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DEVELOPMENT AND VALIDATION OF A CAR FOLLOWING MODEL FOR
SIMULATION OF TRAFFIC FLOW AND TRAFFIC WAVE STUDIES AT
BOTTLENECKS

Benekohal, Rahim Farahnak, Ph.D.
The Ohio State University, 1986

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UMI
DEVELOPMENT AND VALIDATION OF A CAR FOLLOWING MODEL FOR
SIMULATION OF TRAFFIC FLOW AND TRAFFIC WAVE STUDIES
AT BOTTLENECKS

DISSERTATION

Presented in Partial Fulfillment of the Requirements for
the Degree Doctor of Philosophy in the Graduate
School of The Ohio State University

By
Rahim Farahnak Benekohal, B.S.Ag.E., B.S.C.E., M.S.

* * * * *

The Ohio State University
1986

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ABSTRACT

A car following model, CARSIM, is developed and validated under various traffic conditions at microscopic and macroscopic levels. It realistically simulates normal flow as well as stop-and-go operations. CARSIM provides a marginally safe following distance for all vehicles. It uses variable reaction times for drivers, and takes into account the start-up delay of the drivers. CARSIM may be used as a single regime model or as a dual regime model.

Different maximum deceleration rates one for before the disturbance and another for after the disturbance is used to reflect the slower reaction of drivers to accelerate than to decelerate. A 2 second start-up delay is suggested for the stopped cars. When the flow is not in the steady state condition, the time headway computed as the reciprocal of volume is not equal to the mean of vehicles' headway.

After an extensive validation the model is used to study the propagation of traffic waves at a microscopic level and the effects of mainline traffic control on congestion.
alleviation. The effects of the lead car's deceleration rates and traffic volume on the propagation of slowdown and stopping waves are investigated. A set of equations and graphs are developed to find the queue length, speed of wave propagation, location of a wave, time, and the number of vehicles affected by the waves. The rates the vehicles will be affected by the slowdown waves are 2.75, 2.13, and 1.71 sec/vehicle, and the speed of propagation of the slowdown waves are -9.1, -11.8, and -14.8 ft/sec; and that of the stopping waves are -9.1, -11.74, -14.53 at traffic volumes of 1200, 1500, and 1800 vph, respectively.

The effects of an incident the propagation and dissipation of a slowdown, stopping, starting, and recovering waves are investigated. The slopes of the waves are -9.41, -9.17, -13.51, and -12.38 ft/sec., respectively. The effects of a 3 minute incident lasted about 10 minutes. A strategy which guides the vehicles before reaching the bottleneck is compared with not-guiding. The average number of stopped vehicles decreased from 186.2 to 114.0 (26%). The average time a vehicle spent to travel the section is reduced in spite of a deliberate delay that is caused to upstream traffic by asking them to slow down. When the traffic was guided the congestion is eliminated 141 seconds faster.
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CHAPTER I
SUMMARY

The following three sections provide a summary of the research performed and includes its findings. Each section gives the summary of one chapter and contains references to the pertaining appendices.

1.1 MODEL DEVELOPMENT

Car following studies are attempts to understand the relationships among driver-vehicle units and their behavior in the surrounding environment. A realistic representation of drivers interactions in a car following situation is a crucial part of a microscopic simulation model. Basically, there are two types of car following models: linear and non-linear. In the linear models the response is directly proportional to the stimulus which is the differential speed. However, in the non-linear models the response is a function of differential speed and spacing.

A car following algorithm is a vehicle advancing mechanism which facilitates the proper movement of vehicles in
the system. The algorithm should replicate the real world traffic flow with a required degree of accuracy. A compromise ought to be made between the complexity of the realistic representation of actual traffic behavior and the simplicity of the model.

The available car following algorithms have the following shortcomings: a) they assume that a single regime model can represent congested and non-congested conditions. b) they may not realistically simulate the stop-and-go conditions because the start up delay of a vehicle is not taken into account, or the vehicle is not allowed to have a speed less than a certain value. c) the trajectories from these models are not compared to the real world vehicle trajectories when the traffic operation changes from a free flow to a jam density and back to the free flow condition.

The objective is to develop and validate a car following model with realistic features that can be used for the simulation of traffic flow in normal as well as in stop-and-go situations. Then, utilize the model for the investigation of the propagation of traffic waves at microscopic level and for the evaluation of mainline traffic control.

Information related to car following and traffic dynamics from over seventy publications in the past three decades are put together; see appendix A. In addition to this,
studies pertaining to the dual behavior of traffic in congested and in non-congested conditions are reviewed. Over thirty publications supporting the notion of the difference between congested and non-congested traffic behavior are summarized, see appendix B. The state of the art in microscopic freeway simulation is discussed in appendix C. The need for a car following model to handle normal and stop-and-go traffic flow in a realistic manner and provide the desired level of detail for traffic wave studies are outlined.

A car following model, CARSIM, is developed and tested under various traffic conditions (chapter 3). CARSIM can be used for simulation of freeway and street traffic under normal as well as stop-and-go conditions. It can simulate situations ranging from free flow to jam density. Furthermore, the dual behavior of traffic is taken into consideration and the programmer has the option to use CARSIM as a single regime or as a dual regime model. Moreover, variable reaction times are used for different drivers, and the start-up delay of drivers are taken into account. In its present form the model uses passenger cars and trucks; other types of vehicles can easily be added.

The driver-vehicle units and the characteristics associated with them are very important considerations in this
model. A set of constant or variable characteristics is assigned to every man-machine unit upon generation of the entity. Each vehicle tries to maintain a desired speed when the traffic and the roadway conditions permit, or tries to achieve a desired speed without sacrificing the safety requirements. Every single vehicle is advanced to a new position throughout the system at every update interval (1 second). Before advancing a vehicle to a new position it is always checked to see if there is enough space for this vehicle to react to an adverse situation and to reach a safe speed by administering an acceptable deceleration.

CARSIM provides a safe following distance for all vehicles. Thus no collisions are allowed in the simulation model. The speed and the location of each vehicle is updated such that a non-collision constraint is always satisfied. In the mean time, the acceleration/deceleration values are kept within the boundaries of acceptable limits.

The following vehicle is asked to stop when the lead car has stopped or has a speed of less than 0.5 fps, and the speed difference between the lead car and the following car is less than 3, and the spacing between the two cars is less than the sum of lead vehicle's length plus a constant of 10 feet. Otherwise, a proper acceleration or deceleration rate is computed by the acceleration routine.
In the acceleration routine a maximum of five different acceleration or deceleration values are calculated for each vehicle. These are: 1) acceleration to reach the desired speed constrained by the mechanical ability of the vehicle; 2) acceleration to reach the speed limit or the desired speed; 3) acceleration considering the start-up delay of a stopped vehicle; 4) acceleration from car following; 5) acceleration satisfying the non-collision constraint. Depending on the state of vehicle, one of these values is used to update the speed and location of the vehicle. The selected acceleration or deceleration is always within the range of acceptable values.

The non-collision constraint requires enough spacing between vehicles such that after moving the follower to its new position, there is enough distance for the follower to react to its lead car performance and to reach a safe speed without causing a collision.

The features of CARSIM can be summarized as:

1. It provides a marginally safe following distance for all vehicles.
2. It assigns different driver reaction times to drivers.
3. It can handle normal as well as stop-and-go traffic operations.
4. It takes into consideration the start-up delay of stopped vehicles.
5. It uses different reaction times for a driver in congested and non-congested traffic.
6. It reproduces the effects of severe kinematic disturbances.
7. It can be used as a single or dual regime traffic simulation model.

For more detailed information, one should refer to chapter 3 and appendices F, G, and H. The discussion about the main components of a microscopic simulation model is given in appendix F, and the descriptions of the routines of this model are included in appendix G. In addition to these, a sample copy of the computer program is given in appendix H.

1.2 MODEL VERIFICATION AND VALIDATION

As a partial verification effort, acceleration/deceleration patterns, velocity change patterns, trajectories of vehicles, and spacing between vehicles are examined, see appendix D. Various artificial disturbances are induced to a platoon of 15 vehicles and their effects at microscopic and macroscopic levels are investigated.

At the microscopic level, the effects of a regular disturbance, an emergency stop, and a stop-and-go operation on
individual vehicle trajectories, speed, and acceleration/deceleration, are analyzed at high and low volume levels. The disturbances propagate deeper along the line of vehicles and cause higher speed reductions when the volume is higher. However, at the lower volume the effects of the disturbances decrease along the line of vehicles and finally vanish. The deceleration rates of the following cars are less than that of the lead car. The non-oscillatory and damping propagation patterns generated by the model are realistic and desirable.

At the macroscopic level, the effects of the regular disturbance and an emergency deceleration on average speed, density, volume, and average headway are examined at high and low volume levels.

The average time headway is computed by either a simple or an exact method. In the simple method the average headway is computed by finding the reciprocal of the volume. However, in the exact method it is the arithmetic mean of the individual vehicle's headway. It is observed that these two procedures do not yield the same average headway when the flow is disturbed. There is a considerable difference when a platoon is in an acceleration or deceleration phase; but, at the steady state the difference is not noticeable. The maximum difference occurs at the beginning
of the acceleration phase. The values computed from the simple method are consistently higher than those from the exact method. The maximum differences are 0.08799 sec. and 0.04431 sec. at 1800 and 1200 traffic volume levels, respectively.

Similar results are obtained about the traffic volumes. At the steady state flow conditions the difference is negligible (less than 0.5 cars). However, when traffic is going through a disturbance this difference cannot be ignored. The volume computed from the average headway is always higher than that computed as the product of the speed and density. When the traffic suffers from the regular disturbance, the differences are 47.31 and 51.32 vph at 1200 and 1800 volume levels. These differences are even larger when a severe kinematic disturbance is induced.

Extensive verification and validation efforts are undertaken to ensure that the simulation model is properly designed (chapter 4). Several input parameters are systematically varied and the sensitivity of the system outputs are carefully examined. The parameters are: maximum deceleration rate, compliance level, start-up delay, reaction time, buffer space, speed for buffer space change, and traffic mix.
The concept of using different maximum deceleration rates is introduced. Using two different maximum deceleration rates, one for before the disturbance and another for after the disturbance provides trajectory plots very close to those obtained from field data. This also reflects the slower reaction of drivers to acceleration than deceleration when going through a kinematic disturbance. The notion of using different maximum deceleration rates is merely for computational purpose. Using a single deceleration rate for all densities provides a good fit for either before the disturbance or after the disturbance, but, not for both cases. Moreover, it does reflect the difference in the car following behavior before and after a kinematic disturbance. It was also noticed that the buffer space for vehicles within the disturbance is less than the values used for vehicles in free flow conditions.

Comparisons of the actual trajectory plots with the simulation results when a start-up delay of 0, 1, 2, 3, or a value which is a function of the reaction time of the driver, revealed that a realistic car following model ought to consider the start-up delay of the drivers. A 2 second start-up delay for all drivers; or 1 second for drivers with a very short reaction time and 2 seconds for the majority of the drivers is suggested.
The reaction time of drivers was increased by 0.0, 0.1, 0.2 and 0.3 seconds and the effects on speed, density, and average headway indicated that the consequence of using longer reaction times is more pronounced during the acceleration phase than the deceleration phase.

The validation of this model is performed at microscopic and macroscopic levels. For the microscopic validation the trajectories and speed change patterns of four different platoons of vehicles covering a wide range of densities (free flow to jam density) are compared to the simulation results. The speed change patterns obtained from the simulation models replicate the real world situations very realistically. The trajectory plots generated by the simulation models for before a severe kinematic disturbance and after that are very close to the plots from the field data. The simulation plots reproduced the situations where vehicles are forced to stop for a while and then start moving again.

For the macroscopic validation the overall performance of the simulation models are compared with that of three different platoons. The variation of flow parameters, the fundamental relations of traffic flow, the relationship between simulation results and field data, and the effects of stochastic input values on response variables are investigated.
Three flow parameters: speed, density, and volume for the platoons are computed and compared with the average values obtained from the simulation model. The simulation model replicates the behavior of the platoons going from a speed of about 80 fps to a speed of zero in less than one minute. The time a simulated platoon reaches the jam density is very close to that of the actual platoon. The platoon volume which is the product of speed and density is not recommended to be used for comparison purposes, especially when there is a breakdown of flow.

The relationships between speed, density, and volume for all platoons are computed from the field data and the simulation models. The speed-density graphs obtained from the simulation models and the field data show a non-linear relationship and the existence of hysteresis phenomenon. The speed-volume and volume-density plots from the simulation produce loops which are very similar to those from the field data. The graphs illustrate that the magnitude of change in speed, density, or volume when a platoon is going into a kinematic disturbance is not the same when coming out of the disturbance because of the dual behavior of traffic.

Three kinds of regression analysis are performed to examine how well the simulation and the real world results
are related. These are: 1) regression of individual simulation replication versus field data; 2) regression of the average of the simulation runs versus field data; 3) all simulation replications combined versus field data for every platoon.

The regression of speed and density obtained from the individual simulation runs versus the field data yield linear relationships with slopes close to unity and small y-intercepts. The R-Squared values for the speeds are 0.98 and higher, for densities 0.97 and higher, and for the volume 0.80 and higher.

When the average of 5 replications is used the results are very close to the previous case. The R-Squared values for the speed and density are at least 0.98, and for the volume at least 0.84. Graphical presentations of the simulation results versus the field data show that, the points are scattered along the line y=x. This indicates that the model values are proportional to the real world values with a constant of variation very close to 1.

The regression of the combined simulation results versus the field data yield the same slope and y-intercept, as it is expected. However, the variance of the slopes and y-intercepts decrease to almost one-half of the values obtained when the average values are used. The ANOVA table
showed a lack of fit for the linear models. However, the plot of the residuals versus time and the predicted variable did not show any definite trends, but rather a cluster of points around line of $y=0$. Such a horizontal band is an indication of the adequacy of the linear model. The false detection is due to the fact that in every replication the successive points are correlated. This correlation cannot be ignored. Thus, the lack of fit test is not appropriate for this analysis and combining the simulation results is not appropriate here.

The range of variation of the response parameters due to the stochastic input values are very narrow and reasonable indicating that the response variables are fairly stable. The consistent patterns in all of the response variables support the validity of this simulation model, and its ability to perform under a variety of traffic conditions.

After extensive validation, the model is used to study the propagation of traffic waves at microscopic level and to evaluate the effects of mainline traffic control on congestion alleviation.

1.3 WAVE ANALYSIS

For the first time, the propagation and dissipation of traffic waves at microscopic level is investigated. First,
the effects of deceleration rates of the lead car on the propagation of traffic waves are examined. Then, the effects of traffic volume on the propagation of slowdown waves and stopping waves are investigated. Later on, the effects of an incident is simulated and the dynamics of four different traffic waves are analyzed. Finally, the consequences of mainline traffic control on congestion alleviation are discussed (chapter 5).

The leader of a group of 200 cars decelerates to a complete stop at a rate of 4, 8, 12, and 16 ft/sec/sec and forces all vehicles to stop. (Average values of five independent replications of each situation are used for analysis.) The effects of the lead car's deceleration rates on the propagation of the slowdown waves are pronounced for the first group of 10 vehicles, but become less important after the propagation reaches more vehicles. A slowdown wave caused by a higher deceleration rate reaches the cars faster and earlier than the one by the lower deceleration rate.

The equations and the graphs to find the location and the time a slowdown wave will affect a certain vehicle are developed. The slopes of the equations for the slowdown waves are approximately -25 ft/vehicle, 1.73 sec/vehicle, and -14 ft/sec when the arrival volume is 1800 vph. When
the average values of 20 simulation runs are used for analysis, the slope is \(-14.45 \text{ ft/sec}\).

