival in \( t \) to \( t+dt \)

\[
P(0) = (1-\lambda dt + \lambda^2 dt^2 / 2! - \lambda^3 dt^3 / 3! + \ldots)
\]

\[
P(1) = \lambda dt(1-\lambda dt + \lambda^2 dt^2 / 2! - \lambda^3 dt^3 / 3! + \ldots)
\]

Since \( dt \) is too small. So ignore the higher powers we have

\[
P(0) = 1-\lambda dt, \quad P(1) = \lambda dt
\]

Similarly the probability of 0 and 1 departure from the queue are

\[
P(0) = 1-\mu dt, \quad P(1) = \mu dt
\]

where \( \mu \) is the mean rate of departure from the queue where \( (n > 0) \) the system can reach a state of \( n \) vehicles at time \( t+dt \)

\[
P(n,t+dt) = P(Nat) P(a \text{ vehicle does not arrive or depart}) + P(n-1,t) P(a \text{ vehicle arrives}) + P(n+1,t) P(a \text{ vehicle departs})
\]

\[
= P(n,t)(1-\lambda dt)(1-\mu dt) + P(n-1,t) \lambda dt + P(n+1,t) \mu dt
\]

Ignoring second and higher powers of \( dt \)

\[
P(n,t+dt) = P(n,t)(1-\lambda dt - \mu dt) + P(n-1,t) \lambda dt + P(n+1,t) \mu dt
\]
\[ P(n,t+dt) - P(n,t) / dt = -P(n,t)[ \lambda + \mu ] + P(n-1,t) \lambda + P(n+1,t) \mu \]

In the limit for steady state solution the rate of change is zero. Hence
\[ P(n)[1 + \lambda / \mu ] = \lambda / \mu P(n-1) + P(n+1) \]

Similarly when \( n = 0 \) there are two ways in which the queue can contain \( n \) vehicles at time \( t + dt \)
\[ P(0,t+dt) = P(0,t)(1 - \lambda dt) + P(1,t) \mu dt \]
\[ P(0,t+dt) - P(0,t) / dt = P(1,t) \mu dt - P(0,t) \lambda dt \]

As before the steady state of the queue probability of \( n \) vehicles in the system is

when \( n = 1 \)
\[ P(1) = \lambda / \mu P(0) \]

\( n = 2 \)
\[ P(2) = [\lambda / \mu]^2 P(0) \]

\( n = 3 \)
\[ P(3) = [\lambda / \mu]^3 P(0) \]

\( n = n \)
\[ P(n) = [\lambda / \mu]^n P(0) \]

when the queue size may be infinite

\[ P(0) + P(1) + P(2) + P(3) + \ldots \ldots \ldots \ldots P(\infty) = 1 \]

\[ P(0) + [\lambda / \mu] P(0) + [\lambda / \mu]^2 P(0) + [\lambda / \mu]^3 P(0) + \ldots \ldots = 1 \]

\[ P(0) = 1 - \lambda / \mu \]

Also \[ P(n) = [\lambda / \mu]^n [1- \lambda / \mu] \]

The expected number in the queue is
\[ E_n = np(n) \]
\[ = 0 P(0) + 1 P(1) + 2 P(2) + 3 P(3) + \ldots + n P(n) \]
\[ = [\lambda / \mu] P(0) + 2 [\lambda / \mu]^2 P(0) + 3 [\lambda / \mu]^3 P(0) + \ldots + [\lambda / \mu]^n P(0) \]
\[ = [\lambda / \mu] P(0)[1 + 2 [\lambda / \mu] + 3 [\lambda / \mu]^2 + \ldots + (\lambda / \mu)]^{n-1} \]
\[ = [\lambda / \mu] P(0)[1 - (\lambda / \mu)]^2 \]

Because there is a probability that the queue will be zero the mean queue length
\[ E_m = (n-1)P(n) \]
\[ = nP(n) - P(n) + P(0) \]
\[ E_n = \lambda / \mu \]
The expected number in the queue as well as the mean queue length, the waiting time before being taken into service and total time in the queue are of considerable importance in the field of traffic. The waiting time distribution may be considered as two parts.

First there is the probability that the waiting time will be zero.

\[ P(0) = 1 - \frac{\lambda}{\mu} \text{ i.e. } n = 0 \]

Secondly there is the probability that the waiting time for a vehicle is between time \( w \) and \( w + dw \)

\[ P(w < \text{wait} < w + dw) = f(w)dw \]

Such a delay is possible as long as there is a vehicle in service which may be expressed as

\[ P(n \geq 1) = P(n) \]

For the waiting time for a vehicle to be exactly between \( w \) and \( w + dw \) all the vehicles in the queue ahead of one being consider.

\[ P(n-1, w) = [\mu w]^{n-1} e^{-\mu w} / n-1 ! \]

\[ P(1, w) = \mu dw \]

\[ f(w)dw = P(n) P(n-1, w) P(1, dw) \]

\[ = [\frac{\lambda}{\mu}]^n [1 - \frac{\lambda}{\mu}] [\mu w]^{n-1} e^{-\mu w} / n-1 ! \mu dw \]

\[ = \frac{\lambda}{\mu} [1 - \frac{\lambda}{\mu}] dw e^{-\mu w} [\mu w]^{n-1} / n-1 ! \]

\[ f(w) = -[\frac{\lambda}{\mu}] [\mu - \lambda] e^{-\mu w} \]

