AN APPROACH TO PREDICT TRAFFIC CONGESTION

by

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(ABSTRACT)

The main objective of this research is to develop a model to predict congestion. This model is developed using the techniques of simulation and as the model requires dynamic modeling, DYNAMO is used. This model incorporates the three-regime linear model for establishing a relationship between speed and density of the traffic stream. The input to this model is obtained from a presence type detector system. These measurements are then used to calculate various parameters and then the state of the traffic flow for the vehicular stream in the test zone is determined. This model also predicts the state of the traffic stream in any other section of the highway behind the test section. The model developed is flexible and easy to incorporate in any traffic control system.

This research is also intended to simulate the various traffic stream models and evaluate their performance regarding their capability to represent highway traffic flow conditions. A thorough review of the fundamentals of traffic flow is required to achieve these objectives. The simulation models developed for these traffic stream formulae incorporate various measures of effectiveness to determine congestion. These measures of effectiveness are used to define congestion. The study of the various traffic stream models is necessary in order to develop a flexible and efficient model to predict congestion. The congestion prediction model developed incorporates all the parameters required to define congestion.
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1. INTRODUCTION

A well planned and efficient transportation infrastructure is the basic requirement for the overall development of any society. The importance of the transportation facilities has increased tremendously in recent years due to the industrial development resulting in the fact that a number of countries across the world allocate a sizable amount of their annual expenditure to develop and maintain transportation facilities. This increase in demand may be attributed to the increased industrial output, increased demand for freight transportation, increased long distance travel by people, changes in government policies and so on which ultimately result in congestion, delays and inefficiency.

A knowledge of the fundamentals of traffic flow characteristics is an essential requirement in the planning, design and operation of the transportation systems. An assessment of traffic and environmental impacts of the new system modifications that are proposed can be accomplished only through a supply-demand framework that requires the understanding of flow characteristics and their interrelationships. A detailed analysis of the trade-off between traffic flow levels and levels of service and also the causes of existing system defects is required to develop operational improvement plans and predict their effects.
The efficiency of any transportation infrastructure depends to a large extent on the abilities of the transportation planners, designers and operators. The development and application of improved techniques for evaluating the impacts of land use changes and also the development of more precise behavior models of the effects of system changes on spatial, temporal, modal and total traveler responses is performed by transportation planners. The transportation designers deal with the problems where capacity increases are needed with serious constraints on available right-of-way and environmental impacts. The real-time balancing of system operation is done by the transportation operators. All these three branches of transportation owing to their dependency on the traffic flow characteristics require a knowledge of those parameters that affect the traffic flow characteristics.

A thorough knowledge of traffic analytical techniques and the ability to select the appropriate technique is required for the development of an efficient transportation infrastructure and also to study congestion characteristics. An understanding of flow characteristics is essential to convert pre-specified demand, supply and control information into mutually compatible qualitative expressions that can serve as direct input to the selected analytical technique. The analytical techniques may vary from a simple equation to a complex simulation model. Traffic stream models can also be use for uninterrupted flow situations where demands do not exceed capacities. For interrupted over-saturated flow situations, more complex techniques such as shock wave analysis, queuing analysis and simulation modeling can be employed. Microscopic analysis may
be employed for moderate-sized transportation facilities where the number of transport units passing through a system is relatively small. Macroscopic analysis may be used for high-density, large-scale systems in which a study of the behavior of group of units is sufficient.

In the past few years level-of-service concept has been widely applied in the design and operation of highways. Level-of-service as applied to the traffic operation on a particular highway refers to the quality of the driving conditions that are provided to the motorist by a particular facility. The factors that are involved in the level of service are: (1) speed and travel time, (2) traffic interruption, (3) freedom to maneuver, (4) safety, (5) driving comfort and convenience and (6) vehicular operational costs. The volume of traffic using a facility affects all these factors, and in general, the greater the volume, the more adverse are the affects. As the ratio of the volume of traffic on a highway facility to the volume of traffic the facility can accommodate (capacity) approaches unity, congestion increases. Basically congestion is a direct result of the nature of supply and demand on a facility. Both capacity and level-of-service are functions of the physical features of the highway facility and the interaction of vehicles in the traffic stream. Congestion results due to excess of demand over capacity and causes delays leading to inefficiency, thereby costing colossal amounts of money to the society, which could be avoided through proper planning, design and operation of highway facilities.
A deep insight into the fundamentals of traffic flow characteristics, traffic stream models and the analytical techniques is a necessity in the planning, design and operations of transportation infrastructure to develop a transportation system which allows minimum of delays and avoids congestion.
2. RESEARCH OBJECTIVES

The primary objective of this research is to develop a model to predict congestion. Prediction of congestion is essential in planning and operation of a highway control system. The various parameters such as speed, volume and density at times preceding breakdown in flow change from one state to another at rapid rates. These changes may occur from various preceding levels and achieve the change in state at various speeds. The predictive nature makes the measurements of these parameters naturally, associated with probabilities of failure. In this research, it is intended to use the various techniques of simulation to develop a model, which can measure these parameters with reasonable accuracy. It is also necessary that an appropriate methodology is adopted in developing this model.

A secondary, nevertheless, equally important objective of this research is to study and analyze the various traffic stream models that are available for transportation engineers by simulating various traffic flow conditions using these models. This research also envisages the selection of a traffic stream model, which can represent the traffic stream characteristics efficiently, so that, it can be used in developing a model to predict congestion. This research also involves a thorough review of the fundamentals of traffic stream characteristics.
3. LITERATURE REVIEW

This literature review is intended to pursue a thorough study of the fundamentals of the traffic flow characteristics. This also reviews the various traffic stream models that are available in transportation engineering. The past research done on these traffic stream models is also reviewed. The various traffic flow parameters that are used in describing traffic flow are studied with the perspective of interrelating these parameters to congestion depiction. In summary, this literature review covers the various aspects of the traffic flow theory.

3.1 TRAFFIC FLOW CHARACTERISTICS

Congestion is a situation in which the movement of traffic on a facility is severely restricted and at times the traffic flow may even become zero. This causes inconvenience and also increases the cost of travel due to decreased fuel efficiency, loss of time etc. Congestion can happen in any type of facility. Facilities are generally classified into two categories: (1) Uninterrupted Flow (2) Interrupted Flow.

(1) UNINTERRUPTED FLOW: The facilities which do not have any fixed elements, such as traffic signals, external to the traffic stream that cause interruptions to the traffic are called Uninterrupted flow facilities. Traffic flow conditions in this type of facility
are as a result of interactions among vehicles in the traffic stream, and between vehicles and the geometric and environmental characteristics of the roadway.

The characteristics of uninterrupted flow can be described by the three parameters, average travel speed, rate of flow and density which are algebraically related as

\[ v = S \times D \]

where,

\[ v = \text{rate of flow} \]

\[ S = \text{average travel speed} \]

\[ D = \text{density} \]

The rate of flow will be zero under two conditions:

(a) When there are no cars on the facility, density is zero, and rate of flow is also zero.

(b) When the density becomes so high that all vehicles stop, the rate of flow is zero.

The density at which all movements stop is called Jam density.

(2) **Interrupted Flow**: The facilities which have fixed elements causing periodic interruptions to traffic flow are called interrupted flow facilities. Flow on an interrupted flow facility is usually dominated by points of fixed operation, such as traffic signals,
Figure 1. Flow-Speed-Density Relationships
STOP and YIELD signs. The flow in this type of facility is characterized by saturation flow rate and lost time, as these are the factors which affect capacity of the facility.

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection in a stable moving queue and is represented algebraically as

\[ s = 3600/h \]

where, \( s \) = saturation flow rate
\( h \) = saturation headway

3.1.3 CONGESTION: A facility is said to be in congestion if the density of that facility exceeds critical density i.e., the traffic demand exceeds the capacity of the facility resulting in increased rate of flow and decreased travel speed.

Congestion is said to exist when speed inversion occurs. As the volume of the traffic stream increases due to increasing number of vehicles in the traffic stream, density increases and the speed remains dependant on the vehicle. The process of increased density reaches a critical stage beyond which the speed decreases rapidly causing congestion. This is called speed inversion.

It can be emphasized here that (1) congestion does not impede the input volume but delays the output volume (2) outgoing volume from congestion may be equal to the
capacity of the facility (3) for congestion to set in, there need not be a capacity restraint. Congestion may set in even due speed inversion. Speed inversion sets in as a result of reduction in speed of vehicle due to discomfort and inconvenience caused to the driver by the surrounding vehicle. Once speed inversion sets in, volume increases and results in congestion.

Congestion does not necessarily reduce volume; it may result in higher volumes. Congestion will be heavily dependent on the type of drivers existing in the traffic stream. The critical values of density and their associated speeds will vary with the location and the driver concerned.

Congestion is usually associated with a high rate of change in density and a correspondingly small rate of change in volume. As congestion develops, the increase in density results in a considerable increase in travel time to the individual driver, with no beneficial effects to the highway system.

The occurrence of speed inversion takes place when speed is numerically equal to density. It can be determined from a U factor where

\[ U = \frac{\text{speed}}{\text{density}} - \frac{\text{density}}{\text{speed}} \]

The factor U has a value of zero when speed equals density and is negative as congestion develops. Speed inversion starts when U is equal to zero.
The rate of flow and speed reach zero in extreme congestion and there will not be any movement of traffic in this facility.

3.1.4 CAUSES OF CONGESTION: Congestion may be caused due to the following factors:

- Inadequate capacity
- Inadequate lane widths.
- Inadequate clearances between the edge of travel lanes and nearest obstructions or objects at the roadside and in the median.
- The various types of vehicles that use the facility is also a cause of congestion as the dimensions and speed of the vehicles varies from one category to the other.
- Heavy pedestrian movements across the streets
- Poor pedestrian signal discipline.
- Inadequate green time for cross streets.
- Extensive parking activity along cross streets, which restricts vehicle movement.
- Very long discharge headway by traffic on cross streets.
- Limited storage capacity (i.e., short streets) on some cross streets.

3.1.5 MEASURES OF CONGESTION:

Measures to quantify congestion are very important in order to estimate the conditions existing on a facility. These measures are required not only to operate a
control system but also to provide an insight into the traffic conditions and are helpful in initiating necessary action to alleviate the conditions. Volume, Speed, Flow and Density of a traffic stream on a facility may be considered as measures of congestion.

3.1.5.1 **VOLUME**: Volume of traffic that can flow on a highway facility has always been of interest to Traffic Engineers as it gives the capacity of the facility. Literature on the capacity of highway facilities dates back to the 1920s with studies dealing with such topics as the effect of street cars on the automobile capacity and the discharge rate at the signalized intersections. In recent years the field of capacity analysis has been extended to levels of service. Hence, today capacity analysis not only includes the determination of capacity but also the trade-off between the quantity of traffic and the resulting level of service to the users.

Volume is defined as the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval. Volume is an important measure of congestion as it gives the physical size of traffic stream on the facility and is helpful in evaluating the traffic conditions, by comparing volume to capacity of the facility.

3.1.5.2 **SPEED**: Speed and travel time are fundamental measurements of the traffic performance of the existing highway system, and speed is a key variable in the redesign or design of new facilities. Most analytical and simulation models of traffic predict
speed and travel time as the measure of performance given the design, demand and control along the highway system. More extensive models then use speed or travel time as an input for the estimation of fuel consumption, vehicle emissions and traffic noise. Speed is also used as indication of level of surface, in accident analysis, in economic studies and in most traffic engineering studies.

Speed is defined as a rate of motion expressed as distance per unit time. In characterizing the speed of traffic stream, average travel speed is used as the representative value. The existing speed of traffic could be compared to the design speed and the traffic conditions could be evaluated.

3.1.5.3 **RATE OF FLOW**: The equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval (less than one hour) is called rate of flow. Rate of flow actually represents the number of vehicles passing a point during a particular time interval and if this is more than the design capacity, then it results in congestion. Rate of flow during peak period is related to hourly volumes through the peak hour factor (PHF) which is defined as the ratio of total hourly volume to the maximum fifteen minute rate of flow within the hour.

\[
PHF = \frac{\text{Hourly Volume}}{\text{Peak Rate of Flow (Within the Hour)}}
\]
3.1.5.4 **Density**: Traffic density is a fundamental macroscopic characteristic of traffic flow. It is an important characteristic that can be used in assessing traffic performance from the point of view of users and system operators. It is also employed as the primary control variable in freeway control and surveillance systems. The difficulty in measuring density inhibited its use until the early 1960s, when the presence type detectors were introduced. The 1985 Highway Capacity Manual uses traffic density as the primary measure of level of service for uninterrupted flow situations. Traffic density is expected to play even a more important role in the future in system-wide traffic performance evaluation and in on-line traffic responsive freeway control systems.

Density is defined as number of vehicles occupying a given length of a lane or roadway, averaged over time. Density can be obtained using the equation

\[ v = S \times D \]

Where, \( v \) = rate of flow  
\( S \) = average travel speed  
\( D \) = density

Density is a critical parameter describing traffic operations. It describes the proximity of vehicles to one another, and reflects the freedom to maneuver within the traffic stream.
3.1.6 **Capacity Analysis** :-

The Highway Capacity Manual (HCM) aids in estimating the highway capacity by describing various techniques which are developed for various conditions. This manual is useful in defining the measures of congestion also as it uses a set of procedures to estimate the traffic-carrying ability of facilities over a wide range of defined operational conditions, in performing the capacity analysis. It provides tools for the analysis and improvement of existing facilities, and for the planning and design of future facilities. The various operational criteria are defined by Highway Capacity Manual using Levels of Service. Ranges of operating conditions for different types of facilities are defined, and are related to amounts of traffic that can be accommodated at each level. The Highway Capacity Manual defines capacity, levels of service and measure of effectiveness which are used in capacity analysis which aids in the traffic flow studies.

3.1.6.1 **Capacity** :- The Highway Capacity Manual\textsuperscript{17} defines the capacity of a facility as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions. Capacity is defined for prevailing roadway, traffic and control conditions, which should be reasonably uniform for any section of facility analyzed. Any change in the prevailing conditions results in a change
in the capacity of the facility. In this definition, the roadway conditions refer to the geometric characteristics of the street or highway, including: the type of the facility and its development environment, the number of lanes (by direction), lane shoulder widths, lateral clearances, design speed and horizontal and vertical alignments. Traffic conditions refer to the characteristics of the traffic stream utilizing the highway facility. Control conditions refer to the types and specific design types of control devices and traffic regulations present on a given facility. The Highway Capacity Manual stresses on the need to recognize the potential for substantial variations in flow during an hour, and focuses analysis on intervals of maximum flow.

3.1.6.2 Levels of Service :- The concept of levels of service\textsuperscript{17} is defined by HCM as a qualitative measure describing operational conditions within a traffic stream, and their perception by motorists and passengers. A level of service definition generally describes these conditions in terms of such factors as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience and safety. Six levels of service are defined for each type of facility. They are given letter designations, from A to F, with level of service A representing the best operating conditions and level of service F the worst.

3.1.6.3 Measures of Effectiveness :- The Highway Capacity Manual defines levels of service, for each type of facility, based on one or more operational parameters which best describe the operating quality for the facility type under consideration. While the
concept of level of service attempts to address a wide range of operating conditions, limitations on data collection and availability make it impractical to treat the full range of operational parameters for each type of facility. The parameters selected to define levels of service for each type of facility are called measures of effectiveness, and represent those available measures that best describe the quality of operation on the facility type under consideration.

3.1.7 Acceleration Noise :-) Acceleration noise⁹ may be defined as the disturbance of the vehicle's speed from a uniform speed or can be considered as a measurement of the smoothness of traffic flow. It is dependent upon three factors namely, (1) the driver, (2) the road and (3) the traffic condition. Acceleration noise varies with the amount and frequency of acceleration and deceleration. The more violent and more frequent these variations are, the higher is the noise.

Congestion may be considered as a factor which influences the amount of acceleration noise. It has been observed in past studies on suburban highways that the acceleration noise increased due to increase in congestion which resulted because of higher volumes and influence of parked cars. The amount of acceleration noise in excess of the natural noise of the facility was basically due to the existing traffic condition. The acceleration noise may therefore be considered as a good measure of congestion as it gives a good indication of the degree of congestion due to its dependency on congestion.
3.2 TRAFFIC FLOW MODELS

The early traffic flow models assumed a single regime phenomenon over the complete range of flow conditions including free-flow and congested flow situations. The first Single-regime model was developed by Greenshields based on observing speed-density measurements obtained from an aerial photographic study. The model requires knowledge of both free-flow speed and jam density parameters in order to solve numerically for the speed-density relationship. The use of jam density presents a problem because according to this model, optimum density is equal to one-half of the jam density value. This is not comparable with observed optimum density values. Greenshields model\textsuperscript{24} expressed speed as

\[ u = u_r (1 - \frac{k}{k_j}) \]

Where,  
\( u = \) speed  
\( u_r = \) free-flow speed  
\( k = \) density  
\( k_j = \) jam density

The Greenberg model\textsuperscript{24} was the second single-regime model in which he concluded that a non-linear model might be more appropriate. Using a hydrodynamic analogy, the equations of motion and continuity for one-dimensional compressible flow
were combined and the following equation was derived:

\[ u = u_o \ln\left(\frac{k}{k}\right) \]

A disadvantage of this model is that free-flow speed is infinity. Another disadvantage is that this model makes a crude estimate of optimum speed as one-half of the design speed.