The propagation of the stopping waves when the lead car decelerates at 8 and 16 ft/sec/sec are graphically presented, and the equations are developed. These can be used to find queue length, speed of propagation, location of stopping wave, time, and the number of vehicles affected at traffic volume of 1800 vph.

The slopes of the equations for the stopping waves propagation give the average spacing of the vehicles when they stop. The average slope is approximately 25 feet which is very reasonable. The slopes of the equations for the stopping waves are approximately \(-24.9 \text{ ft/vehicle, 1.71 sec/vehicle, and } -14.5 \text{ ft/sec}\). The lead car's deceleration rate affects the rate of the wave propagation in the beginning, but does not have noticeable effects later on. On the other hand, it does affect the time the wave reaches a certain vehicle. The average stopping wave propagated backward at a speed of \(-14.48 \text{ ft/sec}\) for the combined data.

The effects of traffic volume on the propagation of slowdown and stopping waves are investigated when the leader of a group of 100 cars decelerates at a rate of 8 ft/sec/sec to a complete stop. The volume levels are 1200, 1500, and 1800 vph. At all three levels an initial rapid
propagation is observed. Beyond the initial phase, the difference in the number of vehicles the slowdown waves affect at different volumes do not increase considerably from location to location.

The graphs and equations to present the relationships between the time, location, and the number of affected vehicles are given for different volume levels. The rate the vehicles will be affected by the slowdown waves are 2.75, 2.13, and 1.71 sec/vehicle at 1200, 1500, 1800 volume levels, respectively. The speeds of the propagation of the slowdown waves for different volume levels are -9.1, -11.8, and -14.8 ft/sec, respectively. The wave propagates faster at higher volume than at lower volumes.

Similar results are obtained about the effects of traffic volume on the propagation of the stopping waves. The graphs and equations developed for the stopping waves can be used to find the queue length at a given time, number of vehicles in the queue, and the rate of propagation. The speeds of propagation of the stopping waves are -9.1, -11.74, -14.53 ft/sec at traffic volumes of 1200, 1500, and 1800 vph, respectively.

The effects of an incident, which blocked the road for 3 minutes, on the propagation and dissipation of a slowdown, stopping, starting, and recovering waves are investigated
at a traffic volume of 1200 vph. While the cars in front of the queue accelerate to reach the desired speed the arriving vehicles join the rear of the queue. Through this process the location of the bottleneck moves upstream towards the upcoming traffic.

The propagation and dissipation of the slowdown, stopping, starting, and the recovering waves are graphically presented and their relationships are examined. At the point a starting wave reaches a stopping wave, the queue of the stopped cars is eliminated. The earlier these two waves meet the faster the queue is dispersed. The recovering wave and the slowdown wave will not intersect, but may get closer to each other. The slopes of the lines representing the slowdown wave and the stopping wave are almost the same (almost parallel). Similar relationships exist between the stopping wave and the recovering wave.

The equations and graphs for the starting wave and the recovering wave can be used for different volume levels or road blockage duration. However, those for the slowdown wave and the stopping wave are only for this particular condition. When the distance between the stopping wave's line and the starting wave's line decreases the queue shrinks, otherwise, the queue remains constant or builds up. The slopes of the slowdown, stopping, starting, and
recovering waves are -9.41, -9.17, -13.51, and -12.38 ft/sec, respectively. The negative slope of the lines indicate the backward movements of the waves. However, the slowdown wave and the recovering wave move forward when it is about to completely eliminate the incident's effect, as it can be seen on the graphs.

The disturbance may last even after the physical removal of the blockage. The effect of the 3 minute incident lasted about 10 minutes, because the arriving cars join the end of the queue and physically block the road.

A control strategy which reduces the number of vehicles joining the rear of the queue is suggested. The attempt is to make the recovering wave reach the stopping wave fast. This can be done by delaying the arrival of upcoming mainline traffic to the bottleneck. A traffic management policy which guides the vehicles before reaching the bottleneck is compared with the common practice of allowing traffic to recover by itself (not-guiding).

The criteria used for comparison of the strategies are: the duration of slowdown, duration of stoppage, time vehicles spent in the system, delay due to control policy, density, and the throughput of traffic.
The duration of the slowdown is reduced from 617 sec. to 476 sec., and the stoppage from 565 sec to 426 sec. The average number of vehicles affected by the incident is reduced from 197.6 to 180.2. The average number of stopped vehicles decreases from 186.2 to 114.0. The average time a vehicle spend to travel the three mile section is reduced in spite of the deliberate delay that was caused to upstream traffic by asking them to slow down. Thus, in some conditions it seems that the traffic can reach its destination earlier by slowing down before reaching the bottleneck.

More important than the time saved for the mainline traffic is the fact that the congestion is cleared earlier and the operation is returned back to the normal condition quickly. When the traffic is guided the congestion is eliminated 141 seconds faster. The delay caused to the motorist when they are guided is less then when they are not guided. There are also some improvements on the overall speed and the throughput of the traffic when it is guided.

The results from this "crude" mainline traffic control strategy are limited to the outlined conditions. More research on this subject is needed.
CHAPTER II
INTRODUCTION

2.1 INTRODUCTION TO TRAFFIC SIMULATION

Traffic simulation is a dynamic discipline and is growing very rapidly. In traffic simulation a complex system such as the traffic environment is represented by a more manageable system (model) and this model is translated into a proper language that a computer can understand. A good traffic simulation model should realistically reproduce the behavior of traffic on the road. The output of a traffic simulation model is the accumulated statistics which are used for analysis of dynamic behavior of traffic.

An efficient traffic simulation model can be employed as an analytical tool in transportation system management and the results of the simulation can be used as a basis for selecting the best traffic operation alternatives. Furthermore, a realistic and a reliable traffic simulation model can be used as a research tool for investigation of behavior of traffic flow under different circumstances.
Generally, traffic flow simulation models are classified as macroscopic or microscopic models. In the macroscopic models, freeway traffic is represented in terms of aggregate measures such as traffic volume, space mean speed, and average density. This category of models sacrifice a great deal of detail but they are efficient (Wicks and Lieberman 1980; Payne 1979). These programs are designed to deal with problems of larger scale. When vehicles are handled in a group, i.e. a platoon of cars, the model is called mesoscopic. Most of the time the macroscopic category includes this type of models, too.

In the microscopic models, the individual vehicle is represented by a set of variables (such as vehicle type, speed, reaction time, acceleration, position, etc.) which are updated at regular time intervals. These models are highly detailed, and more accurate than the macroscopic models (Radelat, 1981; Wicks, Lieberman 1980). However, they require larger computer resources. The most recent and comprehensive in this category is the INTRAS model. Recently an attempt has been made to combine these two approaches into a single integrated simulation model, TRAF, which would make possible to use one approach or a combination of the two approaches.
2.2 TRAFFIC BEHAVIOR ANALYSIS

Generally, there are two different theoretical approaches to describe traffic flow on a roadway: 1) Car-following approach, 2) Hydrodynamic approach

2.2.1 CAR FOLLOWING APPROACH

In a car-following approach traffic is considered as being made up of discrete particles (cars) and the interaction between these particles is examined. This approach corresponds to the microscopic simulation models. The car-following models are in the form of a stimulus-response equation and in general can be represented as (Gerlough, Huber; 1975):

\[ \text{Response (t+T)} = \text{Sensitivity} \times \text{Stimulus (t)} \]

where the response usually is the reaction of the following driver to the motion of the lead car, and T is the reaction time of the following driver.

When the response of the following car to acceleration or deceleration is proportional to the difference in the relative velocity of the lead and the following car, the model is called a linear car-following model. In the linear car-following models the sensitivity has a constant value. For a given difference in velocity between a lead
and a following vehicle, the following driver would respond independent of the spacing of the two cars. A more realistic car following model considers the sensitivity to be inversely proportional to the spacing between cars (Gazis, et al; 1959). This kind of model is called a nonlinear car-following model.

Another approach for the car-following study (Rockwell, Treiterer; 1968) suggests that the following car's acceleration after a delay, T, is a linear combination of the lead car's speed and acceleration (the following car would duplicate the acceleration of the lead car after a lag of T).

2.2.2 HYDRODYNAMIC APPROACH

Unlike the car-following approach, the hydrodynamic approach is concerned with the overall statistical behavior of the traffic stream, such as density, speed, and volume. The traffic is considered as a compressible fluid having certain density and velocity, and the analysis is based on a partial differential equation expressing the conservation of matter. This approach corresponds to the macroscopic simulation models.

In this approach, the behavior of traffic at a bottleneck is assumed to be acting in a shock wave-like manner.
A shock wave is generated when two dynamic waves intersect each other (Lighthill, Whitham; 1955). A shock wave is also defined as the motion of propagation of a change in concentration of vehicles (Gerlough and Huber; 1975). The equation to describe the propagation of a small disturbance in traffic stream can be written in terms of shock wave velocity. An important application of shock wave analysis is related to traffic control at bottlenecks.

A bottleneck is defined as a section of the roadway where the capacity is less than the capacity of upstream and downstream sections; or a stretch of the roadway which exhibits highly congested and unstable conditions resulting in regions of low speed. A bottleneck may be caused by certain physical or control system conditions, or by high traffic density alone (Treiterer et al; 1970).

The hydrodynamic approach (macroscopic level) has been widely used for the investigation of the propagation of shock waves. However, thusfar the propagation and dissipation of traffic waves at the microscopic level have not been investigated. The concept of traffic wave dynamics can easily be explained by the car following approach once this investigation is performed.
2.3 THE PROBLEM STATEMENT

Most of the freeway simulation models are developed for the purpose of ramp metering, corridor traffic management, or priority lane operation evaluation. A simulation model that can realistically represent the traffic behavior in bottlenecks and that can be used for traffic wave analysis is needed.

In the past decade a considerable number of computer models have been developed, the summaries of some of them are given in the FHWA Handbook On Computer Models For Traffic Operations Analysis (Byrne et al, 1982). Most of the models are designed to prevent worsening of the traffic congestion by ramp metering or by diverting of the traffic to an adjacent road. Ramp metering controls only the vehicles entering the freeway and does nothing for the mainline traffic approaching the bottleneck. Sometimes, the ramp is blocked by the stopped vehicles and the disturbance propagates upstream and effects more vehicles on the roadway.

One solution seems to be guiding and control of upstream mainline traffic before it reaches the bottleneck with or without ramp metering. Simulation will be a perfect tool to investigate the effectiveness of this approach. From the available simulation models none of them can be utilized for this purpose.
Available simulation models cannot handle the stop-and-go operations. Thus, should not be used when the vehicles stop and start. They also do not take into account the start-up delay of stopped cars. Therefore, one cannot accurately study the propagation of traffic waves at microscopic level using these models. Furthermore, the dual behavior of traffic in congested and non-congested traffic conditions is not taken into consideration by these models.

The microscopic approach for traffic wave analysis explains the concept of wave propagation easily by means of real world examples and does not make the assumptions the macroscopic approach does. It also yields more accurate results than the macroscopic approach. To find the speed of a shock wave the macroscopic approach uses the state of traffic before and after the disturbance, while the state of traffic after the disturbance is yet to be determined. Besides this, the transition from one state to another state is not taken into consideration and it is assumed that there are continuous relationships between traffic parameters. On the other hand, the microscopic approach does consider the transition state and does not assume that the relationships are continuous. It uses only the present state of the traffic flow to find the speed of propagation of a traffic wave and does not use an assumed after the disturbance state.
A simulation model that represents realistically the traffic movement in bottlenecks, and provides accurate information about traffic waves propagation is needed for this investigation. A reliable car following model that can replicate the real world traffic dynamics in a wide range of densities would be an essential part of this model. Such a realistic model would provide a very powerful tool to study traffic flow, and to investigate the propagation and dissipation of kinematic waves on the roadways. Furthermore, such a model could be used to perform an accurate comparison of alternative traffic control strategies.

2.4 RESEARCH OBJECTIVES

The objective of this research is to develop and validate a realistic and a reliable microscopic model for bottleneck traffic flow simulation and utilize it to study the propagation of traffic waves at the microscopic level. Specifically, the research will be extended to explore the followings:

1. Evaluation of the effect of guiding upstream mainline traffic before reaching the bottleneck.
2. Investigation of propagation and dissipation of kinematic waves at the microscopic level.
3. Development of a car following model with realistic features to handle free flow as well as stop-and-go situations.

4. Formulation and validation of a simulation model to achieve objectives 1 thru 3.

5. Sensitivity analysis on the model parameters, and comparison of simulation results with field data.

2.5 ADVANTAGES OF TRAFFIC SIMULATION

The traffic simulation approach is more appealing partially because of the following reasons:

1. Traffic simulation is less costly.

2. The results are obtained quickly.

3. Sensitivity analysis is performed easily.

4. Traffic disruption is avoided.

5. No physical changes are necessary for experimental purpose.

6. Under identical traffic conditions one can evaluate alternative control policies. In the field studies one can hardly create identical traffic conditions.

7. With traffic simulation one can evaluate an effective, efficient and above all, safe solution prior to implementation of the policy.
CHAPTER III
DEVELOPMENT OF A CAR FOLLOWING MODEL

3.1 INTRODUCTION

A car following study is an attempt to understand and to formulate the interaction among drivers when they follow each other. There has been numerous studies in the past three decades. The summary of some of the studies are given in appendix A.

In simulation of traffic flow, the car following algorithm is the vehicle advancing mechanism by which the velocity and the position of vehicles are determined in every scanning time interval. It facilitates the proper movement of vehicles from the time of entry until they leave the system. The algorithm should represent the actual process of traffic movement with a certain degree of realism.

Some of the important factors to be considered in developing a car following model are: proper advancing of vehicles through the system, accident prevention, operation within the boundaries of physical and behavioral limitations of driver-vehicle units, and realistic representation of the actual system in the computer.
A compromise ought to be made between the complexity of realistic representation of actual traffic behavior and the simplicity of the model. A very realistic model can be too complicated that it may not have any practical value. What should be compromised? This question cannot be answered in its general form. It depends on the purpose of the model and the situation the model is developed for. A compromise can be made on certain characteristics of the driver or the vehicle and still be able to obtain useful results from the simulation model.

Let us assume that we are interested in finding the overall travel time of a vehicle on a given section of a freeway regardless of the magnitude of acceleration or deceleration the vehicle undertakes on that section. A car following model which gives an overall travel time close enough to the actual travel time will serve the purpose. However, if our intention is to study the acceleration or deceleration patterns of vehicles and their behavior in speeding up or slowing down (a vehicle suffering from kinematic disturbance), then the same car following model would not be useful for our purpose. The reason is, the model does not really give us any information on how the cars traveled through the section, only how long it took for them to travel that section.
In the early car following simulation models an important behavior of traffic flow at higher densities was overlooked. The fact that traffic flow does not behave the same in congested and non-congested situations, especially in the transition from one to the other, has not been taken into consideration in developing of traffic flow simulation models. Furthermore, an important step in the validation of a car following algorithm which is the comparison of trajectory plots is not fully performed in the majority of the models. For instance, in validation of the INTRAS car following model only trajectory data for "before and within" disturbance was used, however, "within and after" disturbance trajectory comparison was not reported; see Figure 1.

Understanding the dual behavior of traffic flow (appendix B) would help to develop an algorithm that can be used for single regime as well as dual regime flow. A car following model, CARSIM, has been developed which can be used in traffic concentrations ranging from free flow to jam density. It can also realistically handle stop-and-go situations on a freeway. Before giving the details of the algorithm, the model development philosophy and the simulation language used will be briefly discussed. At the end of this chapter the capabilities and features of CARSIM will be outlined.
Ohio State Vehicle Trajectories Platoon of Twenty Three Vehicles Showing Paths of Vehicles Numbers 1, 5, 10, 15, 20, and 23.