For more generalized cases when service time can no longer be described by a negative exponential distribution, the expected number in the queue when the arrivals are at random is given as

\[ E_n = \frac{\lambda}{\mu} + [\frac{\lambda}{\mu}]^2 \left[ 1 + e^2 \right] / 2[1 - \frac{\lambda}{\mu}] \]

where \( e \) is the coefficient of variation of the service time distribution that is the ratio of the standard deviation to the mean. If the service is exponential then \( e^2 = 1 \)

\[ = \frac{\lambda}{\mu - \lambda} \]
If the service is regular $c^2 = 0$ and

$$E_n = [\lambda / \mu][1 - \lambda / 2\mu] /[1 - \lambda / \mu]$$

In this case it has been shown that the average time a vehicle spends in queuing is given by

$$E_{tr} = \lambda / 2\lambda [\mu - \lambda]$$

The vehicles arrive at random. The number of vehicles arriving in successive time intervals may be represented by the Poisson distribution. The probability of $n$ vehicles arriving in a given interval of time may be calculated from

$$P_n = (\lambda t)^n e^{-\lambda t} / n!$$

This distribution is often referred to as the counting distribution because it describes the number of vehicles arriving at a given point on the highway.

5.6 Queuing Analysis

Queuing process occur in all transportation models and in everyday situations include freeway bottlenecks, parking facilities and so on. The input requirements for queuing analysis include the following five elements.

1. Mean arrival value
2. Arrival distribution
3. Mean service value
4. Service distribution
5. Queue discipline

The mean arrival value is expressed as a flow rate such as vehicle per hour. The arrival distribution can be specified as a deterministic distribution. The input is substituted for the term arrival. The mean service value is expressed as a flow rate such as vehicle per hour. The service distribution can also be specified as a deterministic distribution. The term departure is mean for service. The most common queue discipline encountered is referred to as first in first out. That is vehicles are served in the order in which they arrive. The arrival rate ($\lambda$) is specified in vehicle per hour and is constant for the study period. The service rate ($\mu$) has two states. Zero when the signal is effectively red and up
to saturation flow rate (s) when the signal is effectively green. The service rate can be
equivalent to the saturation flow only when a queue is present. Otherwise the service rate
is equal to the arrival rate if the signal is green. Thus the arrival rate goes through the
origin and slopes up to the right with a slope equal to the arrival rate. During the red
period the service rate is zero. At the start of the green period a queue is present and the
service rate is equal to the saturation flow rate (s). The cumulating arrival line intersects
the cumulating service line during the green period. At this point in time the queue is
dissipated and the cumulative service line overlays the cumulative arrival line until the
end of the green period. Then the pattern repeats itself with the service rate varying again
from zero to saturation flow rate and to arrival flow rate.

A series of identical triangles are formed with the cumulative arrival line forming the
top side of the triangles and the cumulating service line forming the other two sides of
the triangle. Each triangle represents one cycle length and can be analyzed to calculate
the set of five measures of performance. Let us take time duration of queue (t_Q), no. of
\[ \lambda t_Q = \mu (t_Q - r) \]
\[ t_Q = \frac{\mu r}{(\mu - \lambda)} \]
\[ P_{\text{tQ}} = 100 \frac{t_Q}{C} \]
The number of vehicles experiencing queue is represented by the vertical projection of
the queuing triangle. The first vehicle experiencing the queue is the vehicle that arrives
just after the signal turns red. All vehicles arriving during the red as well as the vehicle
arriving during the green but before the queue is dissipated experience the queuing
process and are forced to stop or slowdown considerably. Its value varies between \( \lambda r \) and
\( \lambda c \) and is expressed in number of vehicles.
\[ N_Q = \frac{\lambda t_Q}{3600} \]
\[ N = \frac{\lambda c}{3600} \quad P_{NQ} = 100 \frac{t_Q}{C} \]

where
\[ N_Q = \text{number of vehicles queued} \]
\[ N = \text{number of vehicles per cycle} \]
\[ P_{NQ} = \text{percent of vehicles queued} \]
The queue length is represented by the vertical distance through the triangle. At the beginning of the red period the queue length is zero and increases to its maximum value at the end of the red period. Then the queue length remains equal to zero until the end of the green period when the pattern repeats itself.

\[
Q_m = \frac{\lambda r}{3600} \quad Q^* = \frac{Q_m}{2} = \frac{\lambda r}{7200}
\]

where

- \(Q_m\) = maximum queue length
- \(Q^*\) = average queue length while queue is present

Individual delay is represented by the horizontal distance across the triangle. The first vehicle to arrive after the beginning of the red encounters the largest individual delay. Each vehicle arriving there after experiences a smaller and smaller individual delay until the queue is dissipated. Vehicles arriving thereafter until the beginning the next red encounters no individual delay.

\[
d_M = r
\]

\[
D_Q = \frac{r}{2} \quad D = \frac{rt_Q}{2C}
\]

where

- \(d_M\) = maximum individual delay
- \(D_Q\) = average individual delay while queue is present
- \(D\) = average individual delay

The total delay per cycle is represented by the cross sectional area of the queuing diagram triangle and is expressed in vehicle seconds.

\[
TD = \frac{N_q t}{2}
\]

where \(TD\) is the total delay in vehicle seconds

Queuing Patterns:

A variety of queuing patterns can be encountered. The classification scheme is based on how the arrival and service rate vary over time. Consider the pattern of a constant arrival rate. If the arrival rate is less than the service rate, no queue is
encountered. If on the other hand the arrival rate is greater than the service rate the queue has a never ending growth with the queue length equal to the product of time and the difference between arrival and service rates.