Underwood\textsuperscript{24} proposed the third single-regime model as follows:

\[ u = u_e \frac{k}{k^*} \]

This formulation requires knowledge of the free-flow speed, which is fairly easy to observe, and optimum density, which is difficult to observe and varies depending on the environment of the roadway. Another disadvantage of this model is that speed never reaches zero and jam density is infinity.

A group of researchers at Northwestern University proposed a fourth model when they observed that most speed-density curves appear as S-shaped curves.

The equation\textsuperscript{24} is as follows:
\[ u = u_c e^{-\frac{1}{2} \frac{k}{k_c^2}} \]

In this model speed does not go to zero when density goes to jam density.

Single-regime models were further developed by the introduction of a parameter in the formulation which would provide a more generalized approach. Drew proposed a formulation based on Greenshields model, with the introduction of an additional parameter 'n' as indicated in the following equation:

\[ u = u_c [1 - (k/k_j)^{n+\frac{1}{2}}] \]

Drew⁵ suggested varying 'n' from -1 to +1 and called these models a linear model (n = +1), a parabolic model (n = 0), and an exponential model (n = -1).

Pipes-Munjal²⁴ proposed a similar formulation that would provide a more generalized approach to single-regime models. Their formulation is:

\[ u = u_c [1 - (k/k_j)^u] \]

These are the various single-regime traffic flow models that are extensively used.
3.2.1 **SINGLE REGIME TRAFFIC FLOW MODELS - A STUDY**:

The necessity to develop more accurate interrelationships among the basic variables i.e., speed, density, volume etc., has become imperative as the number of road facilities operating at near capacity has increased. In the past a number of steady-state flow equations for the interrelationships among traffic flow variables have been suggested. Research in the past shows that the microscopic and macroscopic theories of traffic flow can be reduced to the equation of general car-following model formulated by Gazis, Herman and Rothery:

\[
\ddot{X}_{n+1}(t+T) = \frac{\alpha [\dot{X}_{n+1}(t+T)]^m}{[X_n(t) - X_{n+1}(t)]^{1}} \times [\dot{X}(t) - \dot{X}_{n+1}(t)]
\]

Where,

\( \dot{X} = \text{speed} \)

\( \ddot{X} = \text{acceleration/deceleration} \)

\( X_n, \ X_{n+1} = \text{positions of leading car and the following car, respectively} \)

\( T = \text{time lag of response to stimulus} \)

\( m, \alpha = \text{constant parameters} \)
The steady-state flow formulation of this equation can be obtained by integrating the above equation; it is given by Gazis, Herman and Rothery as

$$f_m(u) = cf_e(s) + c'$$

Where,

- $u$ = steady-state speed of the traffic stream
- $s$ = constant average spacing
- $c'$ and $c$ = some appropriate constants consistent with physical restrictions.

The microscopic and macroscopic traffic flow models can be obtained by selecting proper combinations of the exponents 'm' and 'l' in equations mentioned above.

An evaluation of single regime traffic flow models based on the above equations is done by Avishai Ceder and Adolf D. May to obtain an accurate relationship among the basic variables of traffic flow.

In this evaluation, the traffic flow variables were sampled over the range of all possible concentrations to ensure appropriate speed-concentration relationships. The evaluation method involved determination of the number of measurements falling in the most sparse 5-vehicle-per-mile concentration range and then a like number of measurements were randomly sampled from each of the other 5-vehicle-per-mile ranges. This statistical procedure provides uniform distribution of the data points over the available concentration range. In the evaluation procedure, an $m,l$ matrix is used in
which the various microscopic and macroscopic theories of traffic flow can be positioned. Each \( m \) and \( l \) combination represents a specific model that can be expressed mathematically by the equations discussed above.

For the single-regime model, only models with an \( x \)-intercept (jam concentration) and a \( y \)-intercept (free-flow speed) were considered. This limited the investigation to the region where \( m < 1 \) and \( l > 1 \). It is the equations discussed that the speed function and the spacing function of the sensitivity component remain in the numerator and denominator respectively. This limited the investigation to the region where \( m > 0 \) and \( l > 0 \) and the combination of these two requirements restricted the investigation of the \( m,l \) matrix to the region where \( 0 < m < 1 \) and \( l > 1 \). An upper limit was placed on ‘\( l \)’ such that \( l < 3.1 \) so that this limit on ‘\( l \)’ covers all the previous macroscopic models. For this range of \( m \) and \( l \) values, the following macroscopic equation can be derived:

\[
u^{1-m} = u_f^{1-m}[1-(k/k_j)^{l-1}]\]

where, \( u, u_f = \) steady-state and free-flow speeds

\( k, k_j = \) concentration and jam concentration

The constant, \( \alpha \), can be determined for the restricted \( m,l \) region as follows:

\[
\alpha = \frac{l-1}{1-m} \times \frac{u_f^{1-m}}{k_i^{l-1}}
\]
It can be observed from these equations that \( m \) and \( l \) are the basis for evaluating driver behavior at both the microscopic and macroscopic levels. If the restrictions for \( m \) and \( l \) considered previously are applied, a dependency of \( m \) and \( l \) on traffic flow characteristics can be obtained as follows:

The fluctuations of \( m \) can be determined from the equation:

\[
\ln\left[1 - \frac{k_o}{k_j}\right] = m = 1 - \frac{\ln\left(\frac{u_o}{u_f}\right)}{\ln\left(\frac{u_o}{u_f}\right)}
\]

The non-linear fluctuations of \( l \) can be estimated from the equation:

\[
\frac{k_o}{k_j} \left( l - 1 \ln\left(\frac{u_o}{u_f}\right) \right) = 1
\]

This analysis is applied to Greenberg, Greenshields, Underwood and Drake et al macroscopic models. None of these were found to be superior to the other macroscopic integer models.

It is observed that the Greenshields model results in a linear speed-concentration relationship and usually exhibits the undesirable characteristic of an extremely low jam concentration. The Greenberg model results in a concave-shaped speed-concentration
relationship and does not have a y-intercept which means that free-flow speed is infinity. The Underwood model results in a concave-shaped speed-concentration relationship and usually exhibits the undesirable characteristic of an extremely high free-flow speed and does not have an x-intercept which means jam concentration is infinity. The Drake et al model also results in a concave-shaped speed-concentration relationship in the low concentration range and a convex-shaped relationship in the high concentration range. It has the undesirable characteristic of not having an x-intercept resulting in a jam concentration of infinity. The non-integer m, l models tend to minimize or eliminate the undesirable features of the integer m, l models.

3.2.2 **MULTI-REGIME TRAFFIC FLOW MODELS:**

Multi-regime traffic flow models were developed in order to overcome the inadequacies of the single-regime models which represent the field data with reasonable accuracy in only one traffic flow regime while they tend to neglect the other regimes of the traffic flow pattern. Edie\textsuperscript{24} was the first to propose the idea of two-regime models in 1961 because of the difficulties experienced in using the single-regime traffic flow models. He proposed the use of the Underwood model for the free-flow regime and the Greenberg model for the congested regime. This model could be represented as:

**Free-flow regime:**

\[
\nu = 54.9e^{-\nu^{163.9}}
\]
if $k \leq 50$

Congested flow regime:

$$u = 26.8 \ln \left( \frac{162.5}{k} \right)$$

if $k \geq 50$

Where, 

$u = \text{speed}$

$k = \text{density}$

The development of the multi-regime models was continued by a research team at Northwestern University\textsuperscript{24} and three additional multi-regime model formulations were proposed. The first was the use of the Greenshields-type model for the free-flow regime and the congested flow regime separately. The equation for this model could be represented as:

Free-flow regime:

$$u = 60.9 - 0.515k$$

if $k \leq 65$
Congested flow regime:

\[ u = 40 - 0.265k \]

if \( k \geq 65 \)

Where, \( u \) = speed  
\( k \) = density

The second model proposed by this team suggested a constant speed model for the free-flow regime and a Greenberg model for the congested flow regime which could be represented as:

Free-flow regime:

\[ u = 48 \]

if \( k \leq 35 \)

Congested flow regime:

\[ u = 32 \ln \left( \frac{145.5}{k} \right) \]

if \( k \geq 35 \)
The last proposed model by the research team at Northwestern University was a three regime linear model with the free-flow regime, transitional flow regime and congested flow regime each being represented by the Greenshields model as follows:

**Free-flow regime:**

\[ u = 50 - 0.098k \]

if \( k \leq 40 \)

**Transitional flow regime:**

\[ u = 81.4 - 0.913k \]

if \( 40 \leq k \leq 65 \)

**Congested flow regime:**

\[ u = 40 - 0.265k \]

if \( k \geq 65 \)

Where, \( u = \) speed

\( k = \) density
TWO-REGIME TRAFFIC FLOW MODELS:

The investigation on single-regime models is extended to two-regime models to obtain improved representation of data sets, especially at near capacity levels of flow. The procedures used in the two-regime model evaluation were identical to those used in the single-regime model evaluation with a few exceptions. The free-flow speed and maximum flow criteria were removed for the congested flow regime. Moreover, only the data points with concentration values more than 50 vehicles per mile were included. For the free-flow regime, jam concentration criterion was removed and only data points with concentration values of less than 60 vpm were included.

CONGESTED-FLOW REGIME:

The criteria used in the selection of congested-flow regime models were mean deviation and jam concentration. The boundaries of the m, l combinations investigated were \(0 < m < 1\) and \(0 < l < 3.1\). The extended region in the m, l matrix for congested-flow regime is \(0 < m < 1\) and \(0 < l < 1\). The macroscopic equation for extended region is

\[
u^{1-m} = \alpha \frac{1-m}{1-l} (k^{l-1} - k^{l-1})
\]
FREE-FLOW REGIME:

The criteria used in selecting the free-flow regime models were mean deviation, free-flow speed and maximum flow. The boundaries of the m, l combinations investigated were \(0 < m < 1\) and \(0 < l < 3.1\). The Greenshields, Greenberg and Underwood models are evaluated and found that none of these macroscopic models appears to be appropriate.

The main difficulty in multi-regime traffic flow models was the determination of the breakpoint between regimes. The Northwestern research group applied the work of Quandt on likelihood functions to identify the breakpoints between regimes for all the four multi-regime models proposed. The attributes and characteristics of theses models are studied and compared by superimposing these equations and breakpoints on a freeway data set. All the multi-regime models tracked the freeway data set in a very reasonable manner and much better than any of the single regime traffic flow models. In regard to maximum flow, the Edie model slightly overestimates while the other models slightly underestimate. The Two-regime linear traffic flow model slightly overestimates the free-flow speed and the modified Greenberg model slightly underestimates the free-flow speed. The optimum speed is slightly overestimated by the Edie and three-regime linear traffic flow models. All the models underestimate the jam density.
In summary, the multi-regime models provide a considerable improvement over single regime models. However, the multi-regime and the single regime models had different strengths and weaknesses. Further each model discretely different than exhibiting as a continuous spectrum of a family of models.
4. SIMULATION BY DYNAMO

4.1 INTRODUCTION

DYNAMO is a computer program which compiles and executes continuous simulation models. Continuous models are useful when the system in question depends more on aggregate flows than on the occurrence of single, discrete events. An engineer is not concerned with the behavior of an individual molecule, but rather with the performance of the system as a whole. All of these problems can be studied by aggregating the individual events into a continuous flow and examining this flow in the context of the continuous variables that affect it and are affected by it. Typically, these variables form a closed loop feedback system, where $X$ affects $Y$ affects $Z$ affects $X$. These feedback systems exhibit behavior that cannot be predicted by looking at the components of the system in isolation.

DYNAMO is an effective tool for building and simulating continuous feedback models. It has been used to study business, social, economic, biological, psychological and engineering systems very widely. The basic tool of continuous simulation is the process of integration. Integration appears everywhere in nature and is essential to representing the process of changes in the real systems. Since digital computers cannot integrate exactly, integration in DYNAMO is approximated by a simple arithmetic process.
4.2 DYNAMO Equations

The equations for any model can be written by determining various factors such as the equation type of the quantity to be computed, the various parameters affecting this variable and the relation between these variables. There are seven types of equations in DYNAMO with five of them being used for variables. Two types of equations to represent constants are also used in DYNAMO. These are: (1) Given Constants (C), which can take only numerical values and (2) Table Statements (T), which are arrays of numerical values upon which the table look-up functions operate. The various equations of DYNAMO are:

Level Equations (L):

Level equations are the integral equations of DYNAMO. They relate a quantity at the current time to its value at the previous time that calculations were made, and to its rate of change during the interval between calculations.

Auxiliary Equations (A):

These are simple algebraic functions of levels and other auxiliary variables at the current time. Auxiliary equations may not depend upon other auxiliaries which in turn
depend on the auxiliary defined; i.e., simultaneous equations among auxiliary equations
are not permitted.

**Rate Equations (R):**

Rate equations are similar to auxiliary equations. They are algebraic functions
of levels and auxiliaries at the same time instant. The rate quantities are the flows in
conservative sub-systems.

**Supplementary Equations (S):**

Supplementary equations are algebraic equations that are computed only to
provide output. If there is a significant number of solution intervals between each output
period, a small saving in computer time can be made by computing as supplementaries
those quantities that are only printed or plotted, or are used only in other supplementary
equations.

**Initial Value Equations (N):**

These are algebraic equations that are computed only during model initialization.
An N statement must be provided for each level variable, and may be provided for an
auxiliary or rate variable. DYNAMO will use the active equations of uninitialized
auxiliaries or rates as initial value equations, if necessary. Constants may also be computed by initial value equations. The calculation would be made only once, and the N equation would be the only equation for the computed constant.

4.3 DYNAMO Conventions

Quantity names must be given a time subscript (timescript) to indicate the type of quantity and order of computation. Because of the inability of DYNAMO program to handle subscripts, time postscripts are used in defining the variables. Levels, auxiliaries and supplementaries have a single letter timescript while rates always have a double letter timescripts. The present time at which a level or auxiliary is calculated is defined as the time \( K \), and the forthcoming time interval for which a rate is calculated is defined as the time \( K \) to \( L \), the quantity being defined will always be given the timescripts \( K \) or \( KL \), depending on its quantity type. Quantities on the right hand side of the equal to sign may have timescripts \( J, K \), or \( JK \), depending on their own quantity type and the quantity type being defined. A summary of the DYNAMO postscript convention is shown in the table.

4.4 BUILT-IN FUNCTIONS

A function is a convenient shorthand for indicating that a particular mathematical process is to be performed. A number of functions are built into DYNAMO. In
addition to these, functions can be defined by using the macro facility and the external function facility. A macro is like a function, but is actually computed by one or more DYNAMO statements. The distinction between functions and macros is not important unless errors occur, because error messages relating to macros may include strange names. Of the built-in functions, the DELAY, SMOOTH, PULSE, RAMP, SAMPLE and some tables are actually macros. A function or macro is used on the right hand side of the equal sign as if it were a quantity name or number. The proper number of arguments must be provided in the parentheses after the function name, in the proper order, with commas to separate them. These arguments may be names or expressions. When using macros the equation type is determined by the nature of the variable being computed, not by the function. A few built-in functions are discussed here in order to give a good perspective of these functions.

DELAY AND SMOOTH FUNCTIONS

There are two classes of delays: material and information. The difference between these classes does not affect the numerical results unless the length of the delay is changing. In the material delay whatever is being delayed is considered a physical substance and is conserved as the length changes; in the information delay it is not. Delays can also be characterized by their "order", the number of levels through which the variable being delayed passes. The order determines how quickly the output starts
to change after a change in the input. Low-order delays respond more quickly than high-order delays having the same average delay time.

**DYNAMO** provides a first and a third order material delay and a third order information delay. The **SMOOTH** macro may be used as a first order information delay; the equations are identical. To access the total quantity contained in a third order material delay, **DELAYP** function can be used, whose third argument is that quantity. This third argument is an output argument and must not be defined elsewhere.

The **DELAY** functions have the forms:

- **DELAY1(IN,DEL)**
- **DELAY3(IN,DEL)**
- **DELAYP(IN,DEL,PPL)**
- **DLINF3(IN,DEL)**
- **SMOOTH(IN, DEL)**

Where,

- **IN** - is the input to the delay
- **DEL** - is the average length of delay
- **PPL** - is the amount in transit

The **SMOOTH** function provides a way to exponentially average a quantity. It is a built-in macro that besides smoothing the input, initializes the smoothed value to the initial input. This macro can also be used as a first order information delay.
LOGICAL FUNCTIONS

DYNAMO supports logical functions similar in nature to those available in higher level languages such as FORTRAN and PASCAL. These functions take the place of the conditional, or "IF", statements used in other computer languages to select between variables based on the result of a comparison or logical test.