**Figure 1:** INTRAS trajectory comparison. (From INTRAS Manual Volume I)
3.2 MODEL DEVELOPMENT PHILOSOPHY

Accurate representation of a freeway traffic flow in a simulation model depends upon the quality of reproducing the real-world vehicular dynamics and the interactions between drivers and the environment. The vehicular dynamics and interactions are handled by means of a vehicle advancing mechanism based on a car following algorithm. The driver's interaction with the vehicles and the immediate environment is deterministically or stochastically assigned through a set of characteristic parameters to a man-machine unit.

Individuality of a driver-vehicle unit and the characteristics associated with it are a very important consideration for this model. Each driver will try to maintain a desired speed if the traffic condition and the roadway configuration permit, or will try to achieve a desired speed without sacrificing the safety requirements. Every single vehicle will be advanced to a new position throughout the system at every updating time.

A set of constant or variable characteristics will be assigned to every man-machine unit upon generation of the unit. Only two categories of vehicles are used because in dense traffic maneuverability of vehicles are limited. The response of a driver in the form of acceleration or decel-
eration to a certain stimulus will be computed using these characteristics. If the computed response value is within the boundary of the acceptable values and if it satisfies the safety criteria, then it will be used by the vehicle advancing mechanism to update the location and the speed of the vehicles.

In advancing a vehicle to a new position it is always checked to see if there is enough space for this vehicle to react to an adverse situation and to reach a safe speed by administering an acceptable deceleration. It is unrealistic to assume that a driver could react immediately, likewise it is untrue to assume that there is always an absolutely safe spacing between vehicles in a car following situation. Treiterer (1975 Final Report) reported that most of the time the spacing in high density freeway traffic is marginally safe, but not absolutely safe.

A reliable car following model that can replicate the real-world traffic dynamics in a wide range of densities is an essential part of a microscopic freeway traffic simulation model. Such a realistic model, along with proper use of simulation techniques, and a powerful simulation language, such as SIMSCRIPT 11.5, would provide a very useful analytical tool to study accurately the traffic dynamics on the roadways.
3.3 SIMULATION LANGUAGE AND APPROACH

The language used for programming of this simulation model is SIMSCRIPT II.5. SIMSCRIPT II.5 is very suitable for programming of this simulation model, and provides a useful study tool with a lot of capabilities and features. For more information about SIMSCRIPT II.5 consult Russell (1982, 1983) and SIMSCRIPT II.5 Reference Handbook.

The approach to organize a simulation model can be event-oriented (event-scheduling) or process-oriented (process-interaction). While in some languages the choice of language determines the approach to be used for organization of simulation, in SIMSCRIPT II.5 one can use both approaches.

In the event-scheduling approach, the simulation can be regarded as the execution of a sequence of chronologically ordered events at specified times. An event can contain a set of decisions, calculations, or state-changing actions (e.g. arrival of a car). Between events time elapses but not within an event. For each possible event there is usually one subroutine that keeps track of activities and computations.

On the other hand, in process-interaction approach, every entity is provided a process which is a sequence of
actions that moves the entity through the system. A pro-
cess can be scheduled to start at a certain time, and can
be interrupted or delayed. Usually all the codes required
to change the state of an entity is grouped in one subrou-
tine. An entity may go through several processes before
leaving the system.

The approach used in this simulation model is a combina-
tion of synchronous and asynchronous event-scheduling
approaches. This selection is due to the nature of the sim-
ulation model and the type of statistics to be collected.
The synchronous events can happen at regular time inter-
vals, however, the asynchronous events can happen at any
time. An event-oriented simulation can easily be written in
any general-purpose high-level programming language. How-
ever, process-oriented simulation usually requires a spe-
cial purpose language(Bratley; Fox; Schrage; 1983). One
may use any simulation language or a general purpose lan-
guage to translate the logic of this model to that lan-
guage. For this reason the flow charts and description of
individual routines are included where they might be need-
ed.
3.4 DESCRIPTION OF THE CAR FOLLOWING ALGORITHM (CARSIM)

3.4.1 CAR FOLLOWING ROUTINE

The car following algorithm is the most important part of a microscopic simulation model. It is basically a vehicle advancing mechanism that facilitates the movement of vehicles from one point to another along the road. It computes the speed and the location of each vehicle at the end of every scanning time interval. In conjunction with the acceleration routine it determines the proper acceleration or deceleration the following vehicle should maintain during each updating time interval.

A new car following model, CARSIM, is developed, tested, and used under various traffic density conditions. It can be used for simulation of normal highway or street traffic, or when the traffic is in a stop-and-go condition. With a few changes it may also be used for intersection traffic flow simulation.

Traffic operations can vary from high speed low density to low speed high density conditions, and finally to jam densities. The traffic behavior is not the same for the entire range of the density. Uniform treatment of traffic in all density levels is an oversimplification of actual behavior of traffic. Uniform treatment has been very popu-
lar among traffic simulation model developers, although it is not necessarily the most accurate and realistic approach.

In CARSIM the dual behavior of traffic is taken into consideration and unique features are included in the model. One of the unique features of CARSIM is the capability to handle vehicles that decelerate to a complete stop, and after a while accelerate from a zero speed to their desired speeds in a realistic manner. This feature is extremely important in study of the effects of kinematic disturbances on freeway traffic. During rush hour traffic operation is more likely to be in a stop-and-go situation. An accurate simulation model should be able to replicate this condition, especially, if it is used for the evaluation of peak hour traffic.

CARSIM provides a safe following distance for every vehicle during their life in the system. There are no collisions allowed. A vehicle will be advanced to a position that gives it enough spacing to decelerate to a safe speed or a complete stop if the lead car reduces the spacing by decelerating.

The acceleration or deceleration of a following vehicle is determined in the acceleration routine; and based on this value the speed and the location of the following
vehicle is computed. These values are computed so that the non-collision constraint is satisfied at all times, and the following vehicles acceleration and deceleration are within the limits of maximum acceptable values. The following sections will include the flow charts and details about the car following model.

CARSIM can be programed in any computer language. In appendix H only the translation of this algorithm to SIMSCRIPT II.5 is given. The flow charts for car following and acceleration routines are given for readers who are not familiar with SIMSCRIPT II.5, or would program CARSIM using another language.

Once the vehicle enters the system, this car following model checks whether the vehicle is the first to enter the system (leader), or is a following vehicle. If the vehicle is first to enter the system, then the desired speed of the vehicle is checked. For a vehicle traveling at its desired speed or at the speed limit, an acceleration value of zero is assigned. The speed and the location is updated using the basic equation of motion. For a vehicle traveling slower than the desired speed or the speed limit a proper acceleration rate is computed from the acceleration routine.
CONTINUE ACCELERATION ROUTINE

A2 = (DS.Vf-Vp)/DT

Compute AC from comfortable deceleration routine

A2 < AC, & AC < min of (A4, A5)
YES
AXL = AC

NO

A4 < A2, & A2 < min of (A4, A5)
YES
AXL = A2

NO

AXL = A5

RETURN
ACCELERATION ROUTINE

START

1st car

compute A5

\[ A_5 = 2 \left( \frac{X_t - X_e - V_p \cdot DT - L - K}{DT} \right) \]

return

A1, A2, A3, A4, A5
CAR FOLLOWING ROUTINE

START

1st car? YES

find AXL from acceleration routine

V_P <= V_L + 3 and V_L <= 10 and X_L - X_P >= L + 10

compute velocity = V_P + AXL * DT

velocity < 0

AXL = -V_P / DT

NO

V_P = velocity

AXL = min. of (AXL, 0.0)

AXL = min. of (A1, A2, A3)

AXL = min. of (AXL, 0.0)

driver complies

find AXL from comfortable deceleration routine

NO

V_P = V_P + AXL * DT

X_P = X_P + V_P * DT + AXL (DT) / 2

NO

YES

AXL = 0

NO

YES

V_P = DS * V_P
However, for the first vehicle traveling faster than the desired speed or the speed limit either an deceleration of zero is assigned when the driver does not comply with the proposed speed limit, or a comfortable deceleration rate is computed from the comfortable deceleration routine. After determining a proper value for acceleration or deceleration, the speed and the location of the vehicle is updated using the following equations:

\[ U = U + a*t \]
\[ S = S + U*t + 0.5*a*t^2 \]

On the other hand, for vehicles following the first vehicle the procedure is a little different than that of the leader. The acceleration subprogram is called to determine the acceleration or deceleration rate. The discussion of this routine is given in the next section. A vehicle will be directed to stop at the end of current time interval if all of the following conditions are satisfied:

1. The lead car has stopped or its speed is less than 0.5 fps.
2. The speed of the following vehicle is less than the sum of the lead vehicle's speed plus a constant of 3 fps.
3. The spacing between the two cars is less than the sum of the lead vehicle length plus a constant of 10 feet.
When all three conditions are met, the acceleration is equal to the ratio of speed of the follower to the time interval used for updating, with a negative sign. When the conditions are not satisfied, the acceleration or deceleration is assigned from the acceleration routine and the speed of this vehicle at the end of the update time is computed. This speed is either positive or negative. If the speed is positive and the vehicle was not stopped, then the speed and location of the vehicle are updated. However, if the vehicle was stopped before, then the vehicle is allowed to move and the speed and the location of it is computed. In the second case where the speed is less than zero, the updated speed of the follower is set equal to zero and the acceleration of the follower equal to the negative value of the ratio of speed to the update time interval. The vehicle is stopped if it was moving and the location of this vehicle is computed for the end of the current scanning time period using the acceleration routine.

This car following logic would take care of vehicle advancement when the acceleration or deceleration of the vehicle is determined from the acceleration routine. The following section describes how the acceleration or deceleration rates are determined in the acceleration routine.
3.4.2 ACCELERATION ROUTINE

The acceleration routine is considered as the heart of this simulation model. It determines the proper acceleration or deceleration a vehicle should have while satisfying all safety and operational constraints. Several acceleration or deceleration rates are computed every time interval and the most suitable one is selected.

For the following situations the acceleration or deceleration is computed for every vehicle in every update time.

Situation 1 -

The following vehicle is moving and its acceleration depends on its present speed; A1.

Situation 2 -

The following vehicle will reach its desired speed or the speed limit at the end of current time interval; A2.

Situation 3 -

The following car was stopped and has to start from a stopped position; A3.

Situation 4 -

The following vehicle's performance is determined from the car following algorithm: A4.

Situation 5 -
The acceleration or deceleration rate is computed from the car following algorithm when the non-collision constraint is satisfied; A5.

In addition to the computed values, the maximum allowable deceleration rates of the following and the lead vehicles are taken into consideration in order to keep the computed values within a reasonable boundary. An in depth discussion of the logic and procedure for computation of acceleration or deceleration for the situations is given in the following sections.

3.4.2.1 SITUATION 1-COMPUTATION OF A1

Al is the acceleration of a moving vehicle or a vehicle ready to move which depends on the current speed of the vehicles, and is constrained only with the mechanical ability of the vehicle. Al is taken from Table 1 for a given vehicle type and speed. This table is constructed based on the maximum acceleration rate information taken from table 6.44 and table 6.45 of the Transportation and Traffic Engineering Handbook (T&TEH), 1982 edition.

During the model validation it was observed that the T&TEH table values are too high for normal traffic operation. The simulated vehicles moved faster than their actual
counterparts when T&TEH table values were used, as it was expected. The trajectory comparison of the actual and simulated traffic were used to find a weighting factor for the table values. It was found that 75% of the table values should be used. This decision is supported by the fact that the table values are the maximum attainable rates and an average vehicle-driver unit would employ a fraction of these maximum values.

Thus, the values given on Table 1 are for typical passenger cars and tractor semi-trailer trucks with normal weights of 4000 lb. and 45000 lb, respectively. These values are from standing start to 15 mph and 30 mph, and running speeds of 30, 40, 50, and 60 mph on a level road. For a road which has a gradient, the acceleration rate can be computed by multiplying the table values by the factors used in INTRAS, or by a factor obtained from the information given in the T&TEH.

The normal acceleration and deceleration rates for passenger cars from standing stop to 15 mph and for 10 mph increases in speed at running speeds of 20, 30, 40, 50, and 60 mph are taken from table 6-47 of the T&TEH 1982 edition. These values were observed where the drivers were not influenced to react rapidly. For trucks this information
Table 1
Table of acceleration rates

Typical acceleration rates (ft/sec/sec) from standing start to 15 mph and 30 mph, and at various running speeds thereafter, on a level road.

<table>
<thead>
<tr>
<th>vehicle type</th>
<th>speed mph</th>
<th>0-15</th>
<th>15-30</th>
<th>30-40</th>
<th>40-50</th>
<th>50-60</th>
<th>&gt;60</th>
</tr>
</thead>
<tbody>
<tr>
<td>passenger car</td>
<td></td>
<td>8.80</td>
<td>5.50</td>
<td>5.17</td>
<td>4.17</td>
<td>3.08</td>
<td>2.09</td>
</tr>
<tr>
<td>Tractor Semi-trailer</td>
<td></td>
<td>2.20</td>
<td>1.10</td>
<td>0.88</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
</tr>
</tbody>
</table>

was not given. It is suggested that 75% of the values on Table 1 for trucks may be used.

Table 2
normal acceleration / deceleration rates

Normal Acceleration and deceleration rates for passenger cars (ft/sec/sec).

<table>
<thead>
<tr>
<th>speed change mph</th>
<th>acceleration ft/sec/sec</th>
<th>deceleration ft/sec/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>4.84</td>
<td>7.77</td>
</tr>
<tr>
<td>15-30</td>
<td>4.84</td>
<td>6.74</td>
</tr>
<tr>
<td>30-40</td>
<td>4.84</td>
<td>4.84</td>
</tr>
<tr>
<td>40-50</td>
<td>3.81</td>
<td>4.84</td>
</tr>
<tr>
<td>50-60</td>
<td>2.93</td>
<td>4.84</td>
</tr>
<tr>
<td>60-70</td>
<td>1.91</td>
<td>4.84</td>
</tr>
</tbody>
</table>
3.4.2.2 SITUATION 2-COMPUTATION OF A2

A2 is the acceleration or deceleration of a vehicle to reach the desired speed or the speed limit. Every vehicle has a desired speed assigned to it upon generation of the driver-vehicle unit. The desired speeds are generated from a truncated normal distribution with mean of 55 mph and a standard deviation of 5 mph. The drivers will try to reach their desired speeds as fast as possible while satisfying all of the safety and operational constraints. Once the desired speed is reached, the driver will maintain this velocity or will decelerate if the situation demands it.

The desired speed will be equal to the speed limit when a driver does not want to drive above the speed limit. In CARSIM a courtesy factor which shows what percentages of the drivers will obey the speed limit or the suggested speed is used. Zero means no one will follow the suggested speed or the speed limit, and one indicates that all drivers will travel according to the speed limit or the desired speed.

A2 will be positive if a courtious driver is traveling below the speed limit or the desired speed, and negative when above the suggested speed. It is zero when the desired speed is reached. The desired speed of a courtious driver will change according to the suggested speeds posted along the roadway.
3.4.2.3 SITUATION 3-COMPUTATION OF A3

A3 is the acceleration a stopped vehicle employs when it starts moving again. When a platoon of vehicles is subjected to a kinematic disturbance, the speed of vehicles will decrease gradually and finally they may come to a complete stop. Most of the models cannot handle this situation. However, in CARSIM the stopped vehicles are moved in a realistic manner.