Consider the graph. In [Fig 5.6.1.a] the arrival rate is less than the service rate, no queuing is ever encountered. On the other hand, the arrival rate is greater than service rate the queue has a never ending growth with a queue length equal to the product of the time and the difference between the arrival and service rates as in [Fig 5.6.1.b]. If the
arrival rate is constant, but the service rate is less than the arrival rate for some periods of time, greater than the arrival rate for other periods of time, the service rate does not have to be in the form of a square wave. That is several changes in service rates of different amounts can be encountered which has been in [Fig 5.6.1.c,d]. In [Fig 5.6.2.a,b] the arrival rate varies over time, while the service rate constant over time. For queuing to occur and then be dissipated, the arrival rate must be greater than the service rate for some periods of time and less than the service rate during the other periods of time. The graph [Fig 5.6.2,c,d] shows that complex situation where both arrival and service rate vary over time. For queuing to occur and then be dissipated the arrival rate must exceed the service rate and later be less than the service rates. This indicates a square wave type of arrival rate and inverted square wave type of service rate.

![Fig 5.6.3](image)

![Fig 5.6.4](image)
The queuing diagram for the incident situation is given in the graph. The arrival rate ($\lambda$) is specified in vehicle per hour and is constant for the period. The normal service rate (without an incident) is indicated in the diagram as ($\mu$) and since it exceeds the arrival rate, no queuing would normally exist. However an incident occurs that reduces the service rate to $\mu_R$ which is below the arrival rate, and this lower service rate is maintained for $t_R$ hours. The cumulative vehicles versus time graph shows the arrivals as a straight line passing through the origin with a slope up and to the right equivalent to the arrival rate($\lambda$). For the first period of time the service line follows the arrival line until the incident occurs. At that point in time the service rate becomes equivalent to $\mu_R$ and maintains a flatter slope until the incident is removed. This continues until the arrival line and the service line intercept at which the service line once again overlays the arrival line.

**Varying arrival rate**

Assume service rate is constant vehicles per hour rate for the entire period. The arrival rate ($\lambda$) takes on the form of a typical peak period demand pattern, with a gradual increase in arrival rates in the early portion of the peak period and a gradual decrease in arrival rates in the latter portion of the peak period. The arrival rate begins at a constant rate of $\lambda_0$ during time period $T_0$ which is less than the service rate ($\mu$). During the time period ($T_1$) the arrival rate ($\lambda_1$) increases linearly from $\lambda_0$ to $\lambda_1$ and some time during this period of time the arrival rate ($\lambda_1$) begins to exceed the service rate. During the time period ($T_2$) the arrival rate remains constant at ($\lambda_2$). Then the arrival rate begins to decrease linearly from ($\lambda_2$) to ($\lambda_3$) and some time during this period the arrival rate ($\lambda_3$) becomes less than the service rate. After the time period ($T_3$) the arrival rate ($\lambda_3$) remains at constant rate ($\lambda_4$). If ($T_1$) & ($T_3$) are set equal to zero the arrival pattern will be rectangular. On the other hand if ($T_2$) is set equal to zero a triangle shaped arrival pattern will result.
The exact time that the arrival rate begins to exceed the service rate is
\[ ET = T_0 + T_1 \frac{(\mu - \lambda_0)}{(\lambda_2 - \lambda_0)} \]

The exact time at which the arrival rate becomes less than the service rate is
\[ T = T_1 \frac{(\lambda_2 - \mu)}{(\lambda_2 - \lambda_0)} + T_2 + T_3 \frac{(\mu - \lambda_2)}{(\lambda_4 - \lambda_2)} \]

The duration of the queuing process (QP) can be determined by investigating two cases.

If the queue is dissipated during time (T_p)
\[ Q_{pp} = T + \left[ \frac{(\lambda_2 - \mu)}{(\mu - \lambda_4)} \left( T + T_2 \right) \right]^{1/2} \]

On the other hand, if the queue is dissipated after the time period (T_p), the equation
\[ Q_{pn} = T/2 \left[ \frac{(\lambda_2 - \mu)}{(\mu - \lambda_4)} + 2 \right] - T_2/2 \left[ \frac{(\lambda_2 - \mu)}{(\mu - \lambda_4)} \right] + T_3 \left[ \frac{(\lambda_4 - \mu)}{(\lambda_4 - \lambda_2)} \right] \]

The number of vehicles adversely affected by the bottleneck can be expressed
\[ N_Q = \mu Q_p \]

The total delay in vehicle hours is
\[ TD = \int_0^T [\lambda(T) - \mu(T)] dt \]

The solution of the integral gives the total delay as a function of the flow rates.
5.7 Network and area traffic control

Area traffic control system plays an important role in determining the equilibrium between demand and supply in an urban highway network. The system provides the additional capability of monitoring the traffic flow, keeping track of its time-varying dynamics in great detail via vehicle detectors, signals, and computer altogether.