CLIP FUNCTION

This function has two names. The name CLIP is the DYNAMO I name and describes one possible use of the function, clipping a variable depending on the value of two other variables. The second name, FIFGE, suggests how the function works. It selects the first argument IF the third is greater than or equal to the fourth. This function provides the equivalent of a "conditional branch" in general purpose computer languages, but it computes both alternatives before selecting the desired one.

CLIP/FIFGE has the form:

CLIP(A,B,C,D) or FIFGE(A,B,C,D)

and returns the value:

CLIP/FIFGE = A if C > D
CLIP/FIFGE = B if C < D

Where, A, B, C, D are the inputs to the function, but not necessarily distinct variables.
MAXIMUM AND MINIMUM FUNCTIONS

The MAXIMUM function sets the lower bounds by selecting the larger of the two variables. It has the form:

\[ \text{MAX(A,B)} \]

and returns the value:

\[ \text{MAX} = A \text{ if } A > B \]
\[ \text{MAX} = B \text{ if } A < B \]

The MINIMUM function sets the upper bounds by selecting the smaller of the two variables. This has the form:

\[ \text{MIN(A,B)} \]

and returns the value:

\[ \text{MIN} = A \text{ if } A > B \]
\[ \text{MIN} = B \text{ if } A < B \]

Where, A and B are the inputs to the function, but need not be different variables.

SWITCH FUNCTION

This function is used in order to select one function or the other based on a comparison of the inputs. This function is especially useful for testing two decision rules by means of reruns. Both decisions are included in the model, and the appropriate one
is chosen by the SWITCH function in each run or rerun. This function has the form:

\[ \text{SWITCH}(A, B, C) \]

and returns the value:

\[ \text{SWITCH} = A \text{ if } C = 0 \]

\[ \text{SWITCH} = B \text{ if } C = 0 \]

**TABLE FUNCTION**

A very important part of many simulation models is the use of tabular data, either empirically or theoretically derived. DYNAMO is able to handle data by means of several types of table functions. TABLE, TABXT, TABHL and TABPL are all invoked in the same manner though there are significant changes in their respective interpolating schemes. Two statements are required to describe a table function. The first one involves five ordered elements inside a parentheses to specify the dependent variable name, the independent variable name, the lowest, highest values of the independent variable and the interval between each independent variable value. A general definition of a table is given below:

\[ \text{TABHL (Y-variable, X-variable, low X, high X, X-interval)} \]

The main use of these table functions relies on the fact that every time the variable represented by the table function is invoked in the code the value of that variable will be interpolated or extrapolated from the known function values. TABLE, TABHL
and TABXT interpolate linearly between declared elements while TABPL performs cubic interpolation. TABHL upgrades TABLE by assigning the extreme point function values to the desired function if extrapolation is required. TABXT performs linear extrapolation at the desired value of the function. A last remark in the definition of the numerical values for TABPL is the fact that a zero must be added for each numerical value defined for the Y-variable.

The analyst must determine through logical relationships and statistical data analyses the values for constants and the multipliers which relate the rates to the levels. Once the model has been calibrated and validated, the DYNAMO model can be used for a variety of applications, including: forecasting and prediction, sensitivity analysis and testing of various scenarios.

**PULSE SOURCE FUNCTION**

The PULSE function has the form:

\[
PULSE(HGHT,FRST,INTVL)
\]

Where, \( HGHT \) - is the pulse height

\( FRST \) - is the TIME of the first pulse

\( INTVL \) - is the interval between pulses
PULSE provides a train of pulses which are of width DT and height HGHT. The first pulse appears at TIME = FRST, and subsequent pulses appear at TIME = FRST + 2*INTVL, FRST + 3*INTVL and so on. HGHT can be a variable, producing pulses of different heights.

RAMP FUNCTION

The RAMP function has the form:

RAMP(SLOPE,START)

and returns the value:

RAMP = 0 if TIME < START
RAMP = SUM(SLOPE*DT) if TIME > START

If SLOPE and START are both constants, RAMP is merely a ramp with SLOPE that starts at time START. If START is a variable, the ramp time is determined by the time interval. If SLOPE is a variable, RAMP is DT times the sum of all SLOPE since the time of START.

RANDOM NUMBER FUNCTIONS

DYNAMO generates random numbers computing a sequence of pseudo-random numbers. Pseudo-random numbers satisfy all statistical tests of randomness, but each
number is calculated from the previous one. There are two random-number functions in DYNAMO. The first:

\[ \text{NOISE()} \]
gives random numbers uniformly distributed between -1/2 and +1/2.

The second has the form:

\[ \text{NORMRN(MEAN,STDV)} \]

Where, MEAN - is the mean of the values returned

STDV - is their standard deviation

NORMRN gives random numbers normally distributed. NORMRN starts with a number from the same generator that NOISE uses, and then alters it to conform to a normal distribution.

**USER-DEFINED MACROS**

Modelers frequently discover that they must repeat a pattern of statements or expressions a number of different places in a model. The ability to devise a shorthand notation for such repeated structures would save the modeler time while constructing the model. Model readers would also benefit by being able to master the structure once and quickly recognize it wherever it is used. The function notation is such a shorthand; the macro feature gives modelers the ability to write functions using ordinary DYNAMO equations and reference them with the shorthand notation.
ARRAYS

DYNAMO provides modeler with the capability to write variable names with subscripts. Instead of simply writing WF.K to represent a workforce, for example, one can write WF.K(S,T), where S could represent the skill level of the work force and T the task to which it is assigned. S and T are called subscripts. Although the notation may initially appear cumbersome, there is a dramatic advantage being able to write variable names using subscripts. It means that one variable name can represent more than one quantity, and one equation can represent more than one element of model structure. The capability to write variables with subscripts enables the modeler to duplicate an entire sector of a model, with different parameters, any number of times without writing more equations. It allows complex, disaggregated models to build up from small, conceptually aggregate sectors.
5. TRAFFIC STREAM MODELING

5.1 TRAFFIC STREAM MODELS

Traffic Stream models provide the fundamental relationships of macroscopic traffic stream characteristics for Uninterrupted flow situations. The traffic stream characteristics include flow characteristics, speed characteristics and density characteristics. The relationships are for free-flow and congested flow conditions away from flow interruptions such as intersections.

Traffic stream models are used in planning, design and operations of transportation facilities. Travel times as a function of traffic load are needed in planning studies of traffic assignment and modal choice. The trade-off between levels of service and service flows are needed in design of new facilities and the redesign of existing ones. Operational control facilities requires the knowledge of relationships between on-line measurements of performance such as density or occupancy and optimum level of control. Although field derived empirical relationships are useful, expressing these relationships mathematically on a theoretical basis provides a sound generalized framework for understanding and application.

In this research, an effort is made to simulate the field conditions, and to analyze some of the existing traffic stream models. Most of the fundamental relationships of
macroscopic and microscopic traffic stream characteristics are formulated for uninterrupted flow situation. As most of the time, traffic stream is subjected to interruptions, it becomes imperative on us to analyze the traffic stream models, density, speed, flow, acceleration etc. are used as the key parameters.

In this research, an attempt is also made to depict congestion by studying the traffic flow patterns under various traffic flow conditions. All the traffic stream models that are used in this research are simulated for free-flow as well as congested flow regimes. The best model that is selected after analyzing all the stream models is used as a tool to depict congestion. This model is constructed in such a way that it can be used to measure various traffic stream parameters throughout the highway which is under study. By delineating those parameters through critical values, congestion could be depicted. The model also has the flexibility of incorporating any changes that are desired in the flow conditions by varying the parameters involved so that congestion could be depicted in a more realistic way in order to meet the requirement of the study zone under consideration.

The various traffic stream models that are used in this study include :-

a) **Single-Regime Models:**

- Greenshields model
- Greenberg model
- Underwood model
o North-Western group stream model

o Drew’s model

b) Multi-Regime Models: -

o Edie’s model

o Two-regime linear model

o Modified Greenberg model

o Three-regime linear model

Apart from these, generalized Non-linear car following model and Linear car following model are also used. In this research, simulation programs for all these models are developed which are explained in the next few articles.
5.2 Non-Linear Car Following Model

Car following theory assumes that the traffic stream is a superposition of vehicle pairs where each vehicle follows the vehicle ahead according to a stimulus-response equation that approximates the behavioral and mechanical aspects of the driver-vehicle-road system. A sensitivity factor is used in the non-linear car following model to represent the response lag of the man-machine system. The general equation for vehicle following is given by

\[ \dot{x}_{i+1}(t) = a_m \frac{\dot{x}_i(t-T) - \dot{x}_{i+1}(t-T)}{[x_i(t-T) - x_{i+1}(t-T)]^m} \]

Where,

- \( a_m \) = constant of proportionality.
- \( x_i \) = velocity of leading vehicle
- \( x_i \) = position of leading vehicle
- \( x_{i+1} \) = acceleration of following vehicle
- \( x_{i+1} \) = velocity of following vehicle
- \( x_{i+1} \) = position of following vehicle

A DYNAMO program is developed to simulate the traffic flow conditions that are represented by non-linear car following model. In this model, the flow of all the vehicles
is controlled by the flow pattern of the leading vehicle. The traffic stream could be controlled by developing a relation between the following vehicle and the leading vehicle. In this model this relationship could be expressed as headway and relative velocity. The headway of each vehicle is controlled in such a way that a minimum headway of 50 ft is maintained. The sensitivity of a vehicle is obtained from the ratio of sensitivity factor to the headway. The sensitivity factor is assumed as the ratio of free-flow speed to jam density. The acceleration of the vehicle could then be obtained by multiplication of the relative velocity and sensitivity.

The leading vehicle is given various speed characteristics in order to represent the various conditions imposed externally and internally. The leading vehicle is given speed characteristics in such a way that it decelerates to the speed limit imposed by a turning or a curve or any other such physical constraint. All the other vehicles in the traffic stream follow the lead vehicle as long as there is no change in the flow characteristics within the stream of vehicles. Due to the interrelationship that exists among this stream of vehicles, all the vehicles adjust to the speed with minor fluctuations which can be detected even in the field.

This model also simulates a condition where in all the vehicles of the traffic stream are stopped at signalized intersection. The vehicles are assumed to stop here for ten seconds before finally starting again. The simulation techniques are used to simulate the merging of some vehicles from another highway on to the main stream. This
merging is done in such a way that the gap is found after the second vehicle and the merging vehicles are released from the minor road as soon as the second vehicle crosses the point of merging.

This model can also be used to analyze the affects of blockage of a lane of the highway. The affects of congestion resulting from the blockage could be studied using various measures of congestion like density, speed, flow etc.

The non-linear car following model is used to study the various traffic stream parameters which influence the flow of the traffic. A plot is obtained between speed and time which shows the changes in speeds of the vehicles in traffic stream due to the existing traffic conditions. It is observed that the speeds of vehicles vary from a very low speed to free-flow speed which is an indication of the effectiveness of the model. The acceleration characteristics, distance travelled, headway and density of the traffic stream are also studied. The density of the traffic stream changes drastically due to change in the speeds of the vehicles. The traffic stream showed densities ranging from free-flow to congested flow conditions. This model is an effective model if the sensitivity parameters are determined accurately. This model also requires us to define the boundaries for determining the state of the traffic flow.
5.2.1 DYNAMO Program For Non-Linear Car Following Model

N  V11 = 0
A  V142.K = CLIP(V11.K, 0, TIME.K, 60)
R  A12.KL = 4
N  V131 = 95.355
R  A1.KL = (A-B*V0)*EXP(-B*TIME.K)
C  A = 4.0
C  B = 0.419485
C  V0 = 0
L  TIME.K = TIME.J + DT
N  TIME = 0
N  VF = 80
C  KJ = 120
C  DCF = 5280
C  VCF = 1.467
NOTE VCF - VELOCITY CONVERSION FACTOR
NOTE DCF - DISTANCE CONVERSION FACTOR
NOTE V1 - VELOCITY OF VEHICLE 1
R  PC1.KL = V1.K
N  X1 = 0
NOTE X1 - POSITION OF VEHICLE 1
NOTE PC1 - POSITION CHANGE OF VEHICLE 1
NOTE H2 - HEADWAY OF VEHICLE 2
A  S2.K = SF/H2.K**2
N  SF = VF*VCF/(KJ/DCF)
R  A2.KL = S2.K*RV2.K
NOTE S2 - SENSITIVITY OF VEHICLE 2
NOTE A2 - ACCELERATION OF VEHICLE 2
L  V2.K = V2.J + (DT)*A2.JK
N  V2 = 0
R  PC2.KL = V2.K
L  X2.K = X2.J + (DT)*PC2.JK
N  X2 = -150
NOTE RV2 - RELATIVE VELOCITY OF VEHICLE 2
NOTE PC2 - POSITION OF VEHICLE 2
NOTE X2 - POSITION OF VEHICLE 2
NOTE H3 - HEADWAY OF VEHICLE 3
NOTE S3 - SENSITIVITY OF VEHICLE 3
NOTE A3 - ACCELERATION OF VEHICLE 3
N   V3=0
N   X3=-300
NOTE RV3 - RELATIVE VELOCITY OF VEHICLE 3
NOTE PC3 - POSITION OF VEHICLE 3
NOTE X3 - POSITION OF VEHICLE 3
NOTE H4 - HEADWAY OF VEHICLE 4
NOTE S4 - SENSITIVITY OF VEHICLE 4
NOTE A4 - ACCELERATION OF VEHICLE 4
N   V4=0
R   PC4.KL=V4.K
N   X4=-450
NOTE RV4 - RELATIVE VELOCITY OF VEHICLE 4
NOTE PC4 - POSITION OF VEHICLE 4
NOTE X4 - POSITION OF VEHICLE 4
NOTE H5 - HEADWAY OF VEHICLE 5
A   S5.K=SF/H5.K**2
NOTE S5 - SENSITIVITY OF VEHICLE 5
NOTE A5 - ACCELERATION OF VEHICLE 5
N   V5=0
R   PC5.KL=V5.K
L   X5.K=X5.J+(DT)*PC5JK
N   X5=-600
NOTE RV5 - RELATIVE VELOCITY OF VEHICLE 5
NOTE PC5 - POSITION OF VEHICLE 5
NOTE X5 - POSITION OF VEHICLE 5
NOTE H6 - HEADWAY OF VEHICLE 6
NOTE S6 - SENSITIVITY OF VEHICLE 6
NOTE A6 - ACCELERATION OF VEHICLE 6
N   V6=0

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X6 = -.750

NOTE RV6 - RELATIVE VELOCITY OF VEHICLE 6
NOTE PC6 - POSITION OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6


NOTE H7 - HEADWAY OF VEHICLE 7
A S71.K = SF/H7.K**2

N V71 = 0

NOTE S7 - SENSITIVITY OF VEHICLE 7
NOTE A7 - ACCELERATION OF VEHICLE 7
NOTE V7 - VELOCITY OF VEHICLE 7

R PC7.KL = V7.K
N X7 = 3000

NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7

NOTE H8 - HEADWAY OF VEHICLE 8
A S81.K = SF/H8.K**2

N V81 = 0

NOTE S8 - SENSITIVITY OF VEHICLE 8
NOTE A8 - ACCELERATION OF VEHICLE 8
NOTE V8 - VELOCITY OF VEHICLE 8

R PC8.KL = V8.K
N X8 = 3300

NOTE PC8 - POSITION CHANGE OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8

NOTE H9 - HEADWAY OF VEHICLE 9


Traffic Stream Modeling
<table>
<thead>
<tr>
<th>Command</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>V91=0</td>
</tr>
<tr>
<td>NOTE</td>
<td>S9 - SENSITIVITY OF VEHICLE 9</td>
</tr>
<tr>
<td>NOTE</td>
<td>A8 - ACCELERATION OF VEHICLE 9</td>
</tr>
<tr>
<td>NOTE</td>
<td>V8 - VELOCITY OF VEHICLE 9</td>
</tr>
<tr>
<td>N</td>
<td>X9=3600</td>
</tr>
<tr>
<td>NOTE</td>
<td>PC9 - POSITION CHANGE OF VEHICLE 9</td>
</tr>
<tr>
<td>NOTE</td>
<td>X9 - POSITION OF VEHICLE 9</td>
</tr>
<tr>
<td>SPEC</td>
<td>DT=0.1/LENGTH=150/PLTPER=1</td>
</tr>
<tr>
<td>PLOT</td>
<td>V1=1,V2=2,V3=3,V4=4,V5=5,V6=6,V7=7,V8=8,V9=9(<em>,</em>)</td>
</tr>
<tr>
<td>RUN</td>
<td></td>
</tr>
<tr>
<td>QUIT</td>
<td></td>
</tr>
</tbody>
</table>
5.3 Linear Car Following Model

The car following laws are simplified descriptions of a very complicated response to the world of stimuli that confronts a driver. In fact, only two stimuli are considered: the relative speed between a vehicle and the one ahead and the spacing between the two vehicles. The progress of a stream of traffic for certain special cases could be studied mathematically by using linear car following model. However, if a complete study of the linear car following theory is desired based on complicated velocity functions of the leading vehicle, the analytical work may soon become formidable. In order to solve problems involving realistic speed changes of the leading vehicle and realistic car following models, the problem can be solved by using simulation. The simulation can be done based on a deterministic model of a vehicle operator, that represent a line of vehicles in various traffic situations. The principle use of simulation is to study the traffic as it relates to the individual vehicle operator and as it depends upon group dynamics. A second use to which the simulation may be applied is that of generating traffic flow data to supply intermediate data points to that actually observed.