The following vehicle will continue to decelerate while maintaining a safe distance from the leader and comes to a complete stop after the lead car stops. The lead car will move again at the time the user or the traffic condition allows it to move. After the lead car moves the follower will get ready to move after a few seconds of "start-up delay". How long should the following car wait after the lead car moves?

Investigation of The Ohio State data revealed that after the lead car moves the following car will wait 1-3 sec. and then will start to move. This delay in starting is incorporated in the logic of CARSIM. The following vehicle will not initiate a move as long as the lead vehicle has not moved. Once the lead vehicle starts to move the following vehicle waits about 2 seconds and then moves. It is not unreasonable to assume that the drivers with shorter
reaction times will wait less than 2 seconds, while the
drivers with longer reaction times will move in 2 seconds.
A 2 second start up delay for all drivers yielded satisfac-
tory results.

Less than 20% of the drivers have a reaction time of
0.68 seconds in a surprise situation. These drivers will
wait 1 seconds, but the rest of the drivers will wait a
maximum of 2 seconds before moving again. A vehicle will
not be allowed to move, regardless of how long it has been
stopped, as long as the non-collision constraint is not
satisfied. For a vehicle starting from a stand still posi-
tion the acceleration rate is 2 ft/sec/sec for passenger
cars and 1 ft/sec/sec for trucks for the first second of
the movement. Thereafter, the acceleration will be deter-
mined according to the car following algorithm.

3.4.2.4 SITUATION 4-COMPUTATION OF A4

A4 is the acceleration or deceleration computed from the
car following algorithm with no non-collision constraint.
It is computed from the following equation:

\[ X_L - (X_F + V_F \times DT + 0.5 \times A4 \times (DT)^2) \geq L_L + 10 \]  

This simply states that the spacing between the lead and
the following car at the end of the update period should be
greater than or equal to the sum of the length of the lead
car and a buffer space of 10 feet. The 10 foot buffer
space is used in INTRAS, too.
The study of trajectory of different platoons of vehicles indicated that the use of a 10 foot buffer space cannot be justified in all conditions. When density of traffic is very high and speed is very low the vehicles do not keep a buffer space of 10 feet. In jam density the quantity of lead vehicle length($L_L$) plus 10 ($L_L+10$) is much less than 30 feet. For platoon 123 consisting of 15 cars, the minimum spacing which occurred near jam density is in the range of 18.41 to 30.99 feet. The mean of the minimum following distance of these 15 drivers was 22.44 feet. If $L_L$ is 20 feet, average minimum buffer space would be 2.44 feet near jam density. Depending on the density of traffic the minimum spacing would be different for different platoons. Minimum spacing occurs mostly when the platoon is forced to stop due to a kinematic disturbance.

3.4.2.5 SITUATION 5-COMPUTATION OF A5

A5 is acceleration or deceleration of a vehicle when at least the non-collision constraint is satisfied. Consider the worst situation of car following a driver might find himself while driving on a road. It would be when the leading vehicle is decelerating at the maximum rate and the following vehicle after detection of this, reacts to the situation by decelerating to keep a safe distance from the lead car. The following vehicle should be advanced to a
position that provides enough distance to react and to decelerate without running into the lead car. The following equation will be used to assure that enough spacing is provided.

\[ X_L - (X_F + V_F \cdot DT + 0.5 \cdot A5 \cdot (DT)^2) - L_L - K = \text{Maximum of} \]

1. \( (V_F + A5 \cdot DT) \cdot \text{BRT} \)
2. \( (V_F + A5 \cdot DT) \cdot \text{BRT} + (V_F + A5 \cdot DT)^2 / (2 \cdot MX.F) - (V_L)^2 / (2 \cdot MX.L) \)

Where:

* A1, A2, A3, A4, and A5 are acceleration or deceleration rates for a given situation in that time interval;
* AC is the comfortable deceleration rate;
* AXL is the proper acceleration or deceleration rate for that time interval;
* BRT is the break reaction time of a driver;
* DS.VF is the desired speed of the following vehicle;
* DT is the scanning time interval;
* V_F is the velocity of the following vehicle;
* V_L is the velocity of the lead vehicle;
* X_F is the position of the following vehicle;
* X_L is the position of the lead vehicle;
* K or BUF is the buffer space between vehicles, mostly 10 feet;
* MX.F is the maximum deceleration rate of the following vehicle;
MX.L is the maximum deceleration rate of the lead vehicle.

It can be seen that the non-collision constraint built in the logic of CARSIM is:

\[(V_p+A5*DT)*BRT+(V_p+A5*DT)^2/(2*MX.F) -(V_L)^2/(2*MX.L)\geq 0\]  
(3)

The value of A5 is determined such that after moving the follower to its new position there will be enough space for the follower to react to a decelerating lead vehicle, and stop the vehicle without having a collision, or to reach a safe driving speed.

A5 is computed from the following equation. Assume the non-collision constraint is satisfied, solve equation 2 for A5.

\[X_L-(X_F+V_F*DT+0.5*A5*(DT)^2)-L_L-K\geq\]
\[(V_F+A5*DT)*BRT+(V_F+A5*DT)^2/(2*MX.F)-(V_L)^2/(2*MX.L)\]

After multiplying through and simplifying the following equation is obtained

\[(DT)^2*(A5)^2+B*A5+C\leq 0\]  
(4)

Where

\[B=2*V_F*DT+2*MX.F*DT*BRT+MX.F*(DT)^2\]  
and
\[C=-2*MX.F*(X_L-X_F-V_F*DT-L_L-K-V_F*BRT)\]

\[+(V_L)^2/(2*MX.L)+(V_F)^2\]
Discriminant of the last equation must be positive or zero in order to have real solutions to the equation. If the discriminant is zero or positive, there will be one double root or two real roots representing the boundary values of the acceptable region. Always the upper bound value should be used. $A_5$ for this stage is computed from the following form:

$$A_5 = \frac{-B + (B^2 - 4*(D_T)^2*C)^{1/2}}{2*(D_T)^2}$$ (5)

The value may be for acceleration or deceleration depending on the values of $B$ and $C$.

Before using this $A_5$ one should check to see whether or not the left hand side of the equation is greater than the maximum of the two expressions on the right hand side. The equation is set up so that the non-collision constraint is satisfied for all $A_5$ computations. $A_5$ is computed using the non-collision constraint, then the expressions on the right hand side are evaluated using this $A_5$. The largest of the two expressions is used for computation of the final $A_5$ value. In case of having a negative discriminant, $A_5$ should be computed using the first expression on the right hand side.

When the speed of a vehicle is less than 5 mph the buffer speed of 10 feet is no longer an appropriate value. Study of trajectories of vehicles in a bottleneck with very
low speed and high density showed that the spacing is a function of density. At a very high density and very low speed the buffer space of 2-6 feet resulted in a closer-to-the-actual trajectory plots. Otherwise, a 10 feet buffer space is used. Note that buffer space is in addition to the spacing provided for the vehicles to travel during the reaction time.

In addition to A1, A2, A3, A4, and A5, a comfortable deceleration rate is determined for each speed group. The rates are equal to the normal deceleration rates given in table 2. The comfortable deceleration rates will be used when a driver wants to slow down to reach the advisory (posted) speed limit. This would prevent a sudden decrease of speed which might cause another kinematic disturbance.

To choose the proper acceleration rate the program finds the minimum of A1, A2, A3, A4 and A5 and uses this as the acceleration for a particular vehicle at a given time interval. The proper deceleration rate is either the comfortable deceleration, or A2, or A5; and always is less than the maximum deceleration rate of 16 ft/sec/sec (0.5g).

After determining the proper acceleration or deceleration, the speed and the location of a vehicle is computed from the following equation;

\[ v_F = v_F + A * x * L * DT \]  

(6)
and \[ X_F = X_F + V_F \cdot DT + 0.5 \cdot AXL \cdot (DT)^2 \] (7)

This vehicle is advanced to a new position and the rest of the vehicles are moved in a similar fashion.

### 3.4.3 OTHER CONSIDERATIONS

The maximum deceleration rate is determined from the braking force as long as slippage does not occur between the pavement and the tire of the vehicle. When slippage occurs it is determined by the effective coefficient of friction at the tire surface contact. Coefficient of friction is a function of pavement type, tire condition, and whether it is dry or wet. A good braking system usually provides more braking force than can be carried on to the pavement. Therefore, maximum deceleration depends on the coefficient of friction.

Neglecting the retarding forces caused by air, engine and rolling resistance, the force required to stop a vehicle is dependent on the weight of the vehicle and the ratio at which it is decelerated. The equation is:

\[ F = M \cdot a = W / 32.2 \cdot a \]

\( F \) is the braking force in pounds and \( W \) is the weight in pounds. The limited value of force \( F \) depends on the grip of the wheel with the road surface. The ratio of this grip to the weight of this vehicle is the coefficient of friction.

\[ f = F / W = W \cdot a / (32.2 \cdot W) = a / 32.2 \]
It can be seen that maximum deceleration is directly proportional to $f$.

The maximum deceleration rate is very important in the computation of the minimum stopping distance. The car following model must provide a spacing headway greater than the minimum stopping distance. In this model the vehicle will not be advanced locationwise unless there is enough space for the following vehicle to decelerate during the reaction time in addition to the minimum stopping distance.

The minimum stopping distance or distance to slow down is

$$s = \frac{v_1^2 - v_2^2}{2a} = \frac{(v_1^2 - v_2^2) \times (1.466)^2}{2(32.2(f+/-g))}$$

or simply

$$s = \frac{(v_1^2 - v_2^2)}{(30(f+/-g))}$$  \hspace{1cm} (8)

Where $S$ is in feet, and $V$ is in mph.

The maximum deceleration rates for the extreme emergency condition where the drivers must bring the car to a stop from a 70 mph speed as quickly as possible without sliding is very close to 20 ft/sec/sec. For 70 mph and 60 mph it was very close to 20 and for 50 mph a little over 20 ft/sec/sec for passenger cars on a test conducted by Wilson (1940) on a dry concrete pavement. He suggested 19.5 ft/sec/sec for a dry road surface.
The observed normal deceleration rates are given in Table 2. These are for dry pavement and for stops from running speeds of 15 and 30 mph and for 10 mph slowdowns from 40, 50, 60, and 70 mph running speeds. Deceleration rates of 8.5-9 ft/sec/sec are reasonably comfortable for passengers (Wilson 1940).

The average deceleration rate comfortable for passengers should not exceed 8.5-9.00 ft/sec/sec. A deceleration rate of 11.05 ft/sec/sec is undesirable, but not alarming to the passengers. However, a rate of 13.90 ft/sec/sec is excessive and uncomfortable to the passengers. In this rate the objects will slide forward off the seat (Wilson, 1940). A deceleration rate of 12-13 feet/sec/sec is considered ordinary emergency deceleration (R.A. Moyer in the discussion of Wilson's paper).

Information processing time and legibility (detection) distance requirement for a traffic control sign is very important. The information processing time is related to message length, and the legibility depends mainly on the letter size, among other factors.

Forbes and Holmes (1939) conducted a study about legibility and its relation to letter size. A total of 4623 observations were obtained from 412 different persons. For a given letter height the legibility distance varied from
an observed lower value to twice that amount. So the range was approximately equal to the lower value. For 80% of the drivers the legibility distance in feet is approximately equal to 50 times the letter size in inches. They suggested to use the 80 percentile values to determine the practical sign legibility.

A rough practical norm for day conditions and normal vision is 50 feet per inch of letter height for a wide; and 33 feet per inch for a narrow letter. A curvilinear relationship was noticed (Forbes & Holmes, 1939) between legibility distance and letter size between 6 in. and 10 in.. For both widths the legibility distance was reduced approximately at 15% for night conditions. A study by Rockwell et al (1973) developed a formula for the maximum legibility distance. The values from this study are higher than the Forbes et al values. Moore and Christie’s (1960) analysis of data showed that 50 feet per inch of letter height would cover the requirements of about 98% of drivers on the road.

The time to read the sign depends on how complex the sign is. Mitchell and Forbes (1942) suggested:

\[ t = \frac{N}{3} \]

where \( N \) is the number of simple names or words on the sign. The use of

\[ t = \frac{2N}{3} \]
It is assumed that the letters on the speed limit sign will be at least 10 inches high; so the sign will be legible at a distance of 500 feet. Drivers who comply with the advisory speed limit signs would start doing so after they see the sign. Therefore, the speed limit for a section starts 500 feet before the sign and ends at 500 feet before the next speed limit sign. The subprogram SPEED LIMIT is called to give the speed limit as a function of the location of the vehicle.

3.5 CAPABILITIES

CARSIM is a very versatile, and highly capable car following model and can be used in various microscopic traffic flow simulation models. The capabilities are:

CARSIM can be incorporated in any microscopic traffic flow simulation model for any kind of roadway. It can be used for simulation of traffic flow on a single lane or multilane highway with no lane change. (A lane changing routine can easily be added.)

Different types of vehicles can be used in CARSIM. In its present form the information about acceleration and
deceleration of passenger cars and tractor semi-trailer trucks are used in the model. One can add the acceleration and deceleration characteristics of any kind of vehicle to the input values with very little effort.

CARSIM may be employed for simulation of the normal flow as well as jam density traffic conditions. The model may be used as a single or dual regime model depending on the choice of the user. It realistically reproduces the effects of severe traffic disturbances.

3.6 FEATURES

The unique features of this model are:

1. It provides a safe following distance for all vehicles at all times; no collisions are allowed. A vehicle is advanced to a position that leaves it enough space to decelerate to a safe speed or a complete stop if its leader car initiates a deceleration. In CARSIM, the delay in reaction of the following car to such a move by the lead car is taken into consideration. A maximum allowable deceleration rate of 16 ft/sec/sec is used in these computations in order to prevent the use of unrealistic deceleration rates.
2) Another unique feature of CARSIM is its ability to handle a situation where the vehicles are forced to decelerate to a complete stop; and after waiting for a while they start to move again and accelerate to reach their desired speed. This process of going through a kinematic disturbance that forces the vehicles to be in a stop-and-go operational mode has been taken into consideration in this model. Representing the stop-and-go situations realistically is a challenge for any car following algorithm. How close the trajectories from the simulation model is to the actual trajectories indicate how realistic the algorithm is.

This is more challenging when vehicles are forced to stop for several seconds and then move in proper order. While the following vehicles are in the process of slowing down to a complete stop, the leading vehicles will start accelerating one after another with a few seconds delay. This delay is similar to a start up delay at an intersection.

To gain more insight in how vehicles follow each other near jam density and what is the pattern of acceleration once they are stopped, the Ohio State trajectory data were investigated. It was found that when the lead vehicle is stopped, the immediately following vehicle with a speed of
about 5 fps will come to a complete stop in one or two seconds. Also it was found that once the lead car starts moving the following vehicle will wait 1 to 3 seconds before initiating any movement.

Furthermore, the spacing between vehicles near jam density (very low speed) is much less than the sum of the lead vehicles length (20 feet) plus a buffer space of 10 feet, which is used in PITT algorithm (car following of INTRAS, 1980) or INTRAS based car following models (Rouphail, 1981; Rathi, 1983). In CARSIM a vehicle length of 16 feet and a buffer space of 3 to 7 feet is used at a very high density situation. When the speed of the following car falls below about 5 mph and the density of traffic is near jam density the buffer space of 3 to 7 feet will be used. Otherwise a buffer space of 10 feet is used.

How long does the following vehicle wait after the lead vehicle starts to accelerate? The examination of O.S.U. data revealed an average value of 2 seconds for the start-up delay. A more reasonable approach is considering the characteristics of the drivers. For a driver with a lower reaction time the start-up delay seems to be less than that of a driver with a higher reaction time. The following values were also used and reasonable results were obtained. If a driver's reaction time for a surprise situation is
less than 1 second the start-up delay for the driver is 1 second. Otherwise, it is a maximum of 2 sec. For simplicity one may use a start-up delay of two sec. for all drivers. I should mention that the 2 seconds start-up delay is not the total time a vehicle is stopped on the freeway, but it is only the delay after the lead car has moved.