The basic paradigm of equilibrium in a transportation network is

\[ L = \text{level of service (such as trip time) on a particular facility} \]
\[ V = \text{volume of flow on this facility} \]
\[ T = \text{specification of the transportation system (including its control measures)} \]
\[ A = \text{specification of the activity system} \]

Then the supply function

\[ L = S(T, V) \]

shows an increase in the level of service as volume increases and the demand function

\[ V = D(A, L) \]

a decrease in volume as the level of service increases (in the negative sense). The resulting equilibrium point \( E(L_0, V_0) \) occurs at the intersection of the two curves. Computing the traffic equilibrium in a signal-controlled highway network, the sampling assumption is made that demand is an inelastic function fixed at a flow pattern \( F_0 \). The equilibrium value in this case \( E(L_0, F_0) \), represents the level of service at which the given demand is serviced. The most important element determining the level of service of traffic in an urban area is at grade control intersection. The effect of traffic flow on travel time between intersections is usually minor compared to its effect on the delay time incurred at the intersection itself. Therefore, the primary determinant of the level of service variable \( L \) becomes the delay time.

Let us consider first one approach to a signaled intersection, assuming that arriving traffic is not modulated by any nearby controlling device, the average delay per vehicle on the approach \( d \) can be regarded as the sum of two components.

\[ d = d_s + d_d \]
where $d_d$ is the delay that would result if the flow were uniform and $d_s$ is the additional delay caused by nature of traffic flow. The average delay per vehicle on the approach can be approximated from the formulae

$$d = k[c(1-g)^2/(2(1-q/s))] + [x^2/2q(1-x)]$$

where

- $c$ = the signal cycle time (sec)
- $G$ = effective green time for the approach
- $g = G/c$ proportion of cycle which is effectively green
- $q$ = arrival flow on approach
- $s$ = saturation flow at the signal stop line (veh/sec)
- $x = q/gs$ degree of saturation

It is seen that at higher degree of saturation the delay rises steeply. Theoretically the delay increases to infinity as the flow approaches capacity. But in practice the flow does not sustain a high value for a long period. It falls off at the end of the peak period and the queue does not reach a length required to cause excessive long delays.
To derive the level of service at which traffic through the intersection will be served both the green time $G$ and the cycle time $c$ have to be determined and all flows must be considered. The signal has two phases corresponding to the two possibilities of movement, N-S and E-W. The sum of the effective green times for the phase is $G_{EW} + G_{NS} = C - L$ where $L$ in this case is the total lost time for the intersection. To calculate the average delay per vehicle on each approach.

We have to obtain the rate of delay

$$D_{EW} = (q_E + q_w)d_{EW}$$
$$D_{NS} = (q_N + q_s)d_{NS}$$

The rate of total delay $D$ considering all flows through the intersection is

$$D = D_{EW} + D_{NS}$$

$G_{\text{min}}$ is the minimum effective green time that still can accommodate the demand on the approach though at a very high rate of delay and is given by $G_{\text{min}} = q_c/s$. The apportioning of green time among the conflicting streams at the intersection can be formulated as the following optimization program

$$\text{Min } D_j = D_j \text{ subject to } G_j = c - L$$

The optimal solution is obtained at an equilibrium point where the marginal rate of delay for the conflicting phases is equalized. The approximate rule for determining the optimal splits of green time

$$G_j^* = [c - L]y_j / Y$$

where $y_j$ is the maximum ratio of flow to saturation flow for the different approaches having simultaneous right of way during phase $j$ and $Y = y_j$.

To determine optimum cycle time $c$ for the intersection, capacity consideration play an important role. For each approach $i$ we must have $q_ic \leq G_j / c$. Summation over all phases at the intersection yields

$$y_j = g_j$$

The minimum cycle time

$$c_{\text{min}} = L / (1-y)$$

such a cycle will use an intolerable amount of delay.
When two or more intersections are in close proximity, some form of linking is necessary to reduce delays to traffic and prevent frequent stopping. A signal controlled intersection has a platooning effect on the traffic leaving it, and it is advantageous to have a signals synchronized. That is operating with a common cycle time. It also becomes necessary to co-ordinate the signals, that is to establish an offset between the signals, so that loss to traffic is minimized. The usual procedure for setting signals on arterial and in networks involves three steps. A common cycle time is determined according to the requirements of the most heavily loaded intersection. The split of green time are apportioned at each intersection according to the interacting flow or capacity ratios. A computer optimization procedure is used to determine a set of offsets throughout the network.