A linear car following model of traffic flow could be described by the differential equation:

$$\dot{v}_i(t) = k_i [v_{i-1}(t-T) - v_i(t-T)]$$

Where,  
$v_i = \text{acceleration of following vehicle}$  
$k_i = \text{density}$
\[ v_i = \text{velocity of following vehicle} \]
\[ v_{i-1} = \text{velocity of leading vehicle} \]

The progress of traffic stream is represented by the linear car following model, in the DYNAMO program for the flow of traffic wherein, the initial speed of all the vehicles at the time of beginning of simulation is assumed to be zero. The speed of the lead vehicle is given a very complex function in order to represent all the practical conditions that have a bearing on the speed of the traffic stream. The lead vehicle is assumed to have non linear acceleration characteristics while the deceleration is assumed to be constant. The non-linear acceleration could be simulated in this model by defining TIME variable. The acceleration of the following vehicle could be calculated as the ratio of the relative velocity to headway. The speed and hence the distance travelled by the following vehicle are controlled by the speed of the lead vehicle and the spacing between the vehicles as these are the two parameters involved in defining acceleration.

In this model the minimum headway is assumed to be 50 ft and it is also assumed that the vehicles try to maintain this atleast this headway whenever the flow of traffic stream is required to be paced up. This model is developed to simulate various realistic traffic conditions like low speed limits, intersection signals, temporary blockage of a lane of highway and merging of vehicles from another highway into the main stream of traffic. This model is also studied to analyze the affect of the maximum speed limit on the traffic flow.
The results (fig. 3) of this simulation are analyzed to study the traffic flow pattern at various speeds and densities. This model shows that it requires a longer time for the entire stream of vehicles to reach the free-flow speed as compared to a few other models. As long as the speed of the traffic stream is less, the headway between vehicles will be very less and this shows an apparent high density indicating the formation of congestion. The density curve is also analyzed for similar phenomenon and it is observed that the density of traffic as represented by this model is high. As a result of low headways the acceleration of the individual vehicles is more leading to more acceleration noise which in turn reduces the speed of the traffic stream. This model can be used in certain special cases only as the flow characteristics resulting from this model are very conservative.

5.3.1 **DYNAMO Program For Linear Car Following Model**

```
N     V11=0
A     V142.K=CLIP(V11.K,0,TIME.K,60)
R     A12.KL=4
N     V131=95.355
R     A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C     A=4.0
C     B=0.419485
C     V0=0
L     TIME.K=TIME.J+DT
N     TIME=0
N     VF=80
C     KJ=120
```
C
NOTE VCF = 1.467
NOTE VCF - VELOCITY CONVERSION FACTOR
NOTE V1 - VELOCITY OF VEHICLE 1
R
PC1.KL = V1.K
NOTE PC1 - POSITION CHANGE OF VEHICLE 1
L
NOTE X1 - POSITION OF VEHICLE 1
N
X1 = 0
A
NOTE H2 - HEADWAY OF VEHICLE 2
A
NOTE RV2 - RELATIVE VELOCITY OF VEHICLE 2
R
A2.KL = RV2.K/H2.K
NOTE A2 - ACCELERATION OF VEHICLE 2
L
V2.K = V2.J + (DT)*A2.JK
NOTE V2 - VELOCITY OF VEHICLE 2
N
V2 = 0
R
PC2.KL = V2.K
NOTE PC2 - POSITION CHANGE OF VEHICLE 2
L
X2.K = X2.J + (DT)*PC2.JK
NOTE X2 - POSITION OF VEHICLE 2
N
X2 = -50
A
NOTE H3 - HEADWAY OF VEHICLE 3
A
NOTE RV3 - RELATIVE VELOCITY OF VEHICLE 3
R
NOTE A3 - ACCELERATION OF VEHICLE 3
L
NOTE V3 - VELOCITY OF VEHICLE 3
N
V3 = 0
R
NOTE PC3 - POSITION CHANGE OF VEHICLE 3
L
NOTE X3 - POSITION OF VEHICLE 3
N
X3 = -100
A
NOTE H4 - HEADWAY OF VEHICLE 4
A
NOTE RV4 - RELATIVE VELOCITY OF VEHICLE 4
R
NOTE A4 - ACCELERATION OF VEHICLE 4
L
NOTE V4 - VELOCITY OF VEHICLE 4
N
V4 = 0
R
PC4.KL = V4.K
NOTE PC4 - POSITION CHANGE OF VEHICLE 4
L
NOTE X4 - POSITION OF VEHICLE 4
N
X4 = -150
A
NOTE H5 - HEADWAY OF VEHICLE 5
NOTE RV5 - RELATIVE VELOCITY OF VEHICLE 5
NOTE A5 - ACCELERATION OF VEHICLE 5
NOTE V5 - VELOCITY OF VEHICLE 5
N V5=0
R PC5.KL=V5.K
NOTE PC5 - POSITION CHANGE OF VEHICLE 5
L X5.K=X5.J+(DT)*PC5.JK
NOTE X5 - POSITION OF VEHICLE 5
N X5=-200
NOTE H6 - HEADWAY OF VEHICLE 6
NOTE RV6 - RELATIVE VELOCITY OF VEHICLE 6
NOTE A6 - ACCELERATION OF VEHICLE 6
NOTE V6 - VELOCITY OF VEHICLE 6
N V6=0
NOTE PC6 - POSITION CHANGE OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6
N X6=-250
NOTE H7 - HEADWAY OF VEHICLE 7
R A7.KL=CLIP(A71.JK,A72,X2.K,3050)
NOTE A7 - ACCELERATION OF VEHICLE 7
C A72=0
NOTE V7 - VELOCITY OF VEHICLE 7
N V71=0
R PC7.KL=V7.K
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7
N X7=3000
NOTE H8 - HEADWAY OF VEHICLE 8
NOTE A8 - ACCELERATION OF VEHICLE 8
C A82=0

Traffic Stream Modeling
NOTE V8 - VELOCITY OF VEHICLE 8
N V81 = 0
R PC8.KL = V8.K
NOTE PC8 - POSITION CHANGE OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8
N X8 = 3300
NOTE H9 - HEADWAY OF VEHICLE 9
NOTE A9 - ACCELERATION OF VEHICLE 9
C A92 = 0
NOTE V9 - VELOCITY OF VEHICLE 9
N V91 = 0
NOTE PC9 - POSITION CHANGE OF VEHICLE 9
NOTE X9 - POSITION OF VEHICLE 9
N X9 = 3600
SPEC DT = 0.1/LENGTH = 160/PLTPER = 1
PLOT V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9 (*, *)
RUN
QUIT
Figure 3. Time-Space diagram for linear car following model
5.4 Greenshields Model

Greenshields model is the first single-regime model developed in 1934, based on observations made by measuring speed and density using aerial photographic study. Greenshields model describes a linear function between speed and density. The model requires the knowledge of the free-flow speed and jam density parameters in order to solve numerically for the speed-density relationship. The free-flow speed is relatively easy to estimate in the field and generally lies between the speed limit and the design speed of the roadway. On the other hand, the estimation of jam density is difficult. Jam density values are difficult to obtain in the field, but a general value of 185 to 250 vehicles per mile per lane can be calculated assuming that a stopped vehicle occupies 21 to 28 ft of the roadway space. Greenshields model can be expressed as

\[ u = u_r (1 - \frac{k}{k_j}) \]

Where, \( u \) = speed of vehicle
\( k \) = density
\( u_r \) = free-flow speed
\( k_j \) = jam density
This model could be analyzed for its effectiveness in representing traffic stream characteristics by developing a simulation model using DYNAMO procedures. As this model is developed for field data, it can be very easily simulated to represent field data. In this model also, the lead vehicle is given a comprehensive velocity function, which represents the traffic flow conditions that are considered in the study. The speed of the traffic stream is controlled by the headway between successive vehicles as headway affects density. The only parameter that causes the variations in speed, once jam density and free-flow speed are determined, is the density of traffic stream for this model. The velocity and headway of all the vehicles in the traffic stream are controlled in such a way that whenever there is an increase in the headway the velocity of the following vehicle increases and as the headway decreases the velocity also decreases. Thus, the stream will have a motion which duplicates the realistic traffic flow and it can also be observed from this model that the paths of two vehicles never cross each other.

In this model an attempt is made to simulate the flow of traffic on a curve, the characteristics at a signalized intersection on red light, sudden blockage of the highway, the affect of speed limit, merging of traffic from a minor highway into the mainstream traffic and also the speed of vehicles merging into the main stream traffic. This model is analyzed to study the variations in traffic flow pattern due to changes in the free-flow speed and jam density values. According to this model the velocity of the traffic stream reaches zero at jam density which is not represented by a few other models.

Traffic Stream Modeling
This model could be analyzed by studying the velocity-time plot obtained through simulation. This plot shows that when all the vehicles start from zero speed, the acceleration pattern for each vehicle is different from that of the other vehicles. The time required to attain maximum speed and hence the total travel time are also different from the values that are expected. When the vehicles are required to reduce their speed, then also the deceleration patterns are different for all the vehicles. It is observed from the density plot that the density of traffic stream varies a lot due to variations in speed. It is also observed that sometimes the density is approaching jam density due to very low speeds. This could be used as a good model depicting congestion but for the fact that this model can represent only a single regime of the traffic flow pattern accurately. The problem with this model the use of jam density value as according to this model, optimum density is equal to one-half of the jam density value. This is not compatible with the observed optimum density values on the order of 40 to 70 vehicles per lane per mile.

5.4.1 DYNAMO Program For Greenshields Model

\[
\begin{align*}
L & \quad V11.K = V11.j + (DT) * A1.JK \\
N & \quad V11 = 0 \\
A & \quad V142.K = CLIP(V11.K, 0, TIME.K, 60) \\
R & \quad A12.KL = 4
\end{align*}
\]
N

R

C

C

C

L

N

N

C

C

C

NOTE

NOTE

NOTE

NOTE

R

L

N

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

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N

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NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

A

A

L

N

R

NOTE

NOTE

NOTE

NOTE

A

V131 = 95.355
A1.KL = (A - B*V0)*EXP(-B*TIME.K)
A = 4.0
B = 0.419485
V0 = 0
TIME.K = TIME.J + DT
TIME = 0
VF = 80
KJ = 120
DCF = 5280
VCF = 1.467
NOTE

VCF - VELOCITY CONVERSION FACTOR
NOTE

DCF - DISTANCE CONVERSION FACTOR
NOTE

KJ - JAM DENSITY
NOTE

V1 - VELOCITY OF VEHICLE 1
PC1.KL = V1.K
X1 = 0
NOTE

X1 - POSITION OF VEHICLE 1
NOTE

PC1 - POSITION CHANGE OF VEHICLE 1
V2.K = VF*VCF*(1 - (K2.K*DCF/KJ))
K2.K = 1/H2.K
X2.K = X2.J + (DT)*PC2.JK
X2 = -100
PC2.KL = V2.K
NOTE

H2 - HEADWAY OF VEHICLE 2
NOTE

V2 - VELOCITY OF VEHICLE 2
NOTE

PC2 - POSITION CHANGE OF VEHICLE 2
X3 = -200
NOTE

H3 - HEADWAY OF VEHICLE 3
NOTE

V3 - VELOCITY OF VEHICLE 3
NOTE

PC3 - POSITION CHANGE OF VEHICLE 3
X4 = -300
PC4.KL = V4.K
NOTE

H4 - HEADWAY OF VEHICLE 4
NOTE

V4 - VELOCITY OF VEHICLE 4
NOTE

PC4 - POSITION CHANGE OF VEHICLE 4
NOTE

X4 - POSITION OF VEHICLE 4
Traffic Stream Modeling
C
V52=0
A
A
A
K5.K=1/H5.K
L
X5.K=X5.J+(DT)*PC5.JK
N
X5=-400
R
PC5.KL=V5.K

NOTE
H5 - HEADWAY OF VEHICLE 5
NOTE
V5 - VELOCITY OF VEHICLE 5
NOTE
PC5 - POSITION CHANGE OF VEHICLE 5
NOTE
X5 - POSITION OF VEHICLE 5
A
A
A
L
N
X6=-500
R

NOTE
H6 - HEADWAY OF VEHICLE 6
NOTE
V6 - VELOCITY OF VEHICLE 6
NOTE
PC6 - POSITION CHANGE OF VEHICLE 6
NOTE
X6 - POSITION OF VEHICLE 6
A
A
A
A
L
N
X7=3000
R
PC7.KL=V7.K

NOTE
H7 - HEADWAY OF VEHICLE 7
NOTE
V7 - VELOCITY OF VEHICLE 7
NOTE
PC7 - POSITION CHANGE OF VEHICLE 7
NOTE
X7 - POSITION OF VEHICLE 7
A
A
A
A
L
N
X8=3300
R
PC8.KL=V8.K

NOTE
H8 - HEADWAY OF VEHICLE 8
NOTE
V8 - VELOCITY OF VEHICLE 8
NOTE
PC8 - POSITION CHANGE OF VEHICLE 8
NOTE
X8 - POSITION OF VEHICLE 8
A
A
A
A
L
N
X9=3600
R

NOTE
H9 - HEADWAY OF VEHICLE 9
NOTE       V9 - VELOCITY OF VEHICLE 9
NOTE       PC9 - POSITION CHANGE OF VEHICLE 9
NOTE       X9 - POSITION OF VEHICLE 9
SPEC       DT=0.1/LENGTH=150/PLT?PER=1
PLOT       V1=1,V2=2,V3=3,V4=4,V5=5,V6=6,V7=7,V8=8,V9=9(*,*)
RUN
QUIT
Figure 4. Time-Space diagram for Greenshields model.
5.5 Greenberg Model

The Greenberg model is the second single-regime model that was developed by observing speed-density data set for tunnels, with particular attention to congested portions. This is a non linear model developed by using hydrodynamic analogy and combining the equations of motion and continuity for one-dimensional compressible flow. One of the important results of Greenberg model was the bridge that was discovered between this proposed macroscopic model and the third General Motors car following model. This bridge was the foundation for later discovery that almost all developed car following theories could be related mathematically to most macroscopic traffic stream models. The Greenberg model requires the knowledge of the optimum speed and jam density parameters. This model has the difficulty of estimating optimum speed nd jam density. A crude estimate is that the optimum speed is approximately one-half of the design speed, as it is even more difficult to estimate optimum speed than free-flow speed.

Greenberg model can be represented as

\[ u = u_o \ln \left( \frac{\xi}{k} \right) \]

Where,

- \( u \) = speed of vehicle
- \( k \) = density of vehicle
- \( u_o \) = optimum speed
- \( \xi \) = jam density
A realistic traffic stream model is developed by using Greenberg model and simulating the various road conditions. The average speed of the traffic stream in this model, depends to a large extent on the headways of the vehicles once optimum speed and jam density are determined. The velocity of vehicles varies as the natural logarithmic function of the density of traffic stream which could be controlled by controlling the speed of the traffic stream. It can be observed from the DYNAMO model that the higher the speed of the vehicles in the stream, the lower is the density of the stream. The speed of the vehicles decreases as the density increases which is obvious in realistic field conditions.

In this model an attempt is made to simulate the various traffic flow conditions like low headways, decreased acceleration, slow moving leading vehicle, progress of traffic on a curve, signalized intersection, blocked road and merging of other vehicles into mainstream and nonlinear acceleration characteristics to the leading vehicle.

The results of this simulation model are interpreted to observe the various speed patterns that exist in realistic traffic flow conditions. This model shows that the speed of vehicles has an essential impact on the density of the traffic stream. It can be observed from the density curve that due to a change in the speed the density keeps on increasing until the normal speed is restored again. This is observed when the vehicles
are slowed down at the curves and also at the intersections. This model is very sensitive to variations in the traffic flow parameters. This model is very efficient in describing the traffic flow in congested conditions. A disadvantage of this model is that the free-flow speed is infinity which is highly unrealistic. This model might probably be used in combination with some other model to depict congestion as this model describes the traffic flow quite efficiently once congestion sets in.