3) Another exclusive feature of CARSIM is that the dual behavior of traffic is reflected in the car following algorithm. The density of traffic is used as the criteria to determine congested and non-congested flow. When the density is 55 or 60 vpm the flow is considered congested and below this density it is non-congested.

For non-congested flow the following and lead vehicles both have the same maximum deceleration rates. However, when flow is congested the following driver is more cautious about the available spacing. On a time-space plot a change of slope is observed for trajectories of vehicles when density is about 60 vpm.

To build a feature in the model that can reflect the difference between congested and non-congested flow operation, several parameters were examined. Finally, it was decided that this difference can be shown by using different maximum deceleration values for the following and the lead car. Maximum deceleration of 13 ft/sec/sec is used
for the following vehicle when density exceeds 60 vpm, while maximum deceleration for the lead car is 16 ft/sec/sec.

The use of different maximum deceleration rates are merely for computational purpose. In the real-world situations different maximum deceleration for the same vehicle, when its status changes from leading to following, cannot be justified. However, the effect of this in computation of deceleration can be justified by examining the nature of the effect. It causes the following driver to increase his spacing and become more aware of the congested operation on the road.

Examination of the trajectory data revealed that the vehicles maintained longer spacing after suffering from a kinematic disturbance when is is compared to the condition before going through the disturbance.

4) The fourth unique feature of this model is using varying reaction times for an individual driver. In a dense traffic the reaction time of a driver is different from the normal traffic condition. Similarly, the reaction time of a driver is not the same for surprise and alerted situations. The drivers are more alert in congested traffic than in free flow traffic conditions. It is reasonable to assume that, in high density traffic the drivers reaction
to a deceleration is shorter than that of a normal traffic condition. For a normal traffic condition, the reaction time of a driver is assumed to be equal to the values in non-alerted (surprise) situations. However, for congested flow the reaction time is assumed to be the same as that for alerted conditions.
CHAPTER IV
VERIFICATION AND VALIDATION OF THE MODEL

4.1 INTRODUCTION

As a partial verification effort, acceleration/deceleration patterns, velocity change patterns, trajectories of vehicles, and spacing between vehicles are examined. Various artificial disturbances were induced to a platoon of 15 vehicles and their effect at microscopic and macroscopic levels are investigated.

At the microscopic level, the effects of a regular disturbance, an emergency stop, and a stop-and-go operation on individual vehicles trajectories, speed, and acceleration/deceleration, are analyzed at high and low volume levels. At the macroscopic level, the effects of the regular disturbance and an emergency deceleration on average speed, density, volume, and average headway are examined at high and low volume levels, (see appendix D).

Extensive verification effort was undertaken to ensure that the simulation model is properly designed. Several input parameters are systematically varied and the sensi-
tivity of the system outputs are carefully examined. The parameters are: maximum deceleration rate, compliance level, start-up delay, reaction time, buffer space, speed for buffer space change, and traffic mix.

The validation of this model is performed at microscopic and macroscopic level. For the microscopic validation the trajectories and speed change patterns of four different platoons of vehicles covering a wide range of density (free flow to jam density) are compared to the simulation results. The speed change patterns obtained from the simulation models replicate the real world situations very realistically. The trajectory plots generated by the simulation models for before a severe kinematic disturbance and after that are very close to the plots from the field data. The simulation plots reproduce situations where vehicles are forced to stop for a while and then start moving again.

For macroscopic validation the overall performance of the simulation models are compared with that of three different platoons. The variation of flow parameters, the fundamental relations of traffic flow, the relationships between simulation results and field data, and the effect of stochastic input values on response variables are investigated.
The details about verification and validation of this model will be discussed in this chapter later on. In the following sections the model validation procedures, the concerns in an output analysis, and the data base used for validation of this model will be discussed first.

4.2 MODEL VALIDATION PROCEDURES

Verification is to check if the model behaves as the experimenter assumes it does, and validation is to test whether the simulation model reasonably approximates a real system (Fishman and Kiviat 1968 statistics of discrete event simulation). After these two steps ensured that the simulation model is properly designed, one can use the model to study the system behavior.

The evaluation of performance of each component of a simulation model and the model as a whole are essential parts in model validation. It is important to know the range of the values of the parameters and the conditions a model should not be applied. A good simulation model should reproduce the real world situations under different conditions. When a data set is used for calibration of a model another data set should be used for evaluation of the performance of the model. Using the same data for calibration and later for evaluation is not an adequate model val-
idation procedure. Some suggestions for validation and verification are as follows.

Taylor (1979) used the four level evaluation procedure given by Pilgrim (1975). These levels are: a) rational examination of model structure and components; b) an estimate of model parameters; c) a verification of the accuracy of the fitted model by using another data set and examining the results; d) The prediction of the possible range of applicability of the model. This procedure was used in a traffic assignment example.

Gafarian and Walsh (1970) discussed a method for statistical validation of a simulation model for freeway traffic near an on-ramp. They used travel time and velocity of vehicles as the measure of effectiveness of simulation model, and compared the simulation results with observed values using Wilcoxon signed-rank test. The procedure for using Chi Squared Test was also discussed.

Fishman (1967) discussed a procedure which exploits the autocorrelations when observations are not independent. To compare the means of autocorrelated observations of two simulation experiments, he suggested that the difference of the sample means be treated as a normal variate for a sufficiently long sample record. Although this procedure is not as good as comparing autocorrelation structure of the
two experiments, but it suffices as an initial step for comparison. A procedure to divide a long enough autocorrelated run to a number of equivalent independent observations was also given.

Mihram (1972) discussed the procedure for comparison of the results from several independent replications of a simulation model with that of one or more observed data. Assume that only one observation is available on the system being mimed by a simulation model and there are N independent simulation model results. These N+1 responses constitute a random sample of size N+1 from the same cumulative distribution function. If these data are rearranged in numerical order, the single observation should not fall in either extreme. Under the assumption that all N+1 orderings of the sample into the two groups are equally likely, the probability of the single observations being at one extreme or the other becomes $2/(N+1)$.

If there is more than one observation available on the system, say N observations, one can apply a number of nonparametric tests. For example, the run test or Kolmogrov-Smirnov test may be used.

If one has the results of two independent runs say $Y_1$ and $Y_2$, then one can test as whether $D=Y_1-Y_2$ is significantly different than zero or a given number. If $Y_1$ and $Y_2$
are coming from a normal distribution with mean \( \mu \) and variance of \( \sigma^2 \), then one may use Chi Squared Test with 1 degree of freedom and compare the table value with \( \frac{(Y_1-Y_2)^2}{\sigma^2} \). Here it is assumed that \( \sigma^2 \) is known.

When the variance is not known one has to have replication of responses. Replication of simulation is obtained by using random and independent seeds. Then the remaining samples are computed and the pooled variance of \( \frac{S_1^2+S_2^2}{2} \) is used to run a t-test.

When a random sample of \( N \) system responses is available one can form frequency histogram on cumulative distribution function and compare this with that of \( N \) simulation results by means of Chi Squared or Kolmogrov-Smirnov Test.

However, if we have the response of the system recorded previously, we can find the mean and the variance and compare these with the mean and the variance of several replications of a simulation. Assuming that the results are normally distributed, a t-test is used for comparing the means and an F-test for comparing the variances.

Torres, Halati, Gafarian (1983) developed a statistical guideline for traffic simulation models. Statistical tests developed were based on differences between the real world
observations and simulation results. Three statistical tests for validation of NETSIM were: 1) standard pared t-test; 2) Hotelling T Squared Test; and 3) modified ANOVA test.

Naylor, Wertz and Wonnacott (1967) gave three alternative forms of analysis of variance for analysis of output from computer simulation experiments and making a decision about their differences and ranking. The techniques are F-test, multiple comparison method and multiple ranking.

Annino and Russell (1979) discussed 10 most frequent causes of simulation failure and suggested how to avoid them. Using an unverified model and lack of understanding of the system are among the causes.

Kleijnen (1976 sci simulation) gave techniques to compare means and variances of two simulations where the outcomes from replications are correlated pairwise. To compare means the t-test is suggested. Distribution free tests such as the Sign test or Wilcoxon's rank test for paired observations, and Mann-Whitney test for comparing the means is suggested. However, in simulation the normality requirement is not as important as independence. Kleijnen (book II p.4553-478) discussed that even average of dependent observations may be approximately normally distributed; the normality assumption is not very crucial,
more important is the assumption of independence. Thus he suggested that the t-test should be used to compare the means because, it is quite insensitive to nonnormality.

To compare variances the following three procedures were suggested: Continue using the traditional F-test but be aware of its conservative character. Second, split each of the observations into 2 groups and do an F-test for the first half observation from system 1 with the second half of observations from system 2. Then do another F-test for the other halves. This approach is conservative too. The third approach suggested is Wilk's test. This is more complicated than the two approaches above but it is exact.

Once again, verification is to be sure that the model will behave as intended, and validation is to see there is an adequate agreement between the model and the system being modeled. A Model should be tested under different experimental frames in order to obtain a high model-confidence. Sargent (1982) suggested model validation should consist of conceptual validation, computerized validation, operational validation and use of adequate and correct data.

The conceptual model validation is performed to ensure that the theories used, the assumptions made in using a theory, and mathematical, logical, and any kind of rela-
tionship used are correct and proper for each submodel and for the overall model. The two techniques that can be used for investigation of each submodel and the overall model are traces and face validation. In traces the behavior of different types of entities are traced through each submodel and overall model to determine if the model's logic is correct and the necessary accuracy is obtained. Face validity is asking experts in the subjects to evaluate if the logic of the submodel and the model is correct and the input-output relationships are reasonable.

The computerized model verification is to check the development of the computer model, to bug it, to run it correctly, and to ensure that the conceptual model is implemented. Each submodel is tested to see if it works properly and the overall model is executed under different conditions to investigate input and output relations. Operational validity is ensuring that the simulation model is a reasonable and accurate representation of the real system with certain levels of confidence. Here the validation can be done subjectively such as graphical representation and examination of it; or can be done objectively such as using statistical techniques. In testing hypothesis the attempt should be on reducing the type II error rather than the type I error.
Kleijnen (1982) discussed how regression analysis is used to obtain a metamodel (a model explaining the simulation model) and how the effect of qualitative or quantitative factors can be investigated by using weighted least squares technique. Metamodel measures sensitivity of response to various input factors.

Kleijnen (1977) discussed that replication of runs is the only alternative for gathering statistics about terminating systems. In a terminating system the simulation run ends if a specific event occurs. Different seeds should be used to get independent replications. Initialization for replicated runs is performed by throwing away the observations at the beginning of each run. The rule-of-thumb is that you might discard the observations as long as the response variable continues to increase or decrease. From several replications one can get the response for each run and then find the mean and the variance of the responses. Then use t-statistics to build a confidence interval for the response. The t-statistic can often be used since it is not very sensitive to deviation from normal distribution. The design and analysis procedures for comparison of many systems and variance reduction techniques were also discussed.
Kleijnen (1975) discussed techniques for validation of simulation models using Chi Squared, factor analysis, spectral analysis, and simple regression analysis between the actual and the simulation outputs by testing if y-intercept is zero and the slope is one. In another paper (1975) he discussed how to use antithetic variate and common random numbers and their possible undesirable negative correlation when used jointly. He compared the three methods for different conditions and concluded that one cannot say which method is best for all systems.

4.3 ANALYSIS OF SIMULATION OUTPUT

A stochastic variable in simulation model would cause the system variable and the results of simulation to fluctuate as the simulation proceeds. Several independent simulation results should be obtained in order to be able to make statistical information about the true value of the response variable. For independent results one can run a t-test and find the confidence level for the mean of observations. However, in simulation the results do not always meet the requirements of a traditional t-test or f-test.

Let us consider a dense traffic situation and the time it takes for a leading vehicle to travel a certain section of a freeway. The time to travel the same section of the
road for following vehicles will depend on the travel time of the lead car. This shows that the travel times of successive vehicles in a dense traffic are autocorrelated. Normally in simulation the sample mean of autocorrelated data can be approximated by a normal distribution when sample size is large (p.303 Gordon's book). Thus the mean of the travel times of all drivers on that section will be simply the average of the values, but the variance of travel time will not be simply \((\text{Sigma Squared})/n\). The variance should be computed from another equation which takes into account the autocorrelation, (see Fishman's, Kleijnen, or Shannon's book).

Now let us use this procedure for finding the mean and variance of average travel time of a platoon of vehicles on a section of a freeway. From a single simulation run one can get only one value for the travel time. The simulation run must be replicated and several values for average travel time ought to be obtained. Each average value is independent and can be assumed to be asymptotically normal (Kleijnen II p.457). Therefore, the mean of the average travel times and the variance of the average travel times can be computed using a simple mean and variance formula. One can use a t-distribution to find an interval estimate of \(\mu\). The distribution of \( ((\text{mean}-\mu)/(\text{sqrt of (variance/n)))} \) is approximately a t-distribution with \(n-1\) degrees of
freedom (n is number of replications). This approximation is improved as the number of observations in each replication is increased. However, increasing the number of replications may not have the same effect, see example of M/M/1 queue problem (Fishman page 220). If it could be assumed that the travel time of each driver is independent of the leading car one could find a variance for each run and use the mean value for these variances as the variance of several replications with a degree of freedom one less than the number of replication (Gorden book page 305).

The results of simulation studies should be collected after the system reaches a steady state condition. The data before steady state is eliminated in order to minimize the bias on the mean value of the response variable. There is not a definite rule on how the interval bias should be eliminated and how much of the simulation run should be ignored for this purpose. One way of finding out how much of the data should be truncated is by plotting the response variable versus time, and locating the beginning of the region of steady state situation.

To study an autocorrelated data time series approach is more suitable. Usually for time series study a long run of simulation is needed in order to be able to find autocovariance between values of the variable separated some units
from each other. Spectrum analysis is another way of studying autocorrelated data.

The sample size used depends on the level of precision desired. In simulation normally the response variable is the sum of a number of different contributing variables which may not be independent. One can invoke central limit theorem and assume no autocorrelation and use the following formula to find the sample size \( n = \frac{(\sigma_0^2 + \sigma_1^2)Z^2}{d^2} \), (Shannon's book page 188) if \( \sigma_0 \) is unknown one can run a pilot study and find an estimate of the variance and use \( t \) instead of \( Z \). Here \( d \) is the difference acceptable between the estimate and the true parameter or half width of the desired confidence interval. Note that approximately 95% of observations are within two \( \sigma_0 \) on either side of the mean.

When two operating conditions are to be compared, one can introduce a negative correlation between replications of runs under one operating condition to reduce the variance for within runs; and then introduce a positive correlation between runs under different operating conditions to reduce the variance for the difference between runs. This procedure is suggested as an efficient experiment design if there is not an initial transient phase (Emshoff book p. 198) This variance reduction method is using antithetic
variable and common random numbers jointly. Kleijnen (1975) discusses possible undesirable effect of such a combination and indicates that because of cross-correlations the results may even be worse than using each method alone.

Start up policies in simulation and reducing the effect of initial transient state was surveyed by Wilson and Pritsker (Aug. 1978; Sep. 1978), and the procedure for evaluation of different start up policies is given. Deleting data from the beginning of the simulation output to reduce initial transient effect causes loss of information and an increase in the variance. They found that a bias reduction is achieved in expense of variance increase. The net effect of deletion of initial observation was to increase the mean square error of the sample mean. Starting from empty and idle situation (no truncation of initial condition) gave the lowest value for estimation of mean square error.