The signal controlled traffic network consists of a set of links \((i,j)\) connecting to the adjacent signals \(S_i\) and \(S_j\). Let

- \(G_{ij} (R_{ij}) = \text{effective green (red) time at } S_j \text{ facing link } (i,j)\)
- \(L_{ij} = \text{lost time at signal phase serving link } (i,j)\)
- \(\phi_{ij} = \text{offset time between } S_i \text{ and } S_j \text{ along } (i,j)\)
- \(q_{ij} (s_{ij}) = \text{average flow (saturation flow) on link } (i,j)\)

The link performance function is composed of a deterministic delay. The deterministic component

\[ Z_{ij} (\phi_{ij}, R_{ij}, c) \]

is given by average delay incurred per vehicle in a periodic flow through \(S_j\). The stochastic component which arises from variations in driving speeds, marginal friction and turns is expressed by the occurrence of an over flow. Queue is a non-homogeneous poisson process with a periodic intensity function represented by the flow pattern on the link. Therefore it can be considered the total delay in the network \(D\) to be composed of two components.

\[ D = D_d + D_s \]

where

\[ D_d = q_{ij} Z_{ij} (\phi_{ij}, R_{ij}, c) \]
\[ D_s = Q_{ij} (R_{ij}, c) \]
A number of constraint equations involving the decision variables are necessary to model the network. First the algebraic sum of offsets around any loop of the network must equal an integral multiple of the cycle time

\[ Q_{ij} = n_i c \]

where \( n_i \) is an integer number associated with loop \( i \). Effective green and effective red are related by

\[ G_{ij} + R_{ij} = c \]

In order for the network to be able to handle the given flow we must have for each link the capacity constraint as

\[ q_{ij}c \leq S_{ij} G_{ij} \]

For practical consideration including pedestrian crossing times and driver behaviour are prescribed as

\[ R_{ij} \geq R_{ij \text{ min}} \]
\[ C_{\text{min}} \leq c \leq C_{\text{max}} \]

Assuming for simplicity two phase intersections we have

\[ R_{ij} - l_{ij} = G_{kj} + l_{kj} \]

where \((i,j)\) and \((k,j)\) are assigned conflicting phases at \( S_j \)

Thus the network signal setting problem can be stated in a general form as the following non-linear optimization program

\[ \text{Min } D = D_0 + D_v \]

subject to

\[ \phi_{ij} = n_i c \]
\[ G_{ij} + R_{ij} = c \]
\[ R_{ij} - l_{ij} = G_{kj} + l_{kj} \]
\[ q_{ij} \leq S_{ij} G_{ij} \]
\[ R_{ij} \geq R_{ij \text{ min}} \]
\[ C_{\text{min}} \leq c \leq C_{\text{max}} \]
\[ G_{ij}, R_{ij} \geq 0, n_i \text{ is an integer} \]

This can be solved by mixed integer programming.
Computer vision and Neural Network for traffic monitoring

The ever increasing use of video cameras for a range of traffic surveillance and control task together with a steady fall in computer provides a clear opportunity for the introduction of reliable automatic video image analysis systems. Over the past decade scientist have developed a number of image processing systems for traffic analysis. It would be better to analyze the traffic representation by Artificial Neural Network System and Vision Technology. This is the first advanced hybrid neural network based computer vision system to be applied to monitor traffic current video image processing system for traffic analysis. It falls in to three categories; First straightforward detection and counting system capable of providing traffic data such as vehicle count, speed and headway measurements; Second congestion monitoring and incident detection systems for assessing traffic conditions based on spatial and temporal analysis of the traffic scene without measuring individual vehicle statistics and third vehicle identification, classification and tracking systems.

Systems in the first category generally employ algorithms which maximize processing speed to detect in real time, changes in image intensity, representing moving objects along the road. Systems in the second category more effective use of spatial information contained within video image to provide some description of traffic movements. Systems belonging to the third category demand most system resources as well as algorithm complexity. However it is evident that there is a need for low cost yet reliable traffic detection and analysis systems which are

- Adaptive to changes in real world environment
- Capable of operating independently of human operators
- Capable of intelligent decision
- Capable of monitoring multiple cameras
- Capable of continuous operation

When considering the design of more intelligent systems there are two general classes of adaptive decision making systems
Expert systems

Learning systems

Expert systems are based on explicit encoding of the knowledge of a human expert and are generally considered as alternatives to learning systems. In order to develop an expert system an articulate human expert must define all the rules and knowledge to be incorporated into the expert system. In many real world situations human expertise and experience may be scarce or to expensive to acquire, thus making expert systems unattainable.

A learning system is one which is able to make correct decisions based on criteria extracted from examples of successfully solved cases and examples of different traffic conditions or different classes of vehicle exist in abundance, learning systems begin to be much more attractive. Furthermore a learning system is still capable of capturing “Expert” knowledge by opting a process in which the expert chooses the most appropriate information to be presented to the system during its training phase. A learning system has in theory the potential to discover new relationship from the input pattern and improve performance by searching through the data in successfully solved cases.

Individually expert system and learning system have their own strengths and weaknesses. Neural Networks excel in pattern matching and classification but are not very well suited for precise numerical computations. In addition to that although a neural network may correctly identify a given situation there may be no explanation of how or why the system has come to that conclusion. In real world situations there will be limitations on what can be learnt from examples and hence an expert system may be used to reinforce a decision. By combining appropriate elements of both systems improved performance may be achieved.