5.5.1 DYNAMO Program For Greenberg Model

A
A
L
N
V11=0
A
A
A
V142.K=CLIP(V11.K,0,TIME.K,60)
A
L
R
A12.KL=4
N
V131=95.355
R
A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C
A=4.0
C
B=0.419485
C
V0=0
L
TIME.K=TIME.J+DT
N
TIME=0
N
V0=65
C
KJ=120
C
dCF=5280
C
VCF=1.467
NOTE
VCF - VELOCITY CONVERSION FACTOR
NOTE
DCF - DISTANCE CONVERSION FACTOR
NOTE
KJ - JAM DENSITY
NOTE
VO - OPTIMUM SPEED
NOTE
V1 - VELOCITY OF VEHICLE 1
R
PC1.KL=V1.K
L
N
X1=0
NOTE
X1 - POSITION OF VEHICLE 1
NOTE
PC1 - POSITION CHANGE OF VEHICLE 1
A \quad V2.K=(V0*VCF*\text{LOGN}(KJ/(DCF*K2.K)))/2
A \quad K2.K=1/H2.K
L \quad X2.K=X2.J+(DT)*PC2.JK
N \quad X2=-100
R \quad PC2.KL=V2.K
NOTE \quad H2 - HEADWAY OF VEHICLE 2
NOTE \quad V2 - VELOCITY OF VEHICLE 2
NOTE \quad PC2 - POSITION CHANGE OF VEHICLE 2
NOTE \quad X2 - POSITION OF VEHICLE 2
A \quad V3.K=V0*VCF*\text{LOGN}(KJ/(DCF*K2.K))/2
N \quad X3=-200
R \quad PC3.KL=V3.K
NOTE \quad H3 - HEADWAY OF VEHICLE 3
NOTE \quad V3 - VELOCITY OF VEHICLE 3
NOTE \quad PC3 - POSITION CHANGE OF VEHICLE 3
NOTE \quad X3 - POSITION OF VEHICLE 3
A \quad V4.K=V0*VCF*\text{LOGN}(KJ/(DCF*K4.K))/2
A \quad K4.K=1/H4.K
L \quad X4.K=X4.J+(DT)*PC4.JK
N \quad X4=-300
R \quad PC4.KL=V4.K
NOTE \quad H4 - HEADWAY OF VEHICLE 4
NOTE \quad V4 - VELOCITY OF VEHICLE 4
NOTE \quad PC4 - POSITION CHANGE OF VEHICLE 4
NOTE \quad X4 - POSITION OF VEHICLE 4
C \quad V52=0
A \quad V51.K=V0*VCF*\text{LOGN}(KJ/(DCF*K5.K))/2
A \quad K5.K=1/H5.K
L \quad X5.K=X5.J+(DT)*PC5.JK
N \quad X5=-400
R \quad PC5.KL=V5.K
NOTE \quad H5 - HEADWAY OF VEHICLE 5
NOTE \quad V5 - VELOCITY OF VEHICLE 5
NOTE \quad PC5 - POSITION CHANGE OF VEHICLE 5
NOTE \quad X5 - POSITION OF VEHICLE 5
A \quad V6.K=V0*VCF*\text{LOGN}(KJ/(DCF*K6.K))/2
N \quad X6=-500
R \quad PC6.KL=V6.K
NOTE \quad H6 - HEADWAY OF VEHICLE 6
NOTE \quad V6 - VELOCITY OF VEHICLE 6
NOTE \quad PC6 - POSITION CHANGE OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6
N X7 = 3000
R PC7.KL = V7.K
NOTE H7 - HEADWAY OF VEHICLE 7
NOTE V7 - VELOCITY OF VEHICLE 7
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7
N X8 = 3300
R PC8.KL = V8.K
NOTE H8 - HEADWAY OF VEHICLE 8
NOTE V8 - VELOCITY OF VEHICLE 8
NOTE PC8 - POSITION CHANGE OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8
N X9 = 3600
NOTE H9 - HEADWAY OF VEHICLE 9
NOTE V9 - VELOCITY OF VEHICLE 9
NOTE PC9 - POSITION CHANGE OF VEHICLE 9
NOTE X9 - POSITION OF VEHICLE 9
SPEC DT = 0.1/LENGTH = 150/PLT/PERS = 1
PLOT V1 = 1,V2 = 2,V3 = 3,V4 = 4,V5 = 5,V6 = 6,V7 = 7,V8 = 8,V9 = 9(*,*)
RUN
QUIT
Figure 5. Time-Space diagram for Greenberg model
5.6 Underwood Model

Underwood model is developed as a result of traffic studies on the Meritt Parkway in Connecticut, and focuses its attention on the free-flow regime. This model has overcome the disadvantage of Greenberg model wherein, the free-flow speed goes to infinity. This model requires knowledge of the free-flow speed and the optimum density. The free-flow speed could be easily determined in the field, but optimum density is very difficult to observe and varies depending on the roadway environment. Underwood model may be represented as

\[ u = u_f e^{-\frac{k}{k_o}} \]

Where,

- \( u \) = speed of vehicle
- \( k \) = density
- \( k_o \) = optimum density
- \( u_f \) = free-flow speed

The Underwood model expresses an exponential relationship between speed of vehicles and the traffic stream density which could be easily analyzed by simulating this model for various realistic traffic conditions. The speed of various vehicles in the traffic stream varies continuously due to variations in the headways of vehicles resulting from relative motion of these vehicles. The density of traffic stream varies continuously due
to change in the position of vehicles once the simulation is started. The speed of the vehicles is represented as a rate variable in this model which affects the position of the vehicle thereby resulting in change in headway. Hence, as the velocity changes, the headway also varies and this process continues until the traffic stream attains stability wherein, the speed of the traffic stream remains constant.

This model could be simulated by giving the lead vehicle a speed function in such a way that this function takes care of all the changing conditions in traffic stream. The leading car is assumed to have a non-linear acceleration while its deceleration rate is constant. The leading car is given the speed limits for curves and also for other interruptions. All the remaining vehicles follow the leading vehicle in speed characteristics also.

This model is analyzed for high density traffic flows, zero starting speed, blockage of road, low speed limits, interruptions like traffic signals. All these characteristics help to build a model which can analyze realistic conditions applied to this model. This model doesn’t represent the case wherein a lane of highway is suddenly blocked because, the velocity cannot become zero according to this model thereby showing crossing paths of the vehicles which is unrealistic.

The results of this simulation indicate that this model has a drawback as this model doesn’t allow the speed of vehicles to approach zero which is not practicable. In
the case of interruptions caused by signals, this model cannot be used as the speed doesn’t become zero. The acceleration characteristics of the traffic stream according to this model is pretty uniform. The density parameter also causes a problem, as the jam density in this model goes to infinity. This model also has the problem of determining the optimum density. This model is totally focused on free-flow regime and can be affectively used in this regime only. This model alone cannot be used to depict congestion.

5.6.1 DYNAMO Program For Underwood Model

N     V11=0
A     V142.K=CLIP(V11.K,0,TIME.K,60)
R     A12.KL=4
N     V131=95.355
R     A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C     A=4.0
C     B=0.419485
C     V0=0
L     TIME.K=TIME.J+DT
N     TIME=0
N     VF=80
C     KO=60
C     DCF=5280
C     VCF=1.467
NOTE  VCF - VELOCITY CONVERSION FACTOR
NOTE  DCF - DISTANCE CONVERSION FACTOR
NOTE  KO - OPTIMUM DENSITY
NOTE  VF - FREE-FLOW SPEED
NOTE  V1 - VELOCITY OF VEHICLE 1
PC1.KL = V1.K
X1 = 0

NOTE
X1 - POSITION OF VEHICLE 1
NOTE
PC1 - POSITION CHANGE OF VEHICLE 1
A
V2.K = VF*VCF*EXP(-K2.K*DCF/KO)
A
A
K2.K = 1/H2.K
L
X2.K = X2.J + (DT)*PC2.JK
N
X2 = -100
R
PC2.KL = V2.K

NOTE
H2 - HEADWAY OF VEHICLE 2
NOTE
V2 - VELOCITY OF VEHICLE 2
NOTE
PC2 - POSITION CHANGE OF VEHICLE 2
NOTE
X2 - POSITION OF VEHICLE 2
A
A
A
L
N
X3 = -200
R

NOTE
H3 - HEADWAY OF VEHICLE 3
NOTE
V3 - VELOCITY OF VEHICLE 3
NOTE
PC3 - POSITION CHANGE OF VEHICLE 3
NOTE
X3 - POSITION OF VEHICLE 3
A
A
A
L
N
X4 = -300
R
PC4.KL = V4.K

NOTE
H4 - HEADWAY OF VEHICLE 4
NOTE
V4 - VELOCITY OF VEHICLE 4
NOTE
PC4 - POSITION CHANGE OF VEHICLE 4
NOTE
X4 - POSITION OF VEHICLE 4
A
A
A
K5.K = 1/H5.K
L
X5.K = X5.J + (DT)*PC5.JK
N
X5 = -400
R
PC5.KL = V5.K

NOTE
H5 - HEADWAY OF VEHICLE 5
NOTE
V5 - VELOCITY OF VEHICLE 5
NOTE
PC5 - POSITION CHANGE OF VEHICLE 5
NOTE
X5 - POSITION OF VEHICLE 5
A
A
A
L
N
X6 = -500
R

Traffic Stream Modeling
H6 - HEADWAY OF VEHICLE 6
V6 - VELOCITY OF VEHICLE 6
PC6 - POSITION CHANGE OF VEHICLE 6
X6 - POSITION OF VEHICLE 6

A
A
A
A
L
N
X7 = 3000
R
PC7.KL = V7.K

H7 - HEADWAY OF VEHICLE 7
V7 - VELOCITY OF VEHICLE 7
PC7 - POSITION CHANGE OF VEHICLE 7
X7 - POSITION OF VEHICLE 7

A
A
V81.K = VF*VCF*EXP(-K8.K*DCF/KO)
A
A
L
N
X8 = 3300
R
PC8.KL = V8.K

H8 - HEADWAY OF VEHICLE 8
V8 - VELOCITY OF VEHICLE 8
PC8 - POSITION CHANGE OF VEHICLE 8
X8 - POSITION OF VEHICLE 8

A
A
A
A
L
N
X9 = 3600
R

H9 - HEADWAY OF VEHICLE 9
V9 - VELOCITY OF VEHICLE 9
PC9 - POSITION CHANGE OF VEHICLE 9
X9 - POSITION OF VEHICLE 9

DT = 0.1/LENGTH = 150/PLTPER = 1

V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9 (*, *)

RUN
QUIT
Figure 6. Time-Space diagram for Underwood model
5.7 Northwestern Group Model

The analysis of flow-speed-density relationships based on field measurements is a very difficult task. Unique demand-capacity relationships over time of day and over length of roadway must be present. Even then the complete range of flow speed and density values will probably not be recorded. Parameter values of flow, speed and density are often difficult to estimate and can greatly vary between sites. Many other factors affect flow-speed-density relationships, such as design speed, access control, presence of trucks, speed limits, number of lanes and so on. A group of researchers at Northwestern University observed that most speed-density curves appear as S-shaped curves and proposed the Northwestern model which is a development over Underwood model in the aspect of relation between speed of vehicle and density of traffic stream. This model also requires the knowledge of the free-flow speed and optimum density. The Northwestern model is represented as follows:

\[ u = u_f e^{-\frac{1}{2} \left( \frac{k}{k_o} \right)^2} \]

Where, 

- \( u \) = speed of vehicle
- \( k \) = density
- \( u_f \) = free-flow speed
- \( k_o \) = optimum density
The relationship between various vehicles in the traffic stream is obtained by relating the vehicles with the headway between them. As the Northwestern group model has the density parameter in it in the determination of velocity, the flow of traffic could be regulated by controlling the leading vehicle. The leading vehicle is defined by a complicated function which necessarily takes into account all the practical considerations that are faced by any traffic stream. As all the vehicles are interrelated, the characteristics of the leading vehicle are propagated throughout the traffic stream with various degrees of sensitivity. It can be observed from the model, which represents speed as an exponential function of density, that as headway increases the speed increases due to decrease in the density. The speed varies continuously as changing speed causes changes in headways which in turn affect speed. This process continues until stability is achieved when the speed of traffic stream is more or less uniform.

Northwestern model essentially retains all the drawbacks of Underwood model although it represents the speed-density relationship in a more realistic way. The main drawback of this model is that it can never allow speed to approach zero which is undesirable. As a result of this, this model cannot be used to represent the traffic flow where there is a possibility of the stream vehicles coming to rest due to interruptions. Another major disadvantage of this model is that the jam density is infinity which is impossible. The accurate determination of jam density is necessary in order to evaluate the congestive or free-flow conditions that are existing in the field.
This model also is oriented to represent free-flow regime only and hence cannot be used as an individual model in the studies for congestion. This model uses optimum density in calculating the speed of the vehicles which is very difficult to determine and is often inaccurately calculated. This leads to inaccurate results. It might also lead to over estimate the speeds of vehicles in the traffic stream.

5.7.1 DYNAMO Program For Northwestern Group Model

N   V11=0
A   V142.K=CLIP(V11.K,0,TIME.K,60)
R   A12.KL=4
N   V131=95.355
R   A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C   A=4.0
C   B=0.419485
C   V0=0
L   TIME.K=TIME.J+DT
N   TIME=0
N   VO=65
C   KJ=120
C   DCF=5280
C   VCF=1.467
NOTE VCF - VELOCITY CONVERSION FACTOR
NOTE DCF - DISTANCE CONVERSION FACTOR
NOTE KJ - JAM DENSITY
NOTE VO - OPTIMUM SPEED
NOTE V1 - VELOCITY OF VEHICLE 1
R   PC1.KL=V1.K
N   X1=0
NOTE X1 - POSITION OF VEHICLE 1
NOTE PC1 - POSITION CHANGE OF VEHICLE 1
A \ V2.K=(V0*VCF*EXP(-0.5*(K2.K*DCF/KJ)**2)
A \ K2.K=1/H2.K
L \ X2.K=X2.J+(DT)*PC2.JK
N \ X2=-100
R \ PC2.KL=V2.K
NOTE \ H2 - HEADWAY OF VEHICLE 2
NOTE \ V2 - VELOCITY OF VEHICLE 2
NOTE \ PC2 - POSITION CHANGE OF VEHICLE 2
NOTE \ X2 - POSITION OF VEHICLE 2
A \ V3.K=(V0*VCF*EXP(-0.5*(K3.K*DCF/KJ)**2)
N \ X3=-200
R \ PC3.KL=V3.K
NOTE \ H3 - HEADWAY OF VEHICLE 3
NOTE \ V3 - VELOCITY OF VEHICLE 3
NOTE \ PC3 - POSITION CHANGE OF VEHICLE 3
NOTE \ X3 - POSITION OF VEHICLE 3
A \ V4.K=(V0*VCF*EXP(-0.5*(K4.K*DCF/KJ)**2)
A \ K4.K=1/H4.K
N \ X4=-300
R \ PC4.KL=V4.K
NOTE \ H4 - HEADWAY OF VEHICLE 4
NOTE \ V4 - VELOCITY OF VEHICLE 4
NOTE \ PC4 - POSITION CHANGE OF VEHICLE 4
NOTE \ X4 - POSITION OF VEHICLE 4
C \ V52=0
A \ V51.K=(V0*VCF*EXP(-0.5*(K5.K*DCF/KJ)**2)
A \ K5.K=1/H5.K
L \ X5.K=X5.J+(DT)*PC5.JK
N \ X5=-400
R \ PC5.KL=V5.K
NOTE \ H5 - HEADWAY OF VEHICLE 5
NOTE \ V5 - VELOCITY OF VEHICLE 5
NOTE \ PC5 - POSITION CHANGE OF VEHICLE 5
NOTE \ X5 - POSITION OF VEHICLE 5
A \ V6.K=(V0*VCF*EXP(-0.5*(K6.K*DCF/KJ)**2)
N \ X6=-500
R \ PC6.KL=V6.K
NOTE \ H6 - HEADWAY OF VEHICLE 6
NOTE \ V6 - VELOCITY OF VEHICLE 6
NOTE \ PC6 - POSITION CHANGE OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6
A V71.K=(VO*VCF*EXP(-0.5*(K7.K*DCF/KJ)**2)
N X7=3000
R PC7.KL=V7.K
NOTE H7 - HEADWAY OF VEHICLE 7
NOTE V7 - VELOCITY OF VEHICLE 7
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7
A V81.K=(VO*VCF*EXP(-0.5*(K8.K*DCF/KJ)**2)
N X8=3300
R PC8.KL=V8.K
NOTE H8 - HEADWAY OF VEHICLE 8
NOTE V8 - VELOCITY OF VEHICLE 8
NOTE PC8 - POSITION CHANGE OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8
A V91.K=(VO*VCF*EXP(-0.5*(K9.K*DCF/KJ)**2)
N X9=3600
NOTE H9 - HEADWAY OF VEHICLE 9
NOTE V9 - VELOCITY OF VEHICLE 9
NOTE PC9 - POSITION CHANGE OF VEHICLE 9
NOTE X9 - POSITION OF VEHICLE 9
SPEC DT=0.1/LENGTH=150/PLTPE=1
PLOT V1=1,V2=2,V3=3,V4=4,V5=5,V6=6,V7=7,V8=8,V9=9(*,*)
RUN
QUIT
5.8 Drew’s Model

A number of traffic stream models have been proposed over the years. The earlier models assumed a single regime phenomenon over the complete range of flow conditions including free-flow and congested flow situations. Later models attempted to improve on the earlier models by considering two separate regimes and attempted to generalize by introducing additional parameters that could be used to distinguish between roadway environments. Drew’s model is one such a model where in he proposed a formulation based on Greenshields model, but with the introduction of an additional parameter ‘n’ which can be represented as follows:

\[ u = u_f [1 - \left( \frac{k}{k_j} \right)^{\frac{n+1}{2}}] \]

Where,

- \( u \) = speed of vehicle
- \( k \) = density
- \( u_f \) = free-flow speed
- \( k_j \) = jam density

A family of models can be developed by varying the parameter ‘n’. Drew suggested varying ‘n’ from -1 to +1 and called these models a linear model when \( n=1 \), a parabolic model for an ‘n’ value of zero, and an exponential model when \( n=-1 \). Drew’s formulation converts to Greenshields model when \( n=1 \). By using a family of models, the traffic stream could be better represented for different conditions.
In this study all the three models that can be generated for ‘n’ values of $-1$, $0$, $1$ are simulated using DYNAMO programs. This kind of study shows how well each model functions under various traffic conditions. Drew’s model converts to Greenshields’ model when $n=1$. Hence, the simulation for this model is essentially similar to the one discussed for Greenshields’ model. The remaining two cases when $n=0$ and $n=-1$ are carefully studied in order to evaluate the efficiency of this model.