Considering bias, variance reduction, and mean square error they suggested starting as close to the steady state mode as possible and keeping all data. Furthermore, they stated that

the judicious selection of an initial condition appears to be more effective than truncation in improving the performance of the sample mean as an estimator of the steady-state mean
The research has indicated that for small and well behaved models truncation should not be performed, but for large scale models this may not be the case (Pritsker, Pegden p 491-495).

4.4 DATA BASE

The data used for validation are The Ohio State trajectory data collected using aerial photogrammetric technique by Treiterer (1975). A KA-62A aerial camera was used. Most aerial surveys were taken from an altitude of about 3000 feet and the length of the freeway covered was about 4500 feet. Photographs were taken in 1 second time intervals. The position of vehicles are estimated to be accurate within +/- 0.50 feet and the speed was determined with an error of +/- 1.0 mph. The study site was I-71 in Columbus (2 lanes per direction), 3.5 miles long, maximum grade 2%, maximum curvature 1 degree, 15 minutes. There were 3 on and 3 off ramps. The data used are from the median lane with less than 1% trucks, taken on July 25, 1967 between 7:45 and 7:50 am on a Tuesday with normal traffic. The data provide a complete record of 115 individual vehicles spacings, headways, longitudinal positions, and velocities for about 4 minutes.

The platoons selected for validation of this model are:
1. Platoon 123: A group of 15 vehicles which had no vehicle entering or leaving the platoon. The platoon leader was vehicle 186 and the last car in the platoon was vehicle 175.

2. Platoon 126: Covered a large range of density in a rather short period of time. The platoon started with 15 vehicles, after several seconds 1 vehicle left the lane but a new vehicle entered into the lane. This group of 14 vehicles went through the kinematic disturbance, and lost 1 car before recovering and another vehicle after recovering. The lead car was 475 and the last one was 457.

3. Platoon 127: This platoon reached a very high density. It started with 10 vehicles and remained a 10 vehicles platoon. The lead car was 452 and the last one was 172.

4. Platoon L123X A group of 5 vehicles was selected from platoon 123 and their following behavior is studied for a longer period of time, 202 seconds. The leader was 452 and the last vehicle was 449.

4.5 VERIFICATION AND SENSITIVITY ANALYSIS

It is performed to determine sensitivity of the system response (output) to the values of the parameters used (input). Sensitivity analysis is an important concept in
simulation modeling and usually consists of systematically varying the input values on a range of interest and examining the effect of it on the results of the models. If there is a great change in responses of the system with a little variation of values of input parameter, then examination of the model is necessary. On the other hand, if the change in the results is none or very small, this indicates that there is no justification or need for accurate estimate of the input parameter.

Sensitivity analysis would provide some indication how the system results will be affected when the input parameters changes. Furthermore, it would give us valuable clues for possible modification of the model in the future. Sensitivity analysis together with the verification of the model is performed for the following parameters: maximum deceleration rate, compliance level, start-up delay, reaction time, buffer space, buffer space change speed, and traffic mix.

4.5.1 MAXIMUM DECELERATION RATE

The car following behavior of the drivers before and after going through a kinematic disturbance is not the same. They become more cautious after the disturbance and maintain longer spacing than before the disturbance. Their slower reaction to acceleration than to deceleration con-
tributes to the existence of a lack of uniform behavior. To examine this difference various maximum deceleration rates are used and their effects on the trajectory plots are presented.

First, the same maximum deceleration rates are assigned for the lead and following cars. When the rate of 16 ft/sec/sec is used for all vehicles at all density levels, it was observed that the simulated vehicles travel faster than the actual vehicles at higher densities, as it can be seen on Figure 2. At lower densities the simulation and actual plots are very close, however, at higher densities there are considerable differences. To overcome this problem a lower maximum deceleration rate of 13 ft/sec/sec is used, and its effect on the trajectory plots is examined. Figure 3 shows the problem of a lack of a good fit even at a lower deceleration rate of 13 ft/sec/sec. This problem can be resolved by providing longer spacing at higher densities.

To provide larger spacing at higher densities, different deceleration rates are used for the following and the lead cars. In Figure 4 one can see the effect of using 13 and 16 ft/sec/sec deceleration rates for the following and the lead car, respectively. The fit is greatly improved at higher densities, but not at lower densities.
Figure 2: Deceleration rate of 16 for all vehicles in all densities. Effect of using a max. deceleration rate of 16 ft/sec/sec for all vehicles in all densities. Simulation results (K, 1-15) for every third car are compared to platoon 123. Volume is 1800, and units are feet and seconds.
Figure 3: Deceleration rate of 13 ft/sec/sec for all vehicles in all densities. Effect of using a max. deceleration rate of 13 ft/sec/sec for all vehicles in all densities. Simulation results (K, 1-15) for every third car are compared to platoon 123. Volume is 1800, and units are feet and seconds.
Figure 4: Different rates for lead and following car in all densities. Effect of using a max. deceleration rate of 16 for leader and 13 ft/sec/sec for following car in all densities. Simulation results for every third car(k, 1-15) are compared to platoon 123. Volume is 1800, and units are feet and seconds.
The attempt to use a uniform deceleration rate regardless of the density of traffic was not as successful as using two different deceleration rates. It is not possible to use the same deceleration rate for before and after a disturbance and obtain a trajectory plot that is very close to the actual trajectory plot. When the maximum deceleration rate is the same for the following and the lead vehicles at all density levels, the objective is to find a single regime model that fits both congested and non-congested regions. The single regime models do not represent closely the traffic behavior at all densities. To represent realistically the traffic behavior on a freeway, it was decided to use the idea of dual regime models instead of single regime models.

For the first time, the concept of using maximum deceleration rate as a function of traffic density is introduced in computation of the required spacing in a car following situation. For non-congested traffic the maximum deceleration rates of the following and the lead car are 16 ft/sec/sec. For congested traffic (density above 60 vpm), maximum deceleration rates of 10, 12, 13, and 14 ft/sec/sec are used for the trailing vehicle. The effect of using different rates is illustrated in Figures 5-8. When maximum deceleration of the following car is 10 or 12 ft/sec/sec the simulated vehicles fall behind the actual vehicles, see
Figure 5 and 6. However, at a rate of 14 ft/sec/sec they move faster than actual traffic, Figure 8. Therefore, it was decided to use a maximum deceleration rate of 13 ft/sec/sec for the following vehicle when the density is higher than 60 vpm; and 16 ft/sec/sec for the lead and the following car otherwise; Figure 7.

Using a lower maximum deceleration rate at higher densities is only for computational purpose. In a real world situations a driver may use a higher deceleration rate at higher densities in order to prevent any accidents.
Figure 5: Deceleration rate of 10 at higher densities. Effect of using a max. deceleration rate of 10 for following cars at higher density and 16 ft/sec/sec elsewhere. Simulation results for every third car (k, 1-15) are compared to platoon 123. Volume is 1800, and units are feet and seconds.
Figure 6: Deceleration rate of 12 at higher densities. Effect of using a max. deceleration rate of 12 for following cars at higher density and 16 ft/sec/sec elsewhere. Simulation results for every third car (k, 1-15) are compared to platoon 123. Volume is 1800, and units are feet and seconds.
Figure 7: Deceleration rate of 13 at higher densities. Effect of using a max. deceleration rate of 13 for following cars at higher density and 16 ft/sec/sec elsewhere. Simulation results for every third car (k, 1-15) are compared to platoon 123. Volume is 1800, and units are feet and seconds.
Figure 8: Deceleration rate of 14 at higher densities. Effect of using a max. deceleration rate of 14 for following cars at higher density and 16 ft/sec/sec elsewhere. Simulation results for every third car(k, 1-15) are compared to platoon 123. Volume is 1800, and units are feet and seconds.
4.5.2 COMPLIANCE LEVEL

Not all the drivers would comply with the proposed speed. The effect of the percentage of drivers complying with the proposed speed limit on the average speed and density is examined by this simulation model. Compliance levels of: 0% (no compliance), 10%, 20%, 30%, 40%, 50%, 60%, 70%, 80%, 90%, and 100% are used.

The proposed speed limit for a 1000 foot long section of the freeway is 66.00 fps while the speed limit is 80.66 fps elsewhere. This section is located 4000 feet from the beginning of the study section. A certain percentage of drivers will start slowing down at a rate determined by the car following algorithm when they reach a point of 3500 ft or beyond. The rest of the drivers will not decelerate because of the proposed speed limit, but will slow down to keep a safe spacing and to satisfy the non-collision constraint of CARSIM. Thus, some of the non-complying drivers are compelled to decelerate by the drivers traveling ahead of them at the proposed speed limit.

Average speed and average density of a platoon of 15 cars, with two different reaction times distributions going through the zone is presented in Figures 9-12. When the reaction times of the drivers are equal to the values from the reaction time table, Figures 9 and 10 are obtained.
4.5.3 START-UP DELAY

Effect of using different start up delay values for vehicles forced to halt in a stop-and-go operational situation is examined. A severe kinematic disturbance makes the vehicles to stop for a period of time and a few seconds after the preceding car is moved, they start moving again. The effect of using a start up delay of 0, 1, 2, 3 seconds, and a value which is a function of the reaction time of the driver on the trajectories of the vehicles are shown on Figures 13-17.

When it is assumed that every successor starts accelerating from a halted position 0 or 1 sec. after the predecessor begins moving, unacceptable differences between the actual and the simulation results are observed; as it can be seen on Figures 13 and 14. Most of the vehicles that should have come to a standstill position due to the kinematic disturbance did not stop. The simulated vehicles did not really show the effect of the kinematic disturbance while in the actual platoon every vehicle had stopped because of the disturbance. It becomes obvious that the start up delay must be taken into consideration in the logic of a realistic car following model.

A start up delay of two seconds, or two seconds for most of the drivers and 1 second for the drivers with reaction
Figure 13: Using a start-up delay of 0 sec. Comparison of the simulation results for every third car (K, 1-15) with platoon 123 when a 0.0 sec. start up delay is used. Volume is 1800, units are feet and seconds.
Figure 14: Using a start-up delay of 1 sec. Comparison of the simulation results for every third car (K, 1-15) with platoon 123 when a 1.0 sec. start up delay is used. Volume is 1800, units are feet and seconds.
Figure 15: Using a start-up delay of 2 sec. Comparison of the simulation results for every third car (K, 1-15) with platoon 123 when a 2.0 sec. start up delay is used. Volume is 1800, units are feet and seconds.
Figure 16: Using varying start-up delay. Comparison of the simulation results for every third car (K, 1-15) with platoon 123 when start up delay is a function of reaction time. Volume is 1800, units are feet and seconds.
Figure 17: Using a start-up delay of 3 sec. Comparison of the simulation results for every third car (K, 1-15) with platoon 123 when a 3.0 sec. start up delay is used. Volume is 1800, units are feet and seconds.
time of less than one second, resulted in a very close-to-actual trajectory plots; Figures 15 and 16. All vehicles are stopped and started in a manner very close to the real world traffic movement. The trajectory plots for every third vehicle in actual traffic and simulated traffic show a very close agreement.

Using a larger start up delay and its effect on the trajectory plots are presented in Figure 17. Every vehicle is assumed to wait for 3 seconds after the predecessor moves and then move. The plot shows unacceptable trajectory plots for simulated vehicles.

Thus, using a 2 second start up delay for all vehicles is very well justified. One may use 2 seconds for most of the drivers and 1 second for about one-fifth of them as it is shown in Figures 15 and 16.

The stop-and-go operational condition as well as normal traffic can be realistically reproduced using this simulation model. This unique feature enables one to evaluate accurately the alternative policies for alleviation of freeway congestion.

4.5.4 REACTION TIME

The reaction time of the drivers from the reaction time table are increased by 0.0, 0.1, 0.2, 0.3 and 0.4 seconds
and the effect on the average speed, on the average density, and on the average headway are studied. The lead vehicle decelerates at a rate of 6 ft/sec/sec for 6 sec., at 0.0 ft/sec/sec for 3 sec., and then accelerates at 6 ft/sec/sec for 6 seconds to reach a speed of 80.66.

The effects of this disturbance on the average speed of a platoon of 15 cars are shown in Figure 18. This plot reveals that the effect of a longer reaction time is more pronounced on the acceleration phase than the deceleration phase. Drivers with a longer reaction time have to decelerate at a higher rate in order to keep a safe spacing between his car and the car ahead. On the other hand, in acceleration phase, the drivers have more freedom to choose the time and the rate of acceleration.

The density of the traffic increases during deceleration phase because the lead vehicle slows down and reduces the spacing between the first and the last car in the platoon. The sharp increase in the graphs of Figure 19 reflect this situation. The rate of increase changes before the platoon reaches a constant density. This transition from a higher rate of increase to a constant density represents the acceleration of vehicles to reach the speed of 80.66. It should be reminded that this disturbance did not reach the last car in the platoon.
Figure 18: Effect of increasing reaction time on speed. Effect of increasing the reaction time of the drivers by 0.0, 0.1, 0.2, and 0.3 seconds on average speed when a platoon of 15 cars goes through the regular disturbance. Volume is 1800, and units are ft/sec and seconds.
Figure 19: Effect of increasing reaction time on density. Effect of increasing the reaction time of the drivers by 0.0, 0.1, 0.2, and 0.3 seconds on density when a platoon of 15 cars goes through the regular disturbance. Volume is 1800, and the units are vpm and seconds.
Different constant density levels are reached by the platoons since the drivers reaction times are not the same in different platoons. A platoon assembled with drivers of higher reaction times maintained a higher constant density because of longer spacing between vehicles. However, the difference between two consecutive graphs is increasing as the magnitude of the added value is increasing. This is due to the fact that the reaction time is one of the many factors influencing the spacing between the vehicles.

Reaction times of the drivers affect the average headway of the platoon, as it can be seen on Figures 20 and 21. The average headway of the platoon is computed by two different methods.

The first procedure is simply finding the average headway from the traffic volume. Since speed and density are available for every second one can easily find the volume. The average headway is the reciprocal of the volume. This procedure is used to compute headways on Figure 20.

The second procedure is computing the headways between all vehicles at every time interval and then finding the average; This procedure is exact by definition. Figure 21 is plotted using headways computed from this exact procedure. These two procedures are discussed in the partial validation section( appendix D).
Figure 20: Effect of increasing reaction time on headway (simple method). Effect of increasing the reaction time of the drivers by 0.0, 0.1, 0.2, and 0.3 seconds on average headway (simple method) when a platoon of 15 cars goes through the regular disturbance. Volume is 1800, and units are seconds and seconds.
Figure 21: Effect of increasing reaction time on headway (exact method). Effect of increasing the reaction time of the drivers by 0.0, 0.1, 0.2, and 0.3 seconds on average headway (exact method) when a platoon of 15 cars goes through the regular disturbance. Volume is 1800, and units are seconds and seconds.
As the reaction times of the drivers increase the average headways increase, too. The average headways reach a constant value when the average density and the average speed reach a constant value. The peak of these graphs can be easily explained by considering the average density and the average speed of the platoon at that particular time.

4.5.5 BUFFER SPACE

A buffer space is additional spacing provided between vehicles for safety reasons. Its value depends on density of traffic. Three different values of buffer space examined in this analysis are 3, 6, and 10 feet. It should be mentioned that a vehicle length of 16 feet is used in CARSIM.