Neural Network Classification

Neural network is a parallel distributed information processing system. It consists of a large number of highly interconnected, very simple processing elements known as
neurons. Each neuron has a number of inputs and one output which branches out to inputs of other neurons. There may be one or more layers of neurons in a network. The output of a neuron is a function usually non-linear sum of all inputs through weighted links. The knowledge of a network is therefore distributed throughout out weighted links. The weights are modified during the learning process by repeatedly showing an input pattern than adjusting the weights to produce the desired target output pattern.

The Biological Neurons

The human brain is an extremely complex interconnected neural network of over $10^{11}$ processing elements known as neurons. Each neuron is connected to $10^4$ other neurons which suggest approximately $10^{15}$ interconnections. A biological neuron consists of a cell body around the dendrites. The connecting points between neurons are called synapses. A single neuron receives stimulus from other neurons by its dendrites at the synapses, sums the stimulus at its cell body and based on the sum of the stimulus sends an output to other neurons through its axon.

Artificial Neuronal Network

Artificial neural network are computing techniques which are able to imitate activities of the human brain. A neural network is trained so that an application of a set of inputs produce the desired set of outputs. Training is accomplished by sequentially applying input vectors, while adjusting network weights according to a predetermined procedure. During training, the network gradually converge to values such that each input vector produces the desired output vector. Training is of three different types.

a. Supervised training
b. Unsupervised training
c. Graded training

The most popular one is the supervised training. It requires the pairing of each input vector with a target vector representing the desired output. The two vectors together is
referred to as a training pair. An input vector is applied, the output of the network is calculated and compared to the corresponding target vector. The difference (error) feedback through the network and weights are changed according to an algorithm that tends to minimize the error. The vectors of the training set are applied sequentially, errors calculated, and weights adjusted for each vector, until the error for the entire training set is an acceptably low level; in terms of the error criterion chosen.

The perceptron training algorithm employs the square difference criterion expressed as

\[ E = \frac{1}{2} \sum_{\rho} \sum_{j} (T_{\rho j} - O_{\rho j})^2 \]

where
- \( T \) = target activation
- \( O \) = actual output activation
- \( j \) = output unit
- \( \rho \) = input vector pattern

**The Perceptron Training Algorithm**

**Weight Initialization:** Set all weights and node threshold to small random numbers.

**Calculation of activation:** Activation level of an input unit is determined by the instant presented to the network. Activation level of an output unit is determined by

\[ O_j = F \left( \sum W_{ij} x_i - \theta_j \right) \]

**Weight training:**

Adjust the weight by

\[ W_{ij}(t+1) = W_{ij}(t) + \Delta W_{ij} \]

The weight change \( \Delta W_{ij} \) may be computed using the delta rule which states that

\[ \Delta W_{ij} = \eta \delta_j O_i \]

where
- \( \eta \) = learning rate coefficient
- \( \delta_j = T_j - O_j \) (the difference between the target output & actual output of unit j)
- \( O_i \) = activation level of unit i

Repeat the iterations until convergence.
ANN Paradigms

Adaline network and Back Propagation network are two of the widely used ANN paradigms in solving traffic monitoring problems.

Adaline Network

As a branch of artificial intelligence, the most remarkable achievements of neural networks have been made in pattern recognition. The range of ANN has however quickly become wider in recent years. A considerable number of potential applications of neural networks are in traffic.

In ANN the transfer function generally executes a threshold logic. In adaline the threshold logic is ignored and instead weighted sum is used as the output of the processing unit directly. Then the output is compared as

\[
\text{out} = \sum_{i=0}^{n} w_i x_i
\]

where

- \( \text{out} \) = output of the processing unit
- \( x_i \) = \( i^{th} \) input
- \( w_i \) = weight of the connection between \( i^{th} \) input and adaline

\( x_0 \) is a bias input and it is always set to 1. \( w_o \) is the weight corresponding to \( x_o \). The learning of the adaline network is implemented with an error correcting strategy which adjusts the weights according to the difference between actual output and the desired output. The weights are adjusted step by step through many iterations with the formula

\[
w_{\text{new}} = w_{\text{old}} + \delta w
\]

where

- \( w_{\text{new}} \) = updated weight
- \( w_{\text{old}} \) = the weight being updated
- \( \delta w \) = weight adjusting rate
The weight adjusting rate is determined by the following formula.

\[ \delta w_{ij} = x_i \left( out_j - t_j \right) \ln \]

where
- \( out_j = \) output of processing unit \( j \)
- \( t_j = \) desired output of processing unit \( j \)
- \( \ln = \) learning parameter

Trip generation prediction is a major process in traffic planning. The number of trips generated from a specific geographical zone is considered as an effect of the socioeconomic activities taking in that zone based on this consideration. Zonal socioeconomic indexes such as income population employment and the number of vehicles owned in the zone are generally used to infer the zonal trip rate generated.

To model a relationship between the trip rate and socio-economic indexes the most common existing approach is regression analysis. Linear regression analysis constructs a mathematical in the following form

\[ Y = c + b_1x_1 + b_2x_2 + \ldots + b_nx_n \]

where
- \( Y = \) dependent variable (trip rate)
- \( x_i = \) independent variable (socio-economic index)
- \( b_i = \) regression coefficient
- \( c = \) constant \( i = 1,2,\ldots,n \)

The constant and coefficient are determined with the least square error method. Such regression analysis is generally based on the following assumptions

The variance of the \( Y \) values about the regression line must be the same for all magnitudes of the independent variables.