The main parameter that affects the flow of traffic in this model is the density of the traffic stream. The vehicles in the traffic stream are interrelated using this parameter which affects the speed of the following vehicles. It can be observed from the equation that the jam density and free-flow speed are needed to be calculated in order to use this model. Once these two parameters are fixed, headway between vehicles is the only factor affecting the flow of traffic.

The traffic stream is simulated by first giving a speed characteristic to the leading vehicle in such a way that it has to pass through a low speed limit zone. Then it is simulated to stop at a signalized intersection. The traffic stream progresses further where it meets three merging vehicles at certain gaps. The model is also developed to study the affects of high densities, low speeds and blockage of road.
The results of this simulation are obtained as plots with time on one axis and the various parameters like speed, density, distance on the other axis. From the results it is clearly evident that these three models together represent a traffic stream which is simulated for various traffic flow conditions very well. The non-linear acceleration pattern is well propagated through all the vehicles in the traffic stream. The headway and speed are continuously adjusted until stability in traffic flow is achieved. This model also has the disadvantage of the difficulty in the estimation of jam density. At densities higher than 125 vehicles per lane per mile, this model shows the speed approaching zero which need not be the case. This model cannot be individually used in the congestion depiction models as it is not very efficient over the entire range of densities.

5.8.1 DYNAMO Program For Drew's Model

A
A
L
N
V11=0
A
A
A
V142.K=CLIP(V11.K,0,TIME.K,60)
A
L
R
A12.KL=4
N
V131=95.355
R
A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C
A=4.0
C
B=0.419485
C
V0=0
L
TIME.K=TIME.J+DT
N
TIME=0
N
Vf=80
C
KJ=120
NOTE
VFC - VELOCITY CONVERSION FACTOR
NOTE
DCF - DISTANCE CONVERSION FACTOR
NOTE
KJ - JAM DENSITY
NOTE
V1 - VELOCITY OF VEHICLE 1
R
PC1.KL=V1.K
L
N
X1=0
NOTE
X1 - POSITION OF VEHICLE 1
NOTE
PC1 - POSITION CHANGE OF VEHICLE 1
A
V2.K=VF*VCF*(1-(K2.K*DCF/KJ)**(0.5))
A
A
K2.K=1/H2.K
L
X2.K=X2.J+(DT)*PC2.JK
N
X2=-100
R
PC2.KL=V2.K
NOTE
H2 - HEADWAY OF VEHICLE 2
NOTE
V2 - VELOCITY OF VEHICLE 2
NOTE
PC2 - POSITION CHANGE OF VEHICLE 2
A
V3.K=VF*VCF*(1-(K3.K*DCF/KJ)**(0.5))
A
A
L
N
X3=-200
R
NOTE
H3 - HEADWAY OF VEHICLE 3
NOTE
V3 - VELOCITY OF VEHICLE 3
NOTE
PC3 - POSITION CHANGE OF VEHICLE 3
A
V4.K=VF*VCF*(1-(K4.K*DCF/KJ)**(0.5))
A
A
L
N
X4=-300
R
PC4.KL=V4.K
NOTE
H4 - HEADWAY OF VEHICLE 4
NOTE
V4 - VELOCITY OF VEHICLE 4
NOTE
PC4 - POSITION CHANGE OF VEHICLE 4
A
C
V52=0
A
V51.K=VF*VCF*(1-(K5.K*DCF/KJ)**(0.5))
A
A
K5.K=1/H5.K
L
X5.K=X5.J+(DT)*PC5.JK
N
X5=-400
R
PC5.KL=V5.K
NOTE
H5 - HEADWAY OF VEHICLE 5
NOTE
V5 - VELOCITY OF VEHICLE 5
NOTE
PC5 - POSITION CHANGE OF VEHICLE 5
NOTE
X5 - POSITION OF VEHICLE 5
A
V6.K = VF*VCF*(1-(K6.K*DCF/KJ)**(0.5))
A
A
L
N
X6 = -500
R
NOTE
H6 - HEADWAY OF VEHICLE 6
NOTE
V6 - VELOCITY OF VEHICLE 6
NOTE
PC6 - POSITION CHANGE OF VEHICLE 6
NOTE
X6 - POSITION OF VEHICLE 6
A
A
V71.K = VF*VCF*(1-(K7.K*DCF/KJ)**(0.5))
A
A
L
N
X7 = 3000
R
PC7.KL = V7.K
NOTE
H7 - HEADWAY OF VEHICLE 7
NOTE
V7 - VELOCITY OF VEHICLE 7
NOTE
PC7 - POSITION CHANGE OF VEHICLE 7
NOTE
X7 - POSITION OF VEHICLE 7
A
A
V81.K = VF*VCF*(1-(K8.K*DCF/KJ)**(0.5))
A
A
L
N
X8 = 3300
R
PC8.KL = V8.K
NOTE
H8 - HEADWAY OF VEHICLE 8
NOTE
V8 - VELOCITY OF VEHICLE 8
NOTE
PC8 - POSITION CHANGE OF VEHICLE 8
NOTE
X8 - POSITION OF VEHICLE 8
A
A
V91.K = VF*VCF*(1-(K9.K*DCF/KJ)**(0.5))
A
A
L
N
X9 = 3600
R
NOTE
H9 - HEADWAY OF VEHICLE 9
NOTE
V9 - VELOCITY OF VEHICLE 9
NOTE
PC9 - POSITION CHANGE OF VEHICLE 9
NOTE
X9 - POSITION OF VEHICLE 9
SPEC
DT = 0.1/LENGTH = 150/PLTPER = 1
PLOT
V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9(*,*)
RUN
QUIT

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Figure 8. Time-Space diagram for Drew's model
5.9 Edie’s Model

Edie was the first to point out the possibility of discontinuity in the flow-concentration curve under maximum flow conditions. He proposed two separate two-regime models because of reservations of using car following based models under free-flow conditions. These models, which describe macroscopic relationships, are based on the convertibility models developed from the microscopic car-following model. Edie proposed the use of Underwood model for the free-flow regime and the Greenberg model for congested flow regime. Edie’s model uses density to determine the breakpoint to differentiate between regimes. This model uses a breakpoint of \( k = 50 \), such that if \( k \) is less than or equal to 50 then the traffic flow is considered to be in free-flow regime while \( k \) greater than or equal to 50 indicates congested flow regime. The equations for free-flow and congested flow regimes can be represented as follows:

**Free-flow regime:**

\[
u = 54.9e^{-\frac{k}{163.9}}
\]

if \( k \leq 50 \)
Congested flow regime:-

\[ u = 26.81a \left( \frac{162.5}{k} \right) \]

if \( k \geq 50 \)

Where, \( u = \) speed of vehicle

\( k = \) density

The traffic stream could be simulated using this model very efficiently as this model is capable of representing the flow over a wider range as compared to other single regime models. Even though this is a multi-regime model the simulation techniques remain the same. The leading vehicle in this case also plays a major role. The traffic stream progress is defined by describing the motion of the leading vehicle. This could be accomplished by incorporating all the parameters like the low speed regions, interruptions, high densities and merging regions into the speed model of the leading vehicle. The following vehicle adopts these factors with a certain degree of sensitivity which is dictated by the speed model that is used.

In this model two formulations are used to describe the speed of the following vehicle and the simulation model switches one formula for the other whenever the boundary condition between the two regimes described by these two models are met. In this model the speed of the traffic stream depends on the density only as it is the only variable in this model.
The analysis of the various parameters discussed could be carried out very efficiently by obtaining the plots of the various parameters involved in the variations caused to the speed of the vehicles like density, headway, acceleration and distance. It is observed from this simulation that this model represents the traffic stream quite efficiently for the given conditions. The speed variations throughout the traffic stream are also uniform and all the vehicles accelerate steadily depending on the various other sensitivity factors that affect the acceleration process. This model however has the difficulty in interpreting jam density as it underestimates this value and hence depicts congestion at a stage wherein the field congestion has not set in yet. Hence, this model has to be calibrated well in order to use it in the congestion depiction model.

5.9.1 DYNAMO Program For Edie’s Model

N  V11=0
A  V142.K=CLIP(V11.K,0,TIME.K,60)
R  A12.KL=4
N  V131=95.355
R  A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C  A=4.0
C  B=0.419485
C  V0=0
L  TIME.K=TIME.J+DT
N  TIME=0
N  VF=80
C  DCF=5280
C  VCF=1.467
NOTE VCF - VELOCITY CONVERSION FACTOR
NOTE DCF - DISTANCE CONVERSION FACTOR
NOTE V1 - VELOCITY OF VEHICLE 1
R PC1.KL = V1.K
N X1 = 0
NOTE X1 - POSITION OF VEHICLE 1
NOTE PC1 - POSITION CHANGE OF VEHICLE 1
C V22 = 75
A V2.C.K = 26.8*LOGN(162.5/K2.K)
A V2.F.K = 54.9*EXP(-K2.K/163.9)
NOTE V2 - VELOCITY OF VEHICLE 2
A K2.K = DCF/H2.K
L X2.K = X2.J + (DT)*PC2.JK
R PC2.KL = V2.K
N X2 = -100
NOTE H2 - HEADWAY OF VEHICLE 2
NOTE PC2 - POSITION CHANGE OF VEHICLE 2
NOTE X2 - POSITION OF VEHICLE 2
C V32 = 75
A V3.F.K = 54.9*EXP(-K3.K/163.9)
NOTE V3 - VELOCITY OF VEHICLE 3
N X3 = -200
NOTE H3 - HEADWAY OF VEHICLE 3
NOTE PC3 - POSITION CHANGE OF VEHICLE 3
NOTE X3 - POSITION OF VEHICLE 3
C V42 = 75
A V4.C.K = 26.8*LOGN(162.5/K4.K)
A V4.F.K = 54.9*EXP(-K4.K/163.9)
NOTE V4 - VELOCITY OF VEHICLE 4
R PC4.KL = V4.K
N X4 = -300
NOTE H4 - HEADWAY OF VEHICLE 4
NOTE PC4 - POSITION CHANGE OF VEHICLE 4
NOTE X4 - POSITION OF VEHICLE 4
C V52=75
A V5C.K=26.8*LOGN(162.5/K5.K)
A V5F.K=54.9*EXP(-K5.K/163.9)
NOTE V5 - VELOCITY OF VEHICLE 5
L X5.K=X5.J+(DT)*PC5.JK
R PC5.KL=V5.K
N X5=-400
NOTE H5 - HEADWAY OF VEHICLE 5
NOTE PC5 - POSITION CHANGE OF VEHICLE 5
NOTE X5 - POSITION OF VEHICLE 5
C V62=75
A V6C.K=26.8*LOGN(162.5/K6.K)
A V6F.K=54.9*EXP(-K6.K/163.9)
NOTE V6 - VELOCITY OF VEHICLE 6
N X6=-500
NOTE H6 - HEADWAY OF VEHICLE 6
NOTE PC6 - POSITION CHANGE OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6
C V72=75
A V7C.K=26.8*LOGN(162.5/K7.K)
A V7F.K=54.9*EXP(-K7.K/163.9)
A K7.K=MAX(1,K71.K)
R PC7.KL=V7.K
N X7=3000
NOTE H7 - HEADWAY OF VEHICLE 7
NOTE V7 - VELOCITY OF VEHICLE 7
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7
C V82=75
A V8C.K=26.8*LOGN(162.5/K8.K)
A V8F.K=54.9*EXP(-K8.K/163.9)
A K8.K=MAX(1,K81.K)

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R
PC8.KL = V8.K
N
X8 = 3300
NOTE
H8 - HEADWAY OF VEHICLE 8
NOTE
V8 - VELOCITY OF VEHICLE 8
NOTE
PC8 - POSITION CHANGE OF VEHICLE 8
NOTE
X8 - POSITION OF VEHICLE 8
A
C
V92 = 75
A
A
A
A
A
K9.K = MAX(1, K91.K)
A
L
R
N
X9 = 3600
NOTE
H9 - HEADWAY OF VEHICLE 9
NOTE
V9 - VELOCITY OF VEHICLE 9
NOTE
PC9 - POSITION CHANGE OF VEHICLE 9
NOTE
X9 - POSITION OF VEHICLE 9
SPEC
DT = 0.1/LENGTH = 150/PLT_PER = 1
PLOT
V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9 (*, *)
RUN
QUIT
5.10 Two-Regime Linear Model

Two-regime linear model is developed by using Greenshields-type model for the free-flow regime and the congested flow regime separately. This model is developed in order to counter the deficiencies that Greenshields model has over some portion of the density range. The most disconcerting feature of any single-regime model is its inability to track faithfully the measured field data near capacity conditions. A discontinuity can be observed in the flow-speed-density relationships as depicted by measured field data in the vicinity of capacity conditions. The Two-Regime Linear model uses a density of 65 vehicles per lane per mile as the breakpoint to differentiate between free-flow and congested flow regimes. This differentiation is very essential as the behavior of Greenshields model, which is adapted in this model, has different characteristics under these conditions. The two-regime linear model could be expressed as follows:

**Free-flow regime:**

\[ u = 60.9 - 0.515k \]

\[ \text{if } k \leq 65 \]
Congested flow regime:

\[ u = 40.0 - 0.265k \]

if \( k \geq 65 \)

Where, \( u \) = speed of vehicle

\( k \) = density

The different formulations that are used for the two regimes in the two-regime linear model are represented in the simulation program by using a CLIP function wherein, the appropriate equation is used whenever the boundary condition is met. The leading car is used to represent the traffic stream characteristics, as all the factors that affect the traffic flow are embedded in the velocity function of the leading vehicle. All the other vehicles in the traffic stream would follow their immediately preceding vehicle. There will be a certain time lag before all the vehicles attain the maximum speed as the speed of the following vehicles is controlled by the two-regime linear model. The only parameter in this model which affects the speed of the vehicles is the density of traffic stream which in turn depends on the headways of the various vehicles in the stream. In this model three vehicles are made to merge into the mainstream at certain gaps in order to study the affects of merging vehicles on the speed of the vehicles according to this model.
The various traffic characteristics that are introduced into the speed function of the leading vehicle are transmitted to all the vehicles in the stream very effectively in this model. This model underestimates the rate at which the speed of all the vehicles in the traffic stream is increased as it takes a longer time as per the simulation results, for each vehicle to achieve maximum speed. This is probably due to underestimation of jam density which is inherent in the model. The density plot obtained for this model is a very good tool to show the efficiency of this model to represent realistic traffic flow conditions. This model is very good in adjusting the stream after the merging of vehicles is accomplished. It can be concluded that this model is much more efficient than the single regime models.