The effect of using the same buffer space for all speeds are shown in Figures 22-24. In Figure 22 a buffer space of 3 feet is used. It shows that vehicles are not spaced out properly, especially when they are stopped. The buffer space is increased to 6 feet and Figure 23 is obtained. Some improvements near jam density are resulted. Further increase of the buffer space to 10 feet improved the fit at higher densities, but influenced vehicles such that the affect of disturbance is not observed by simulated vehicles, Figure 24.
Figure 22: A 3 feet buffer space in all densities. Effect of using a buffer space of 3 feet in all speeds. Simulation results (k, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
Figure 23: A 6 feet buffer space in all densities. Effect of using a buffer space of 6 feet in all speeds. Simulation results (k, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
Figure 24: A 10 feet buffer space in all densities. Effect of using a buffer space of 10 feet in all speeds. Simulation results (k, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
Using a lower buffer space value near jam density results in a higher deceleration or a lower acceleration rate for the following vehicle. In real world traffic conditions vehicles near jam density do not take a buffer space of 10 feet. However, in normal traffic they may maintain a 10 foot buffer space. This difference is taken into consideration in the logic of CARSIM for computational purpose.

When density is very high and speed is very low, a buffer space of less than 10 feet should be used. A value between 3 and 7 depending on the density of traffic resulted in a very close to actual trajectories. However, making buffer space a function of density makes the model more complicated. Therefore a 3 foot buffer space near jam density, and a 10 foot elsewhere is used in the computation of acceleration or deceleration rate. The speed at which the value of the buffer space should be changed is discussed in the next section.

4.5.6 BUFFER SPACE CHANGE SPEED

At very low speed or when the vehicles are stopped, the spacing is less than when they are moving. Three different speed values below which the driver may sustain a lower buffer space are examined. These values are 10, 7 and 3 fps and the corresponding Figures are 25-29. If the speed
of the following vehicle is below this value the buffer space of 3 feet is used otherwise a 10 foot buffer space is used.

The speed at which a lower buffer space value should be used varies from 2 to 5 mph (3 to 7 fps). For platoon 123 both values were used and the results are good at both 3 fps, and 7 fps speed levels. The result for 3 fps is presented in Figure 27.
Figure 25: Different buffer spaces when speed is less than 10. Effect of using a buffer space of 3 feet when the speed of follower is less than 10 ft/sec., and 10 otherwise. Simulation results (K, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
Figure 26: Different buffer spaces when speed is less than 7. Effect of using a buffer space of 3 feet when the speed of follower is less than 7 ft/sec., and 10 otherwise. Simulation results (K, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
Figure 27: Different buffer spaces when speed is less than 3. Effect of using a buffer space of 3 feet when the speed of follower is less than 3 ft/sec, and 10 otherwise. Simulation results (K, 1-15) are compared with platoon 123. Volume is 1800 and units are feet and seconds.
The percentage of trucks in the traffic affects the speed and headways of a roadway traffic. Trucks with lower acceleration rates slow the entire line of vehicles when passing is not allowed. In a single lane flow condition, the rate of effect of traffic mix decreases by increasing the percent of trucks beyond a certain value. Five platoons of vehicles are generated randomly with 0%, 5%, 10%, 15%, and 20% trucks. The platoon of 15 vehicles is passing through a zone with a speed limit of 66 fps, while the speed limit on both sides of the zone is 80.66 fps. All vehicles must reduce their speeds when they enter the zone, and accelerate after leaving it. The length of the section is 1500 feet.

The average speed of the platoons are given in Figure 28, and the average densities in Figure 29. When the percentage of trucks increases the platoon requires longer time to reach the speed limit. Moreover, slow acceleration capability of trucks causes the average speed of the platoon to increase at a lower rate. In Figure 28 a platoon with no trucks reaches the speed limit over a half minute earlier than a platoon with trucks. More trucks in the platoon makes the difference more pronounced.
Figure 28: Variation of speed of platoon Vs traffic mix. Effect of traffic mix on average speed of a platoon of 15 vehicles passing through a zone with speed limit of 66 ft/sec. The percentage of trucks varies from 0% to 20%, volume is 1800, units are ft/sec and seconds.
Figure 29 represents the density of platoons at different times. This graph should be interpreted very carefully, otherwise misleading results could be obtained. When a platoon with zero percent trucks decelerates, the density increases due to the fact that the spacing between the first and the last car in the platoon decreases. During the acceleration phase the opposite happens and the density decreases. However, a platoon with a certain number of trucks does not produce this density fluctuations, rather it shows a decreasing trend.

For example, a platoon with 5% trucks shows a decrease in density right from the beginning. This is because of the increasing space headway between the first and the last car in the platoon. At time 48, there is an increase in density because the lead vehicle decelerates and reduces the spacing between it and the last car in the platoon. The density decreases again once the lead car is not decelerating.

The reason 10%, 15% and 20% truck levels all coincide is because of having a truck in the position of the leading cars in the platoon. This is a drawback in using the density as the criteria for evaluation of traffic conditions. No matter what is going on within the platoon, the density is always computed from the spacing between the lead and
Figure 29: Variation of density of platoon vs traffic mix. Effect of traffic mix on density of a platoon of 15 vehicles passing through a zone with speed limit of 66 ft/sec. The percentage of trucks varies from 0% to 20%, volume is 1800, units are vpm and seconds.
the last car of the platoon. Perhaps it is time to think about another way of computing the density of a platoon which is not solely dependent on the spacing of the first and last car in the platoon. One suggestion would be weighting of the spacing between vehicles and putting more emphasis on the middle cars than the end of cars.

4.6 VALIDATION

Validation of this model is performed at microscopic and macroscopic levels. At the microscopic level the location and speed of individual vehicles from actual data were compared with those obtained from the simulation model. However, at the macroscopic level aggregate parameters such as speed, density, and volume from field data are compared with the results of the simulation.

Four different platoons of vehicles covering a wide range of traffic operations are used for comparison of real world results with those obtained from the simulation models. Four simulation models each representing one particular platoon are adopted and used to replicate the actual situations. In each one of these models the attributes of the vehicles are generated randomly from the respective distributions, and are assigned to the vehicles. The location and the speed of the leaders of the simulated platoon
and the actual platoons are the same. For each vehicle in the model, the desired speed is assigned as the maximum speed a particular vehicle reached in the real world traffic, which is a random number.

Five independent replications are made for each actual traffic situation. It is essential to make several independent simulation runs to represent the same real situation. This is due to the fact that there are several stochastic variables in the simulation model which would cause the results to fluctuate from one independent run to another. In the following sections first the microscopic level and then the macroscopic level validation will be discussed.

4.6.1 VALIDATION AT MICROSCOPIC LEVEL

The speed and location of a vehicle on the road are the two most important variables in study of traffic flow dynamics. At every time interval (1 second), for every vehicle in the system the two variables are computed from the simulation models, and are compared with the real world data for each platoon.

4.6.1.1 SPEED COMPARISON

The speeds of vehicles are computed every second for all vehicles in the model. For each platoon 5 independent rep-
lications were made and an average of 5 speeds are used as the speed of the vehicle at that particular time. These speeds and the actual speeds are plotted for different platoons in Figure 30-33.

In Figure 30 the speeds are given for the first vehicle, the last vehicle, and every third vehicle in platoon 123 with 15 cars. The simulation model generated the same pattern developed by the actual platoon. All vehicles came to a standstill and then accelerated to reach their desired speed. The fact that for a given vehicle the speed change curve is similar and parallel to that of actual counterpart vehicle indicates the ability of this simulation model to replicate the real world situation realistically. If the reaction time of the actual drivers were known the agreement would have been improved considerably.

In Figure 31 the speed patterns are represented for all vehicles in platoon 126. Three vehicles leave the platoon, and one vehicle enters into the platoon at different times. The speed change patterns are very close to that of the actual traffic. After the disturbance the last few vehicles reach a higher speed than the actual vehicles because of higher desired speed. In the actual platoon these vehicles are no longer in the platoon.
**Figure 30:** Speed change patterns from simulation vs platoon 123. Comparison of speed change patterns for every third car from simulation model (K, 1-15) versus platoon 123. Volume is 1800, units are ft/sec and seconds.
Figure 31: Speed change patterns from simulation Vs platoon 126. Comparison of speed change patterns from simulation model(K, 1-16) versus platoon 126. Volume is 1800, units are ft/sec and seconds.
Figure 32: Speed change patterns from simulation Vs platoon 127. Comparison of speed change patterns for every other car from simulation model(K, 1-10) versus platoon 127. Volume is 1800, units are ft/sec and seconds.
Figure 33: Speed change patterns from simulation vs platoon L123x. Comparison of speed change patterns from simulation model(K, 1-5) versus platoon L123x. Volume is 1800, units are ft/sec and seconds.
These two graphs represent the situations of having a wide range of speed in a platoon before going through a kinematic disturbance. In platoon 123 the speed difference between the first and the last car in the platoon is about 70 fps at the beginning. For platoon 126 the difference is not high at the beginning, but increases before suffering from the disturbance. Both platoons are forced to stop for a while due to a severe kinematic disturbance.

The situations where the vehicles in the platoon go through a severe kinematic disturbance in a very short period of time (less than 1 minute) are shown in Figure 32 and 33. Figure 32 exhibits the simulation and actual speed change patterns for every other vehicle including the last vehicle in a platoon of 10 cars. There is a very good agreement between the results of the model and real data.

The speed change patterns developed before and after a severe kinematic disturbance and the effect of the speed fluctuation of the leader on the following cars, are shown in Figure 33. The simulation model duplicates the real situation remarkably well for the entire period of 202 seconds.

From the speed change pattern comparison, it is clear that the model duplicates the real world speed change patterns very well in various traffic conditions.
4.6.1.2 TRAJECTORY COMPARISON

The location of every vehicle is computed at every time interval for all platoons. For each platoon 5 independent replications are made. Thus there are 5 locations for each car in every second. The plot of the average of these 5 values versus the actual data for different platoons are shown in Figures 34-37. For platoon 123 the trajectory plots for every third vehicle including the last vehicle is shown. Whereas for platoon 126 and L123x the plots show the trajectories for all vehicles in the platoons. For platoon 127 the plots are given for every other vehicle including the last vehicle.

The trajectory comparison is one of the most difficult tasks in validation of traffic flow models when vehicles are going through a kinematic disturbance. When flow is not in steady state, it is quite challenging to generate a trajectory plot that is close enough to the actual trajectory plot. It is not very difficult to reproduce close-to-actual trajectory plots when the flow is not subjected to a severe kinematic disturbance, or only the acceleration or deceleration phase of a kinematic disturbance is simulated. For instance, the INTRAS model used only deceleration phase of a platoon for trajectory comparison and did not use the acceleration phase, see Figure 1. It becomes complex when
the whole phase of a stop-and-go operation is realistically replicated.

The trajectory plots of four different platoons of vehicles when suffering from a severe kinematic disturbance are compared with the trajectories of the actual platoons. Such a detailed comparison is unique for this model and the very close agreement between the simulation and the actual data indicates the ability of this model to reproduce the real world situation in a very realistic manner, see Figure 34-37.

These Figures are very close to the plots of every replication versus the actual data. The variations among runs are very reasonable and in all runs the plots are showing an accurate replication of the actual situation.

From Figures 33-37 one can observe the "widening" effect of the disturbance at higher densities. When the deflection points before and after the stoppage for different vehicles are connected to each other, the two lines diverge as the disturbance propagates. This phenomenon happens in real world traffic and not only causes propagation of disturbance along the line of vehicles, but also causes longer delay for the following vehicles. Figure 37 represents a situation where the platoon is subjected to a kinematic disturbance as well as normal traffic operation without
Figure 34: Trajectories from simulation Vs platoon 123. Comparison of trajectories plots for every third car from simulation model(K, 1-15) versus platoon 123. Volume is 1800, units are feet and seconds.
Figure 35: Trajectories from simulation Vs platoon 126. Comparison of trajectories plots from simulation model(K, 1-16) versus platoon 126. Volume is 1800, units are ft/sec and seconds.
Figure 36: Trajectories from simulation Vs platoon 127. Comparison of trajectories plots for every other car from simulation model(K, 1-10) versus platoon 127. Volume is 1800, units are ft/sec and seconds.
Figure 37: Trajectories from simulation vs platoon L123x. Comparison of trajectories plots from simulation model (K, 1-5) versus platoon L123x. Volume is 1800, units are ft/sec and seconds.
severe speed changes. The data used here are about twice as long as in duration than the other models. The attempt is to show that the model can be used for a longer or a shorter period, for normal or congested traffic.

From comparison of the trajectory plots one can conclude that at the microscopic level the model performance is excellent. This model is very appropriate for detail study of traffic dynamics such as, wave propagation and dissipation.

4.6.2 VALIDATION AT MACROSCOPIC LEVEL

For macroscopic validation the overall performance of platoons of vehicles are evaluated rather than the performance of an individual vehicle. Platoons 123, 126, and 127 are used and the following tasks are performed for each one of the platoons.

1. Comparison of flow parameters over time.
2. Comparison of fundamental relations of traffic flow.
3. Regression of simulation results versus actual data.
4. Examination of variation of responses due to stochastic input.

4.6.2.1 VARIATION OF FLOW PARAMETERS OVER TIME

The three flow parameters used for investigation of propagation of disturbances are speed, density, and volume.
For each one of the three platoons, the respective simulation model is run 5 times (5 independent replications) and the average of speed, density, and volume of traffic is computed in one second intervals in all the runs. Thus, the average values are the means of 5 independent observations at a given time.

The plot of the average speed from the simulation runs versus the speed from the field data for different platoons are shown in Figures 38-40. All three platoons suffered from a severe kinematic disturbance. The simulation results are very close to the actual traffic speeds and produce the same pattern of speed fluctuation. The platoons traveling at a speed of more than 80 fps reached speeds of near zero in less than 1 minute. The rate of speed reduction for a platoon can easily be computed from such graphs. The simulation curves are showing less local fluctuation than the curves for the field data, as it is expected.

The plot of densities versus time for platoons 123, 126, 127, and the simulation counterparts are shown in Figures 41-43. The graphs show the same patterns and fluctuations for both simulation and actual platoons. The time a simulated platoon reaches the jam density is very close to that of the actual platoon.
Figure 38: Average speed from simulation Vs platoon 123. Comparison of the average speed from simulation model(solid line) versus field data(------) for platoon 123. Volume is 1800, units are ft/sec and seconds.
Figure 39: Average speed from simulation vs platoon 126. Comparison of the average speed from simulation model(solid line) versus field data(-----) for platoon 126. Volume is 1800, units are ft/sec and seconds.
Figure 40: Average speed from simulation vs platoon 127. Comparison of the average speed from simulation model (solid line) versus filed data (-----) for platoon 127. Volume is 1800, units are ft/sec and seconds.
Figure 41: Density from simulation Vs platoon 123. Comparison of density from simulation model (solid line) versus field data (----) for platoon 123. Volume is 1800, units are vpm and seconds.
Figure 42: Density from simulation vs platoon 126. Comparison of density from simulation model (solid line) versus field data (-----) for platoon 126. Volume is 1800, units are vpm and seconds.
Figure 43: Density from simulation Vs platoon 127. Comparison of density from simulation model (solid line) versus filed data (-----) for platoon 127. Volume is 1800, units are vpm and seconds.
The density of a platoon is computed from the position difference of the first and the last car in the platoon. It is very dependent to the spacing between these cars. Therefore, one should be very careful in using the density of a platoon for comparison of actual and simulated results.

The maximum density a platoon reaches depends on severity of the disturbance and its duration. For example, for platoon 126 it is much less than that of platoon 127. This difference is also created by the two simulation models. An accurate freeway simulation model should produce reasonable results in a wide range of density.