The deviations of \( Y \) values about the regression line must be independent of each other and normally distributed.

The \( x \) values are measured without error.

The regression analysis requires an error free database. The assumptions would not be one hundred percent true in an actual situation. Such a shortcoming affects the prediction accuracy of the model.
The Back Propagation network

The back propagation network employs a generalized form of the delta rule which enables the training of multi-layered network.

Principle

Like a single layer perceptron a BP network typically starts out a random weight initialization. The network adjusts its weights, each time, a training pair is applied. The training take place in two stages.

Forward Pass

This involves presenting a sample input to the network and letting activation flow until they reach the output layer. The equation $O = F(X,W)$ is applied to each layer from the input to the output.
Backward Pass

During this stage the network's actual output from the forward pass is compared with the target output and error estimates are computed for the output units. The weights connected to the output units can be adjusted to reduce the errors. The error estimates for the hidden layers are derived from those of the output layer. Thus the errors propagate back to the connections stemming from the input units. Thus the reverse pass consists of two main steps.

1. Adjusting the weights of the output layer

The delta rule is modified due to the presence of the non-linear activation function.

\[
\text{OUT} = F(\text{NET}) = \frac{1}{1+e^{-\text{NET}}}
\]

\[
F'(\text{NET}) = \text{OUT}(1 - \text{OUT})
\]

\[
\delta_{k,q} \rightarrow \delta \text{ for the neuron } q \text{ in the output layer } k \text{ is expressed as}
\]

\[
\delta_{k,q} = \text{OUT}(1-\text{OUT})(\text{Target} - \text{OUT})
\]

\[
\Delta W_{pq,k} \rightarrow \text{change in the weight connecting a neuron } p \text{ in the hidden layer } j, \text{ to a neuron } q \text{ in the output } k \text{ is expressed as}
\]

\[
\Delta W_{pq,k} = \eta \cdot \delta_{q,k}
\]

therefore

\[
W_{pq,k}(t+1) = W_{pq,k}(t) + \Delta W_{pq,k}
\]

2. Adjusting the weights of a hidden layer

\[\delta\] for the hidden layers must be generated without the benefit of a target vector.
During the forward pass, a neuron $p$ in the hidden layer $j$ propagates its OUT to neurons 1,2,3,4,......... $n$ in the output layer through the interconnecting weights $W_{11}, W_{12}$. During the reverse pass, the same weights pass the $\delta$ value from the output layer to the hidden layer. Each weight is multiplied by the $\delta$ value of the output neuron to which it is connected

$$\delta_{1,k} W_{11,k} + \delta_{2,k} W_{12,k} + \delta_{3,k} W_{13,k} + \delta_{4,k} W_{14,k} + \cdots + \delta_{n,k} W_{1n,k}$$

This sum of products is multiplied by the derivation of the squashing function (non-linear activation function) to get $\delta$ of the hidden layer neuron.

$$\delta_{ij} = \text{OUT}_{i} (1 - \text{OUT}_{i}) \sum_q \delta_{q,k} W_{pq,k}$$

If $r$ is a neuron in the previous hidden layer $i$ then

$$W_{np,i(t+1)} = W_{np,i(t)} + \Delta W_{np,i}$$

$$\Delta W_{np,i} = \eta \delta_{p,j} \text{OUT}_{r,i}$$
Optical character recognition

Neural network architecture appears to lend itself well to optical character recognition. (Example: vehicle number plate recognition). The aim of this is to further investigate the learning and classification capability. To improve the accuracy, it may be trained with more than one character font type while at the same time shifting and rotating the position slightly to reduce position dependency.

5.7 Knowledge based system for traffic monitoring

The development of an Automatic Incident Detection system based on the application computer vision techniques. Computer vision involves the automatic digitizing, processing and interpretation of pictures from the road side CCTV cameras. The AID system is based on the analysis of video images from CCTV cameras installed in strategic sites along the road carried out in processing models which produces the meaningful real time spatial data. These data are further processed at central level to produce the spatial and temporal trailing of the received data to detect and follow up incidents and congestion along the road network.

Local Sensors Modules

The LSM performs the image processing and computer vision procedures. Each LSM has been connected to a single fixed camera. It primarily aims at detecting incidents that occur within or near by the camera field of view (usually several hundred meters). The LSM acts as a traffic sensor that calculates as a set of traffic measurements such as volume, velocity and concentration. With this data it also calculates the current level of service of the road.
Communication interface

It provides the link between the CS and all installed LSM. It is prepared to support most of the standard communication services, while the application protocols ensure the appropriate session management and data formats translation. The incoming information of this module is collected in cycle blocks and transformed into the normalized definition of traffic state.

Decision support module

Every cycle the data coming from sensors are pushed into traffic databases which in turn is linked to other static databases that maintain the other information of systems.