5.10.1 DYNAMO Program For Two-Regime Linear Model

```
N V11=0
A V142.K=CLIP(V11.K,0,TIME.K,60)
R A12.KL=4
N V131=95.355
R A1.XL=(A-B*V0)*EXP(-B*TIME.K)
C A=4.0
C B=0.419485
C V0=0
L TIME.K=TIME.J+DT
N TIME=0
N VF=80
C DCF=5280
C VCF=1.467
```
NOTE VCF - VELOCITY CONVERSION FACTOR
NOTE DCF - DISTANCE CONVERSION FACTOR
NOTE V1 - VELOCITY OF VEHICLE 1
R PC1.KL=V1.K
N X1=0
NOTE X1 - POSITION OF VEHICLE 1
NOTE PC1 - POSITION CHANGE OF VEHICLE 1
A V2C.K=40-0.265*K2.K
A V2F.K=60.9-0.515*K2.K
A K2.K=DCF/H2.K
NOTE V2 - VELOCITY OF VEHICLE 2
NOTE H2 - HEADWAY OF VEHICLE 2
L X2.K=X2.J+(DT)*PC2.JK
N X2=-100
R PC2.KL=V2.K
NOTE PC2 - POSITION CHANGE OF VEHICLE 2
NOTE X2 - POSITION OF VEHICLE 2
A V3C.K=40-0.265*K3.K
A V3F.K=60.9-0.515*K3.K
NOTE V3 - VELOCITY OF VEHICLE 3
NOTE H3 - HEADWAY OF VEHICLE 3
N X3=-200
NOTE PC3 - POSITION CHANGE OF VEHICLE 3
NOTE X3 - POSITION OF VEHICLE 3
A V4C.K=40-0.265*K4.K
A V4F.K=60.9-0.515*K4.K
NOTE V4 - VELOCITY OF VEHICLE 4
NOTE H4 - HEADWAY OF VEHICLE 4
N X4=-300
R PC4.KL=V4.K
NOTE PC4 - POSITION CHANGE OF VEHICLE 4
NOTE X4 - POSITION OF VEHICLE 4
A V5C.K=40-0.265*K5.K
A V5F.K=60.9-0.515*K5.K
NOTE V5 - VELOCITY OF VEHICLE 5
NOTE H5 - HEADWAY OF VEHICLE 5
L  X5.K=X5.J+(DT)*PC5.JK
N  X5=-400
R  PC5.KL=V5.K
NOTE  PC5 - POSITION CHANGE OF VEHICLE 5
NOTE  X5 - POSITION OF VEHICLE 5
A  V6.C.K=40.0-0.265*K6.K
A  V6.F.K=60.9-0.515*K6.K
NOTE  V6 - VELOCITY OF VEHICLE 6
NOTE  H6 - HEADWAY OF VEHICLE 6
N  X6=-500
NOTE  PC6 - POSITION CHANGE OF VEHICLE 6
NOTE  X6 - POSITION OF VEHICLE 6
A  V7.C.K=40.0-0.265*K7.K
A  V7.F.K=60.9-0.515*K7.K
NOTE  V7 - VELOCITY OF VEHICLE 7
NOTE  H7 - HEADWAY OF VEHICLE 7
N  X7=3000
R  PC7.KL=V7.K
NOTE  PC7 - POSITION CHANGE OF VEHICLE 7
NOTE  X7 - POSITION OF VEHICLE 7
A  V8.C.K=40.0-0.265*K8.K
A  V8.F.K=60.9-0.515*K8.K
NOTE  V8 - VELOCITY OF VEHICLE 8
NOTE  H8 - HEADWAY OF VEHICLE 8
N  X8=3300
R  PC8.KL=V8.K
NOTE  PC8 - POSITION CHANGE OF VEHICLE 8
NOTE  X8 - POSITION OF VEHICLE 8
A  V9.F.K=60.9-0.515*K9.K
NOTE  V9 - VELOCITY OF VEHICLE 9
NOTE  H9 - HEADWAY OF VEHICLE 9
Figure 10. Time-Space diagram for Two-Regime linear model
5.11 Modified Greenberg Model

The work on developing a family of single-regime models was extended to developing a family of two-regime models. A discontinuity in the flow-speed-density relationships as depicted by measured field data in the vicinity of capacity conditions is observed. Even with the development of the family of single-regime models, the prediction of flows approaching capacity was not completely satisfied. The modified Greenberg model is developed in order to overcome some of the deficiencies of the single-regime Greenberg model. Greenberg’s model is essentially focused on representing the congested flow regime and the free-flow regime is neglected. Hence, in this model the free-flow regime speed is constant while Greenberg’s model represents the congested flow regime. The free-flow and congested regimes are demarked by using a breakpoint for density which is given by \( k = 35 \), such that if \( k \) is less than or equal to 35 then the flow is in free-flow regime, otherwise the flow is in congested flow regime. The modified Greenberg model can be represented as follows:

**Free-flow regime:**

\[
u = 48
\]

\[
if \ k \leq 35
\]
Congested flow regime:

\[ u = 32 \ln \left( \frac{145.5}{k} \right) \]

if \( k \geq 35 \)

Where, \( u \) = speed of the vehicle
\( k \) = density

The progress of traffic stream under realistic conditions could be simulated by using this model where the entire range of speed-density relationship is represented by the modified Greenberg model. This model is essentially simulated by defining the headways between various vehicles in order to determine the density of the traffic stream which is the only factor affecting the speed of the vehicles in the traffic stream.

The leading vehicle in the stream is given a speed function incorporating all the realistic parameters like low speed regions and signalized intersections etc. These factors are adopted by other vehicles that follow this leading vehicle as it is obvious from the equation that each vehicle's speed is restricted by its distance from the preceding vehicle. This headway is utilized in timing the merging of the vehicles which join the mainstream of the vehicles at some distance away from the point where the leading vehicle starts at the beginning of the simulation.
The simulation is carried out mainly to calculate the values of different traffic flow parameters as the vehicles start moving. This can be achieved by obtaining plots for various parameters that are to be analyzed. In this simulation model, the plot between distance and time gives a nice representation of the movement of vehicles once they start moving. This model assumes a constant speed in free-flow regime which at many times may not be very realistic as even under free-flow conditions there will be variations in the speeds of the vehicle. This model also incorporates all the defects of Greenberg’s model. The breakpoint for congested regime appears to be too low as from this point onwards Greenberg’s model is applied which is not very good under free-flow conditions.

5.11.1 DYNAMO Program For Modified Greenberg Model

\begin{verbatim}
N   V11=0
A   V142.K=CLIP(V11.K,0,TIME.K,60)
R   A12.KL=4
N   V131=95.355
R   A1.KL=(A-B*V0)*EXP(-B*TIME.K)
C   A=4.0
C   B=0.419485
C   V0=0
L   TIME.K=TIME.J+DT
N   TIME=0
N   VF=80
\end{verbatim}
C        DCF=5280
C        VCF=1.467
NOTE     VCF - VELOCITY CONVERSION FACTOR
NOTE     DCF - DISTANCE CONVERSION FACTOR
NOTE     V1 - VELOCITY OF VEHICLE 1
R        PC1.KL=V1.K
N        X1=0
NOTE     X1 - POSITION OF VEHICLE 1
NOTE     PC1 - POSITION CHANGE OF VEHICLE 1
A        V2C.K=32*LOGN(145.5/K2.K)
A        V2F=48
A        K2.K=DCF/H2.K
NOTE     V2 - VELOCITY OF VEHICLE 2
NOTE     H2 - HEADWAY OF VEHICLE 2
L        X2.K=X2.J+(DT)*PC2.JK
R        PC2.KL=V2.K
N        X2=.-100
NOTE     PC2 - POSITION CHANGE OF VEHICLE 2
NOTE     X2 - POSITION OF VEHICLE 2
A        V3C.K=32*LOGN(145.5/K3.K)
A        V3F=48
NOTE     V3 - VELOCITY OF VEHICLE 3
NOTE     H3 - HEADWAY OF VEHICLE 3
N        X3=-200
NOTE     PC3 - POSITION CHANGE OF VEHICLE 3
NOTE     X3 - POSITION OF VEHICLE 3
A        V4C.K=32*LOGN(145.5/K4.K)
A        V4F=48
NOTE     V4 - VELOCITY OF VEHICLE 4
NOTE     H4 - HEADWAY OF VEHICLE 4
R        PC4.KL=V4.K
N        X4=-300
NOTE     PC4 - POSITION CHANGE OF VEHICLE 4
NOTE     X4 - POSITION OF VEHICLE 4
A        V5C.K=32*LOGN(145.5/K5.K)
A        V5F=48

Traffic Stream Modeling
NOTE V5 - VELOCITY OF VEHICLE 5
NOTE H5 - HEADWAY OF VEHICLE 5
R PC5.KL = V5.K
N X5 = -400
NOTE PC5 - POSITION CHANGE OF VEHICLE 5
NOTE X5 - POSITION OF VEHICLE 5
A V6C.K = 32*LOGN(145.5/K0.K)
C V6F = 48
NOTE V6 - VELOCITY OF VEHICLE 6
NOTE H6 - HEADWAY OF VEHICLE 6
N X6 = -500
NOTE PC6 - POSITION CHANGE OF VEHICLE 6
NOTE X6 - POSITION OF VEHICLE 6
A K7.K = MAX(1, K71.K)
A V7C.K = 32*LOGN(145.5/K7.K)
C V7F = 48
NOTE V7 - VELOCITY OF VEHICLE 7
NOTE H7 - HEADWAY OF VEHICLE 7
R PC7.KL = V7.K
N X7 = 3000
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
NOTE X7 - POSITION OF VEHICLE 7
A K8.K = MAX(1, K81.K)
A V8C.K = 32*LOGN(145.5/K8.K)
C V8F = 48
NOTE V8 - VELOCITY OF VEHICLE 8
NOTE H8 - HEADWAY OF VEHICLE 8
R PC8.KL = V8.K
N X8 = 3300
NOTE PC8 - POSITION CHANGE OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8
A V9C.K = 32*LOGN(145.5/K9.K)
A \quad K91.K = DCF/H9.K
C \quad V9F = 48

NOTE \quad V9 - VELOCITY OF VEHICLE 9
NOTE \quad H9 - HEADWAY OF VEHICLE 9
R \quad PC9.KL = V9.K
N \quad X9 = 3600

NOTE \quad PC9 - POSITION CHANGE OF VEHICLE 9
NOTE \quad X9 - POSITION OF VEHICLE 9
SPEC \quad DT = 0.1/LENGTH = 150/PLTPEK = 1

PLOT \quad V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9 (*, *)
RUN
QUIT
5.12 Three-Regime Linear Model

The three-regime linear model is the last of all the multi-regime models suggested by the Northwestern group to represent traffic flow. This model has three regimes namely, free-flow regime, transitional-flow regime and congested flow regime each being represented by the Greenshields formulation. The first difficulty in this model is determining the breakpoint between the regimes. The Northwestern group applied the work of Quandt on likelihood functions to identify the breakpoints between regimes using freeway data set. Then using regression analysis, the best model was selected for each regime. This model uses a ‘k’ value of 40 to define the boundaries of free-flow and transitional flow regime in such a way that if ‘k’ is less than or equal to 40 then it is free-flow otherwise, flow is in transitional flow regime. If ‘k’ is greater than or equal to 65 then the flow is in congested flow regime. Hence, this model uses three equations to represent traffic flow which can be represented as:

Free-flow regime:-

\[ u = 50-0.098k \]

if \( k \leq 40 \)

Transitional-flow regime:-

\[ u = 81.4-0.913k \]

if \( 40 \leq k \leq 65 \)
**Congested-flow regime:**

\[ u = 40 - 0.265k \]

*if k \geq 65*

Where, \( u = \) speed of vehicle

\( k = \) density

This model is simulated based on the assumption that all the vehicles in the stream start with a speed of zero at the time the simulation begins. The leading car is then given a non-linear acceleration and the other cars follow the leading car according to one of the regimes described above. These regimes could be incorporated in every vehicle by providing a CLIP function for the speed of the following vehicles.

The leading vehicle is given certain realistic conditions like low speed regions like curves, bridges etc., interruptions like signalized intersections etc. in defining the speed of the leading vehicle. The following vehicles will have velocity characteristics similar to the leading vehicle, as they are all tied together by the density function. Due to this, the merging of vehicles into mainstream can also be achieved by controlling these parameters. The model is simulated for all the possible realistic conditions and plots are obtained for various traffic flow parameters like density, speed, distance etc.
The plots obtained indicate that the fluctuations in the densities of the traffic stream is very well reflected by corresponding changes in the speed of vehicles in this model. It can also be observed that the stability of traffic flow is attained according to this model in a more realistic way as it has three different regimes to define the traffic flow conditions. The transitional-flow regime plays an important role in representing the speed-density relationships. Unlike other models, where only two regimes are considered, this model has more flexibility as it effectively represents the various conditions during the progress of a traffic stream. The transitional-flow regime takes care of the unstable conditions that arise before congestion sets in. If congestion is depicted at this stage, then precautionary measures could be taken. hence, this model may be used in the congestion depiction model.

5.12.1 DYNAMO Program For Three-Regime Linear Model

\[
\begin{align*}
N & \quad V11 = 0 \\
A & \quad V142.K = CLIP(V11.K, 0, TIME.K, 60) \\
R & \quad A12.KL = 4 \\
N & \quad V131 = 95.355 \\
R & \quad A1.KL = (A-B*V0)*EXP(-B*TIME.K) \\
C & \quad A = 4.0 \\
C & \quad B = 0.419485 \\
C & \quad V0 = 0 \\
L & \quad TIME.K = TIME.J + DT
\end{align*}
\]
N  TIME=0
N  VF=80
C  DCF=5280
C  VCF=1.467
NOTE  VCF - VELOCITY CONVERSION FACTOR
NOTE  DCF - DISTANCE CONVERSION FACTOR
NOTE  V1 - VELOCITY OF VEHICLE 1
R  PC1.KL=V1.K
N  X1=0
NOTE  X1 - POSITION OF VEHICLE 1
NOTE  PC1 - POSITION CHANGE OF VEHICLE 1
A  V2F.K=50-0.098*K2.K
A  V211.K=40-0.265*K2.K
A  V22.K=81.4-0.913*K2.K
A  K2.K=DCF/H2.K
L  X2.K=X2.J+(DT)*PC2.JK
R  PC2.KL=V2.K
N  X2=-100
NOTE  V2 - VELOCITY OF VEHICLE 2
NOTE  K2 - DENSITY
NOTE  H2 - HEADWAY OF VEHICLE 2
NOTE  X2 - POSITION OF VEHICLE 2
NOTE  PC2 - POSITION CHANGE OF VEHICLE 2
A  V3F.K=50-0.098*K3.K
A  V311.K=40-0.265*K3.K
A  V32.K=81.4-0.913*K3.K
N  X3=-200
NOTE  V3 - VELOCITY OF VEHICLE 3
NOTE  K3 - DENSITY
NOTE  H3 - HEADWAY OF VEHICLE 3
NOTE  X3 - POSITION OF VEHICLE 3
NOTE  PC3 - POSITION CHANGE OF VEHICLE 3
A  V4F.K=50-0.098*K4.K
A  V411.K=40-0.265*K4.K
A  V42.K=81.4-0.913*K4.K
R  PC4.KL=V4.K

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N
NOTE
V4 - VELOCITY OF VEHICLE 4
NOTE
K4 - DENSITY
NOTE
H4 - HEADWAY OF VEHICLE 4
NOTE
X4 - POSITION OF VEHICLE 4
NOTE
PC4 - POSITION CHANGE OF VEHICLE 4
A
A
V5F.K = 50-0.098*K5.K
A
A
V511.K = 40-0.265*K5.K
A
V52.K = 81.4-0.913*K5.K
A
K5.K = DCF/H5.K
A
L
X5.K = X5.J + (DT)*PC5.JK
R
PC5.KL = V5.K
N
X5 = -400
NOTE
V5 - VELOCITY OF VEHICLE 5
NOTE
K5 - DENSITY
NOTE
H5 - HEADWAY OF VEHICLE 5
NOTE
X5 - POSITION OF VEHICLE 5
NOTE
PC5 - POSITION CHANGE OF VEHICLE 5
A
A
V6F.K = 50-0.098*K6.K
A
A
V611.K = 40-0.265*K6.K
A
V62.K = 81.4-0.913*K6.K
A
A
L
R
N
X6 = -500
NOTE
V6 - VELOCITY OF VEHICLE 6
NOTE
K6 - DENSITY
NOTE
H6 - HEADWAY OF VEHICLE 6
NOTE
X6 - POSITION OF VEHICLE 6
NOTE
PC6 - POSITION CHANGE OF VEHICLE 6
A
, O,X2,K,3050),0,H4.K,50)
A
V7F.K = 50-0.098*K7.K
A
A
V711.K = 40-0.265*K7.K
A
V72.K = 81.4-0.913*K7.K
A
A
L
R
PC7.KL = V7.K
N
X7 = 3000
NOTE
V7 - VELOCITY OF VEHICLE 7
NOTE
K7 - DENSITY
NOTE
H7 - HEADWAY OF VEHICLE 7
NOTE
X7 - POSITION OF VEHICLE 7
NOTE PC7 - POSITION CHANGE OF VEHICLE 7
A V8F.K = 50 - 0.098*K8.K
A V811.K = 40 - 0.265*K8.K
A V82.K = 81.4 - 0.913*K8.K
R PC8.KL = V8.K
N X8 = 3300
NOTE V8 - VELOCITY OF VEHICLE 8
NOTE K8 - DENSITY
NOTE H8 - HEADWAY OF VEHICLE 8
NOTE X8 - POSITION OF VEHICLE 8
NOTE PC8 - POSITION CHANGE OF VEHICLE 8
A V9F.K = 50 - 0.098*K9.K
A V92.K = 81.4 - 0.913*K9.K
N X9 = 3600
NOTE V9 - VELOCITY OF VEHICLE 9
NOTE K9 - DENSITY
NOTE H9 - HEADWAY OF VEHICLE 9
NOTE X9 - POSITION OF VEHICLE 9
NOTE PC9 - POSITION CHANGE OF VEHICLE 9
SPEC DT = 0.1, LENGTH = 150, PLTPer = 1
PLOT V1 = 1, V2 = 2, V3 = 3, V4 = 4, V5 = 5, V6 = 6, V7 = 7, V8 = 8, V9 = 9 (*, *)
RUN
QUIT
6. A MODEL TO PREDICT CONGESTION

Congestion is said to exist when speed inversion occurs. Congestion may be considered as the stable condition which exists following a speed reduction of the order of 10 mph occurring at high volume rates. Congestion doesn’t necessarily reduce volume; it may result in higher volumes. Congestion is usually associated with a high rate of change in concentration and a correspondingly small rate of change in volume. As congestion develops, the increase in concentration results in a considerable increase in travel time to the motorist, with little or no beneficial effects to the highway system. In order to avoid this the depiction of congestion at an early stage plays an important role.

Congestion could be depicted by using the model developed to simulate the Three-regime linear model. In this model all the parameters that are involved in the traffic flow are considered. a general situation where certain interruptions are possible is also provided. This model could be easily extended to any particular facility. The model may be used in calculating various measures of congestion like density, speed, flow, acceleration etc., by measuring the time headway between vehicles passing a presence-type detector. Two presence type detectors spaced at a distance are required in order to calculate speed of the vehicle, density and other such parameters as distance and time are measured.
The presence-type detector is used essentially to detect the presence and passage of vehicles over a short segment of roadway. When a vehicle enters the detection zone, the sensor is activated and remains so until the vehicle leaves the detection zone. The "on" time referred to as the vehicle occupancy time requires the vehicle to travel a distance equivalent to its length plus the length of the detection zone. Note that the length of the detection zone, not necessarily the length of physical detector is used. The vehicle occupancy time is a function of vehicle speed, vehicle length and detection zone length. Both microscopic and macroscopic characteristics of traffic flow can be obtained from presence-type detectors.

The measurements obtained from these detectors could be fed to the three-regime linear model adapted to calculate measures of congestion like density. If the resulting value of density falls in transition zone then it may be concluded that congestion is about to set in. The field data over a period of time can be calibrated to give information as to when congestion exactly sets in. These detectors should be placed at such locations where there is a high probability of congestion setting in like low speed zones, sharp curves, intersections, bottlenecks etc. The results obtained from these detectors may be used as the input to the model developed to depict congestion.
6.1 DESCRIPTION OF THE MODEL

Congestion may be defined by the microscopic and macroscopic measures of traffic performance such as volume, speed, speed differences, headways, density and the variations of individual speeds, speed differences and headways. Prediction of congestion becomes a critical determination for the operation of a control system. Observations of individual measurements such as speed, volume and occupancy at times preceding breakdown in flow appear to change from one state to another at rapid rates. Such speedy changes may occur from varying preceding levels and achieve the change in state at varying speed. The predictive nature of these measurements appears to be associated with probabilities of failure. Thus, for a given set of conditions there may be an associated probability of failure. Such a system may be setup with certain confidence limits to predict congestion.

In this model, congestion is predicted by using the microscopic measurements that are obtained from a presence-type detector system. Using this type of detector system the speeds of the vehicles can be measured which are used in the calculation of density of traffic stream. The measurements of the traffic performance like speed can be obtained from this type of detector system located at various critical locations at different times of the day. This speed data could be input into a simulation model in order to evaluate the other measures of effectiveness.
The time of entry and the time of exit of the vehicles in the test zone are recorded automatically and this is used as the input for this model by using arrays. The data is collected for a fixed interval of time and the prediction is done for near future whose interval may vary depending on the time of the day. The total number of data points that are input in this model may vary and the model has flexibility to adopt these variations. The time at the beginning of the test section is taken as the starting time and the time at the end of the test zone is taken as the ending time. The distance travelled by vehicles in the test zone is fixed and hence the speed of the vehicles can be calculated very easily.

The speeds of the vehicles are calculated from the data set and this speed data is used to determine the density of the traffic stream. This model uses the three-regime linear model to establish a relationship between the speed of the vehicle and the density of the traffic stream. The densities are calculated using the three formulations that describe the free-flow, transitional flow and congested flow. These formulations can be incorporated into the model using a CLIP function and the breakpoints for these three regimes are determined by the speed of the vehicle. This condition is actually a deduction from the breakpoints that are used in the three-regime linear model.

The values of the densities thus obtained are averaged and the average density of the traffic stream at any given time during the collection period is determined. The density thus obtained is again treated with NOISE function in order to get realistic values. The NOISE function is defined in such a way that only positive values are taken
in obtaining the density. When the NOISE function becomes zero then also the model
does not allow the density to reach zero in order to achieve the realistic conditions. Thus
this model represents both the stochastic and deterministic cases quite efficiently. The
density thus obtained is used in the prediction of the density.

The density for an equal interval of time period is obtained by using the density
at the present time and a proper incremental factor which varies from time to time. As
this model is a continuous simulation model and as it calculates the density for a very
small period of time, the predicted density varies throughout the simulation within a
narrow range and this gives very accurate results as this prediction is based on latest
measurements. The model also takes into affect the fluctuations that are observed in this
prediction process. The density so calculated can be used in predicting congestion by
establishing a set of breakpoints for the various regimes of traffic flow.

This model can predict the traffic flow characteristics in another section of the
highway based on the measurements in this test zone. This could be accomplished by
interrelating the headway between the two traffic streams: the traffic stream in the test
zone and the traffic stream for which the measure of effectiveness are to be predicted.
The speeds of the vehicles could then be obtained by using the three-regime linear model.
From the speeds again the density can be calculated. These parameter are used in
predicting congestion.
6.2 SIMULATION MODEL TO PREDICT CONGESTION

I
NINPUT = 15
I
NLOC = 2
FOR
INPUT = 1, NINPUT = A, B, C, D, E, F, G, H, I, J, L, M, N, O, P
FOR
LOC = 1, NLOC = START, END
T
T(START) = 5/25/35/65/75/95/125/143/165/189/200/233/247/256/266
T
A
TT.K(INPUT) = T(START) - T(END)
A
V1.K(INPUT) = CLIP(DIST/TT.K(INPUT), 0, TIME.K, T(INPUT, START))
C
DIST = 300
A
A
K2.K(INPUT) = (50 - V1.K(INPUT)) / 0.098
A
A
K4.K(INPUT) = (40 - V1.K(INPUT)) / 0.265
A
K5.K(INPUT) = (81.4 - V1.K(INPUT)) / 0.913
A
D1.K = SUM(DUM)
T
DUM(*) = 1/1/1/1/1/1/1/1/1/1/1/1/1/1/1/1
A
KN1.K = NOISE()
A
A
KN.K = 0 - KN2.K
A
DUM.K.K = CLIP(K0.K, AVEK.K, KN.K, 0)
A
K.K = CLIP(DUM.K.K, 0, DUM.K.K, KN.K, 0)
A
AVEK.K = (SUM(K1.K) / (D1.K)) * KN.K
A
K0.K = SUM(K1.K) / (D1.K)
A
KINC.K = RAMP(0.045833, 300)
A
KRED1.K = (KINC.K + K.K)
A
A
A
VRED11.K = 50 - 0.098 * KRED.K
A
A
VRED13.K = 81.4 - 0.913 * KRED.K
A
VRED14.K = 40 - 0.265 * KRED.K
R
PC1.KL = VRED1.K
L
N
X1 = 0
NOTE
INPUT - VEHICLES
NOTE
T - TIME IN SECONDS
NOTE
TT - TRAVEL TIME IN SECONDS
NOTE
DIST - DISTANCE BETWEEN DETECTORS
NOTE
K - DENSITY
NOTE
KRED - PREDICTED DENSITY IN TEST ZONE
NOTE
VRED1 - VELOCITY PREDICTED FOR TEST ZONE VEHICLES
NOTE
PC1 - POSITION CHANGE OF STREAM 1
A
A
KS2PRED.K = 5280 / H2.K
A
A
VRED21.K = 50 - 0.098 * KS2PRED.K
A
A

VPRD23.K = 81.4 - 0.913 * KS2PRED.K

A

VPRD24.K = 40 - 0.265 * KS2PRED.K

R

PC2.KL = VPRD2.K

L

X2.K = X2.L + (DT) * PC2.JK

N

X2 = -200

NOTE

PC2 - POSITION CHANGE OF STREAM 2

NOTE

KS2PRED - PREDICTED DENSITY OF SEGMENT 2

NOTE

VPRD2 - VELOCITY PREDICTED FOR ZONE 2 VEHICLES

NOTE

X2 - POSITION OF STREAM 2

SPEC

DT = 1 / LENGTH = 300 / PLTPER = 1

PLOT

KPRED = K (*,*)

RUN

QUIT
Figure 13. Results of congestion prediction model
6.3 Analysis Of A Freeway Bottleneck: An Application Of The Congestion Prediction Model

The traffic flow on a high volume may be described by studying the travel patterns of the vehicles in the traffic stream. The characteristics of the traffic stream play an important role in the prediction of congestion at a bottleneck. A bottleneck is necessarily a roadway with a flow capacity less than the road ahead and when the traffic volume reaches the capacity on such a facility, then the velocity in this section is much less than that ahead of the bottleneck. A further increase in volume accumulates as a queue in advance of the bottleneck and there will be drastic changes in the traffic conditions in this section of the highway. The demand at a freeway bottleneck consists of vehicles which entered at entrance ramps. If the entering volume is known at each input upstream of the bottleneck and if for each input the percent of vehicles which pass through the bottleneck is known, the traffic flow conditions could be very well depicted using the traffic congestion prediction model developed.

In this analysis, a freeway section as shown in Figure 14 is considered in which the freeway traffic flow conditions are affected by the traffic flow on the entrance and exit ramps. In this study, various scenarios of this section of the freeway are considered in order to study the applications of the traffic congestion prediction approach adopted in this research. Contour maps are developed for each case in order to have a good illustration of the various traffic flow conditions in this section of the...
(1) Entrance ramp 9
(2) Exit ramp 12

Figure 14. Freeway Bottleneck
freeway. In this study the various turning movements that are generated are also calculated. This can be accomplished by determining the percentage of turning vehicles in the entire traffic stream and then calculating the number of turning vehicles based on this percentage.

6.3.1 Case (a) :-

The section of the freeway between the entrance ramp 9 and exit ramp 12 is considered as the freeway bottleneck in this scenario as this section of freeway experiences a high demand due to vehicles moving on the freeway as well as from the vehicles joining the freeway from the entrance ramp 9. In this case, the vehicles that leave the freeway take exit 12 and as they are present in the traffic stream all along the length of the bottleneck, they also contribute to the high density conditions. In this scenario it is assumed that the demand is uniform throughout the peak period. The data for this scenario is input into the model as traffic flow parameters and the results obtained are plotted to develop density-speed ratio contour maps on a Time-Space diagram (Figure 15). The results indicate that the traffic congestion sets in at about 7:20 a.m. and lasts until 8:10 a.m. as the traffic flow during this period is severely restricted due to high volume. The congestion is relieved slowly as the traffic demand tapers down slowly towards the end of the peak period. The ramp control strategies could be well devised if the information regarding traffic flow patterns is well predicted.
Figure 15. Case (a): Contour map obtained when both entrance ramp 9 and exit ramp 12 are open.
Figure 16. Case (b): Contour map obtained when entrance ramp 9 is closed while exit ramp 12 is open
6.3.2 Case (b) :-

This scenario basically considers the effect of closure of the entrance ramp 9 and exit ramp 12 as a method of alleviating the severe traffic flow constraints in this section of the freeway. In this case, the freeway bottleneck is analyzed for a scenario in which the entrance ramp 9 is closed while the exit ramp 12 is allowed to function normally. The contour maps (Figure 16) obtained from this scenario indicate that congestion sets in at about 7:30 a.m. and lasts until 8:05 a.m. This type of traffic flow characteristics may be attributed to the closure of entrance ramp 9. In another case, the exit ramp 12 is closed and it is observed from the results (Figure 17) that the congestion sets in early as compared to other cases and also it takes a longer time for this congestion to relieve. This might be considered as the worst case scenario. In this study, the entrance ramp 9 and exit ramp 12 are closed in order to study the freeway traffic flow characteristics. It is observed from the results (Figure 18) that in this case the congestion sets in very late and lasts for a short time. This is the best possible alternative among the three cases considered.

6.3.3 Case (c) :-

The interchange of locations of entrance ramp 9 and exit 12 is considered in this case in order to study the impact of the traffic on these ramps over the freeway traffic flow conditions. The data input is done similar to case (a) and the results are
Figure 17. Case (b) : Contour map obtained when entrance ramp 9 is open while exit ramp 12 is closed

$K = \text{Density}$

$V = \text{Speed}$
Figure 18. Case (b): Contour map obtained when both entrance ramp 9 and exit ramp 12 are closed.

K = Density
V = Speed
Figure 19. Case (c): Contour map obtained when entrance ramp 9 and exit ramp 12 are interchanged
interpreted to develop contour maps (Figure 19). These results indicate that in this case congestion sets in at 7:30 a.m. and lasts till 8:10 a.m. as a result of high volume during peak period. The congestion period is reduced in this case due to interchange of these ramps.

6.3.4 Case (d) :-

The freeway section considered in Case (a) is studied in this case with a non-uniform demand during the peak period. The demand pattern is shown in figure 20. In this case the entrance ramp is metered and the data is input according to the demand characteristics assumed, into the model. The results (figure 21) of this scenario indicate that traffic congestion sets in at 7:25 a.m. and is relieved at 8:00 a.m. The congestion resulting in this case is mainly due to the high demand during the period 7:20 a.m. to 7:40 a.m. The results of this study indicate that the approach adopted in this research to predict congestion yields desired results.

The results obtained from this case study show that the case when the exit ramp 12 is interchanged with entrance ramp 9 is the most efficient system. It can also be deduced from the results that when exit ramp 12 is closed the congestive conditions set in very early. In case (b) when the entrance ramp 9 is closed, it was observed that the congestive conditions prevail for a very short period of time. The minimum
Figure 20. Peak hour demand profile in the freeway bottleneck section

Figure 21. Case (d) : Contour map obtained when peak hour demand in the freeway bottleneck is defined by the demand profile shown in figure 20
congestion time is observed in the case when both the exit and entrance ramp were closed. In the case where the traffic demand on this facility is non-uniform, it was observed that the congestion resulted only due to the high demand period. In this study the turning movements that take place at the intersections are also determined. These results indicate that the model to predict congestion could be very effectively used in planning various traffic operation strategies. This case study indicates that the model could be very well adapted to conduct pilot studies without actually causing trouble to the users of the highway facility.
7. CONCLUSIONS AND RECOMMENDATIONS

The main objective of this research has been developing a model to predict congestion. The various traffic stream models have been carefully reviewed and are used to simulate the various traffic flow conditions and from the results of this simulation, their relative merits and demerits have been discussed. In this research, the three-regime linear model has been used in the model to predict congestion. This appears to be a very good traffic stream model as it considers three regimes of traffic flow namely, free-flow, transitional flow and congested flow in determining a relationship between the speed and density of the traffic stream.

The traffic stream models are analyzed using the techniques of simulation which appears to be a good methodology as according to this procedure, all these models which have been developed from the data obtained on highways, are again made to represent the same conditions. As the traffic conditions that are applied in this study are generalized, the results obtained truly reflects the capability of the model. This technique also made it possible to bring all these traffic flow models on to one single platform of testing procedure so that any kind of errors resulting due to the favorable traffic conditions to any particular model is minimized.

In the model developed to predict congestion the data that is obtained from the detector system is input in the form of arrays which is a very flexible and convenient
method of representing data. The advantage of this model is that it can be accommodate any variation in the variables that are used to describe this model. This model is so designed that it can be adapted to almost every set of data at any highway facility by changing a few variables, that vary from place to place, to represent the traffic conditions at that particular facility. The model also has the flexibility to change the time interval of taking measurements in the test zone of the highway. The presence type detector system assumed for this model to take measurements is also a very accurate system.

The various measures of effectiveness used in this research in describing congestion are found to very effective in defining the limits of congestion. These parameters can be obtained from all the traffic stream models very easily. A detailed study conducted on these variables has provided a good insight into the fundamentals of traffic flow theory which assisted in developing the model to predict congestion. In this model the three-regime model which is used to describe the traffic flow characteristics, inspite of being a good model has some deficiencies as this model is developed based on the data obtained for some particular study area only. Hence, it is desirable to obtain data for a large and develop a model which could be very easily incorporated into the present model. This would give more accurate results.

The effectiveness of the model developed in this research is mainly due to the dynamic modeling that is adopted in developing it which makes it possible for the model to update its data every time there is a change in data, and correspondingly give new
results. This makes it a very convenient and efficient model. A study conducted on a simple case of a freeway bottleneck has shown that the model is very efficient in handling the complex traffic flow characteristics that are experienced in this case. It is also useful in predicting the turning movements of traffic.

The results of this research indicate that much progress could be made in this field of transportation which will have a tremendous impact on the development of transportation infrastructure. Research could further be continued to develop this model into a possible user friendly software, so that, it benefits the operators of transportation facilities who can obtain information very quickly from this model.

This research may also be extended to developing this model to such an extent that it can be used in the Advanced Traffic Management System (ATMS) and also in Advanced Driver Information System (ADIS) that are proposed for the Intelligent Vehicle/Highway System (IVHS). The prediction of congestion is a very important function as this helps in taking precautionary measures to avoid congestion and increase the efficiency of the highway system thereby, saving billions of dollars to the country’s economy.
8. Bibliography


10. Drew, Donald R., Keese, C. J., *Freeway Level Of Service As Influenced By Bibliography*


Bibliography


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