Decision Evaluation model

Gives the temporal consistency of the successive “cycle states” creating the linkage between alarms and the pattern of evolution (spatial & temporal) of such incident and congestion situations. This process is of key importance for the filtering out of the false alarms. The level of confidence is a parameter related to the accuracy of the data used for the system to detect an incident and set an alarm. The LSM will issue the level of confidence of detection but this may be modified by the CS according to rules that take in to account a number of additional data.

Traffic information database

This will include an increased number of sensors in urban areas and an exchange of traffic information between the metropolitan and urban areas. Additional equipment includes information boards and road side communication facilities for more reliable information concerning. Congestion levels together with predictions of current traffic
conditions such as journey time, accidents, restrictions and car parking space will be available to meet the increasing needs of travelers.

Configuration of the system

The basic concepts of the overall advanced traffic control and management system consists of three basic types of supervisions. The first one is the implementation of the real time control. It collects changing traffic information from the network and according to this information it optimizes vehicle flow by adjusting signals and providing appropriate traveller information.

The second one is daily traffic supervision. This monitors traffic flow and intervenes with control in response to incidents such as accidents, traffic restrictions and so forth.

The third one is the long term traffic management. It gathers information on static variations of traffic situations to establish overall traffic policies to execute large scale traffic regulations.

Traffic Management systems

The lowest control level is the terminal system consisting of several computers. Each computer directly controls the signals and detectors in each region.

At the intermediate level of control, the main function of the system is to develop the traffic information database and provide signal control strategies and management information. The level of control is handled by each of the dedicated computers sharing functions in the decentralised way.

The upper level is the administration component of the system and consists of large-scale information boards, multi function consoles and a group of computers. It is at this level that the so called man machine interface functions such as inquiries for information reference and checks of traffic policies take place.
Traffic control centre

The traffic control centre has large scale information boards, consoles and TV monitoring. All the information related to traffic condition in the whole area is presented to the operator, so that appropriate countermeasures may be taken. The large scale information boards are planned to give an overall picture of the general traffic situations. The consoles house the latest electronic work station as their CPUs and are capable of indicating traffic information on appropriate maps or diagrams. They can also indicate the statistical information about the traffic.

Collection of traffic information

Various types of traffic information are collected using vehicle sensors, registration plate readers and so forth installed at intersections and at road sides. They are used for measurement of present traffic condition and also for forecasting. Traffic characteristics continuously monitored include journey time, traffic flow, congestion, automatic vehicle flow classification and speed. The traffic information thus obtained is used to meet the needs of the new traffic control system, to adjust signal timings and provide information to the operator. Terminals collect information from existing sensors (Example image processing, ultrasonic and microwave detectors. They measure driving speed and classify vehicles.

Signal controls

Signal controls are the key measures to realize comfortable driving conditions by reducing and dispersing congestion and executing various traffic supervision measures. Often at junction of trunk or semi trunk at roads traffic demand exceeds the capacity. It is at critical intersections that traffic flows need to be dealt with to avoid the build up of
congestion throughout the network. The traffic situations at these critical intersections are categorized into three types each with an approximate control strategy. In under-saturated conditions safe and comfortable driving may be achieved by choice of signal cycles with coordinated signal control. Those who choose to drive at an excess speed are compelled to stop at signals. Whilst those who drive at the design speed may pass through the network with minimum delay. Such coordinated signal control strategies will be designed by the use of off-line simulation models.

Control of nearly saturated traffic

In a close to saturated condition one of the cross roads may be congested while another may be less crowded. Under such circumstances the new system employs a control of green split in accordance with the degree of saturation. It improves the green split to balance queues. In this method the splits are allocated using saturation rates and due consideration for number of cars in the queue. This method can also be applied to under as well as over saturated junctions and is very effective for multi phase intersections, where signal control is critical.

Control of over saturated traffic

Over saturation occurs when traffic demand exceeds the traffic handling capacity of an intersection. This results in congestion at each of the lanes. The control of splits will improve the efficiency of critical intersections. Also for a measure of the level of congestion on particular roads travel time ratio control will be applied, with the latter constraint signal control is defined so that the travel time for a vehicle at each of the inflow lanes comes as close as possible to a target value. On the other hand when more cars are entering congested roads from narrow streets vehicle flows may be suppressed by applying offsets.
Traffic management supporting functions

The supporting function includes traffic surveillance, data analysis, investigation, intervention and evaluation of the various counter measures. The congestion simulation uses a time series analysis of traffic volumes.

5.9 Conclusion

In this chapter various counting distributions have been discussed to analyze the current traffic situation in terms of traffic flow, volume and density. It is suggested that sensors and detectors may be used for counting purpose and the datum may be directly fed to the computers connected through communication channels. These datum automatically uses the mathematical models (statistical distributions) and the computers will give the information on current situation. In this work, most of the counting distributions, the theoretical frequency, and the observed frequency are compared and found fit to each other.

The speed-flow-density relation model gives ample space to control the flow. Queuing theory and Queuing analysis helps the computer to estimate the level of congestion and predict the queues. Area traffic control network model helps the traffic planner and Controller to have a centrally coordinated system. The use of artificial neural network and computer vision in traffic have been discussed.

In the light of the above terminology, it may inspire our people to have a centrally coordinated system with the help of computers to control traffic and avoid congestion.