The volumes of the simulated platoons versus the actual platoons are presented in Figures 44-46. The volumes are computed at one second time intervals by multiplying speed by density for that moment. This may cause a large difference between volumes computed from actual data and simulated results when both speed and density are underestimated or overestimated. Therefore, it is not a very good criteria for this comparison. Another reason for not using volume is the fact that traffic volume is not computed directly, but is obtained by multiplying the speed and density for one second and then converting it to hourly volume. At the unstable flow region this relationship may not
always be true. The average speed and the average density should be used instead of volume. If volume is to be used it should be computed directly from the count of the number of vehicles passing a point on the road.

The relationships between the speed, density, and volume computed from the actual data with those obtained from the simulation models will be compared in the following sections.
Figure 44: Volume from simulation Vs platoon 123. Comparison of volume from simulation model(solid line) versus field data(-----) for platoon 123. Volume is 1800, units are vph and seconds.
Figure 45: Volume from simulation Vs platoon 126. Comparison of volume from simulation model(solid line) versus field data(-----) for platoon 126. Volume is 1800, units are vph and seconds.
Figure 46: Volume from simulation vs platoon 127. Comparison of volume from simulation model (solid line) versus field data (-----) for platoon 127. Volume is 1800, units are vph and seconds.
4.6.2.2 FUNDAMENTAL RELATIONS OF TRAFFIC FLOW

The relationships between speed, density, and volume for all platoons are computed from the actual and simulated data. The graphs showing speed-density, speed-volume, and volume-density relationships will be discussed separately.

The speed-density relationships for simulated and actual platoons are presented in Figures 47-49. The actual data shows a nonlinear relationship and the hysteresis phenomenon. The simulation results present the same phenomenon and supports the idea of dual behavior of traffic. For platoon 127 in which all vehicles before and after the disturbance are in car following model the loop is more distinct. When the plot of individual simulation results are made the graphs show even more distinct loops than these plots which are average of 5 replications.

The speed-volume, and volume-density relationships on Figures 50-55 also produced loops that are very similar to those generated from the actual data. All these graphs are indicating that the magnitude of change in speed, density, and volume when the platoon is going into a kinematic disturbance are not the same as when the platoon is coming out from the disturbance. Due to this difference one can see loops on Figures 50-55, which are supporting the hysteresis phenomenon or dual behavior of traffic flow.
Figure 47: Speed-density plots for platoon 123. Comparison of speed-density relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 123. Volume is 1800, units are ft/sec and vpm.
Figure 48: Speed-density plots for platoon 126. Comparison of speed-density relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 126. Volume is 1800, units are ft/sec and vpm.
Figure 49: Speed-density plots for platoon 127. Comparison of speed-density relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 127. Volume is 1800, units are ft/sec and vpm.
In addition to the graphical presentation of the results, the statistical analysis of the simulation results versus the actual data is also carried out. This will be discussed in the following section.
Figure 50: Volume-density for platoon 123. Comparison of volume-density relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 123. Volume is 1800, units are vph and vpm.
Figure 51: Volume-density for platoon 126. Comparison of volume-density relationships obtained from the simulation model (solid lime) with field data (-----) for platoon 126. Volume is 1800, units are vph and vpm.
Figure 52: Volume-density for platoon 127. Comparison of volume-density relationships obtained from the simulation model (solid line) with field data (-----) for platoon 127. Volume is 1800, units are vph and vpm.
Figure 53: Speed-volume plots for platoon 123. Comparison of speed-volume relationships obtained from the simulation model (solid lime) versus field data (-----) for platoon 123. Volume is 1800, units are ft/sec and vph.
Figure 54: Speed-volume plots for platoon 126. Comparison of speed-volume relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 126. Volume is 1800, units are ft/sec and vph.
Figure 55: Speed-volume plots for platoon 127. Comparison of speed-volume relationships obtained from the simulation model (solid line) versus field data (-----) for platoon 127. Volume is 1800, units are ft/sec and vph.
4.6.2.3 SIMULATION RESULTS VERSUS ACTUAL DATA

The traffic parameters computed from the simulation models versus the values obtained from the field data are shown in Figures 38-46. Regression of simulation results versus field data were carried out for speed, density, and volume for all platoons.

Three kinds of regression analysis are performed for each set of data.
A- Individual simulation replication versus field data.
B- Average of simulation replications versus field data
C- Simulation replications combined versus field data.

INDIVIDUAL SIMULATION REPLICATIONS VERSUS FIELD DATA:

Regression of the platoon speed, density, and volume computed from individual simulation runs versus the values obtained from field data are carried out for each replications of each platoon. There are 3 parameters, 5 replications for each platoon, and 3 platoons. Thus, a total of 45 regression lines are obtained. the results are summarized in Tables 3-5.
Table 3

Regression of replications for platoon 123

Results of regressing speed, density, and volume from simulation model versus the values from field data for platoon 123. Five independent replications are made.

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Regression of model speeds versus field speeds for all
Results of regressing speed, density, and volume from simulation model versus the values from field data for platoon 126. Five independent replications were made.

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<td>40.63442</td>
<td>0.02654</td>
</tr>
<tr>
<td>4</td>
<td>393.20728</td>
<td>0.77748</td>
<td>0.91162</td>
<td>46.33629</td>
<td>0.03026</td>
</tr>
<tr>
<td>5</td>
<td>374.60908</td>
<td>0.81648</td>
<td>0.85502</td>
<td>64.35406</td>
<td>0.04203</td>
</tr>
</tbody>
</table>

Three platoons in all 5 replications yield in a slope very
Table 5

regression of replications for platoon 127

Results of regressing of speed, density, and volume from simulation model versus the values field data for platoon 127. Five independent replications were made.

<table>
<thead>
<tr>
<th>replications</th>
<th>b0</th>
<th>b1</th>
<th>R**2</th>
<th>s(b0)</th>
<th>s(b1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPEED</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>-1.53776</td>
<td>1.03852</td>
<td>0.98558</td>
<td>0.56593</td>
<td>0.01182</td>
</tr>
<tr>
<td>2</td>
<td>-1.66176</td>
<td>1.03986</td>
<td>0.98619</td>
<td>0.55426</td>
<td>0.01157</td>
</tr>
<tr>
<td>3</td>
<td>-1.14351</td>
<td>1.02580</td>
<td>0.98533</td>
<td>0.56390</td>
<td>0.01178</td>
</tr>
<tr>
<td>4</td>
<td>-0.03180</td>
<td>0.99381</td>
<td>0.97776</td>
<td>0.67525</td>
<td>0.01410</td>
</tr>
<tr>
<td>5</td>
<td>-1.32494</td>
<td>1.03513</td>
<td>0.98586</td>
<td>0.55818</td>
<td>0.01166</td>
</tr>
<tr>
<td>DENSITY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3.17037</td>
<td>0.94254</td>
<td>0.97955</td>
<td>1.17635</td>
<td>0.01281</td>
</tr>
<tr>
<td>2</td>
<td>3.07176</td>
<td>0.96696</td>
<td>0.98197</td>
<td>1.13201</td>
<td>0.01233</td>
</tr>
<tr>
<td>3</td>
<td>0.65327</td>
<td>0.93097</td>
<td>0.97066</td>
<td>1.39818</td>
<td>0.01523</td>
</tr>
<tr>
<td>4</td>
<td>2.35453</td>
<td>0.89203</td>
<td>0.96485</td>
<td>1.47075</td>
<td>0.01602</td>
</tr>
<tr>
<td>5</td>
<td>3.05183</td>
<td>0.96496</td>
<td>0.97984</td>
<td>1.19575</td>
<td>0.01302</td>
</tr>
<tr>
<td>VOLUME</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>34.03149</td>
<td>0.95374</td>
<td>0.86962</td>
<td>60.76206</td>
<td>0.03474</td>
</tr>
<tr>
<td>2</td>
<td>-93.00927</td>
<td>1.04603</td>
<td>0.89329</td>
<td>59.48541</td>
<td>0.03401</td>
</tr>
<tr>
<td>3</td>
<td>-39.87405</td>
<td>0.93803</td>
<td>0.88502</td>
<td>55.63096</td>
<td>0.03181</td>
</tr>
<tr>
<td>4</td>
<td>53.79439</td>
<td>0.87962</td>
<td>0.89516</td>
<td>49.53159</td>
<td>0.02832</td>
</tr>
<tr>
<td>5</td>
<td>13.69972</td>
<td>0.98503</td>
<td>0.87240</td>
<td>61.98424</td>
<td>0.03544</td>
</tr>
</tbody>
</table>

close to 1 and a y-intercept in the order of a few feet.
The R-Squared values for platoon 123, 126, and 127 are at least 98% or higher. The values for slope, y-intercept, and R-Squared are fairly consistent and do not vary very much from one replication to another.

Regression of the simulation densities versus the real world densities yield a linear relationship with slope close to one and a small y-intercept. The R-Squared value is 97% or higher. When the slope is greater than unity the y-intercept is negative, and when the slope is less than 1 the y-intercept is positive. This combination would reduce the difference between the model results and the actual world outcomes.

The variance of slope and y-intercept do not change very much from one replication to another. Tables 3-5 show that the independent simulation runs yielded fairly consistent results. One can conclude that the model simulates the actual traffic operation accurately in various traffic conditions.

Regression of the model volume versus the actual volume resulted in R-Squared values at least 80% or higher. The slope is not as close to unity as it was in case of the speed and density; and y-intercepts are not close to zero. Lower R-Squared value for volume comparison is due to the fact that instantaneous volumes computed as the product of the speed and density are used.
The advantage of making regression of individual simulation runs versus using average of 5 replications is the additional information one can obtain about variation of the responses among replications. When the average value is used this information is no longer available, but the results are easy to be interpreted. In the following section the average values are used for analysis.

**AVERAGE OF THE REPLICATIONS VERSUS FIELD DATA:**

Average values of speed, density, and volume are computed from the replications for every parameter. Thus instead of 5 simulation results for each platoon, there is only one simulation output which is the average of 5 runs. These average values are used for evaluation of simulation results versus the field data. Tables 6-8 show some of the parameters obtained from regression analysis.

The values computed for slopes and y-intercepts are very close to the previous values. Conclusions similar to those of the previous section can be made here, too. For speed and density the slopes of the regression lines are very close to unity and y-intercepts are in the order of a few feet. The R-Squared values are not less than 0.98. For volume the R-Squared values are greater than 0.84.

Graphical presentation of the average speed, density, and volume versus real world traffic values are presented
in Figures 56-64. As it can be seen, the points are scattered along the line of Y=X, indicating that the model values are proportional to field results with a constant of proportionality of very close to one.

When the average values of several simulation replications are used, the outputs are less sensitive to a particular stochastic input value in a given simulation run. Thus, the results are more stable and show the trend in the response variable. Since each value is the average of 5 points, one might assume they are repeated observations at a given point. The consequence of such an assumption is discussed in the following section.
Figure 56: Simulation speed Vs actual speed for platoon 123. Plot of average speed from simulation model(ASPEED) versus field data(SPDREAL) for platoon 123. Volume is 1800, unit is ft/sec.
Figure 57: Simulation speed Vs actual speed for platoon 126. Plot of average speed from simulation model (ASPEED) versus field data (SPDREAL) for platoon 126. Volume is 1800, unit is ft/sec.
Figure 58: Simulation speed vs actual speed for platoon 127. Plot of average speed from simulation model(ASPEED) versus field data(SPDREAL) for platoon 127. Volume is 1800, unit is ft/sec.
Figure 59: Simulation density Vs actual density for platoon 123. Plot of average density from simulation model (ADENSITY) versus field data (DENREAL) for platoon 123. Volume is 1800, unit is vpm.
Figure 60: Simulation density Vs actual density for platoon 126. Plot of average density from simulation model (ASPEED) versus field data (SPDREAL) for platoon 126. Volume is 1800, unit is vpm.
Figure 61: Simulation density Vs actual density for platoon 127. Plot of average density from simulation model (ASPEED) versus field data (SPDREAL) for platoon 127. Volume is 1800, unit is vpm.
Figure 62: Simulation volume vs actual volume for platoon 123. Plot of average volume from simulation model (AVOLUME) versus field data (VOLREAL) for platoon 123. Volume is 1800, unit is vph.
Figure 63: Simulation volume Vs actual volume for platoon 126. Plot of average volume from simulation model (AVOLUME) versus field data (VOLREAL) for platoon 126. Volume is 1800, unit is vph.
Figure 64: Simulation volume vs actual volume for platoon 127. Plot of average volume from simulation model (AVOLUME) versus field data (VOLREAL) for platoon 127. Volume is 1800, unit is vph.
Table 6

Regression of model speed vs actual speed

Results of regression of model speed versus actual speed when data from 5 independent replications are combined, and when average of 5 runs are computed; for all platoons.

<table>
<thead>
<tr>
<th>type of data</th>
<th>platoon number</th>
<th>b0</th>
<th>b1</th>
<th>R**2</th>
<th>s(b0)</th>
<th>s(b1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>all 5 runs combined</td>
<td>123</td>
<td>1.70655</td>
<td>0.94705</td>
<td>0.98100</td>
<td>0.23682</td>
<td>0.00588</td>
</tr>
<tr>
<td>average of 5 runs</td>
<td>123</td>
<td>1.70554</td>
<td>0.94705</td>
<td>0.98383</td>
<td>0.49177</td>
<td>0.01220</td>
</tr>
<tr>
<td>all 5 runs combined</td>
<td>126</td>
<td>3.26491</td>
<td>0.93782</td>
<td>0.98325</td>
<td>0.30895</td>
<td>0.00676</td>
</tr>
<tr>
<td>average of 5 runs</td>
<td>126</td>
<td>3.26491</td>
<td>0.93782</td>
<td>0.98621</td>
<td>0.63354</td>
<td>0.01386</td>
</tr>
<tr>
<td>all 5 runs combined</td>
<td>127</td>
<td>-1.14070</td>
<td>1.02664</td>
<td>0.98394</td>
<td>0.26244</td>
<td>0.00548</td>
</tr>
<tr>
<td>average of 5 runs</td>
<td>127</td>
<td>-1.14070</td>
<td>1.02664</td>
<td>0.98708</td>
<td>0.52923</td>
<td>0.01105</td>
</tr>
</tbody>
</table>
Table 7

Regression of model density vs actual density

Results of regression of model density versus actual density when data from 5 replications are combined, and when average of 5 runs are computed; for all platoons.

<table>
<thead>
<tr>
<th>type of data</th>
<th>platoon number</th>
<th>b0</th>
<th>b1</th>
<th>R**2</th>
<th>S(b0)</th>
<th>S(b1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>all 5 runs</td>
<td>123</td>
<td>1.44075</td>
<td>0.98431</td>
<td>0.98123</td>
<td>0.52091</td>
<td>0.00607</td>
</tr>
<tr>
<td>combined</td>
<td>126</td>
<td>-5.12594</td>
<td>1.08229</td>
<td>0.97659</td>
<td>0.83388</td>
<td>0.00925</td>
</tr>
<tr>
<td>average of 5</td>
<td>127</td>
<td>2.46060</td>
<td>0.93949</td>
<td>0.97050</td>
<td>0.62838</td>
<td>0.00684</td>
</tr>
<tr>
<td>runs</td>
<td>123</td>
<td>1.43827</td>
<td>0.98432</td>
<td>0.98922</td>
<td>0.88620</td>
<td>0.01033</td>
</tr>
<tr>
<td></td>
<td>126</td>
<td>-5.12594</td>
<td>1.08229</td>
<td>0.98183</td>
<td>1.65879</td>
<td>0.01841</td>
</tr>
<tr>
<td></td>
<td>127</td>
<td>2.46060</td>
<td>0.93949</td>
<td>0.97765</td>
<td>1.22708</td>
<td>0.01336</td>
</tr>
</tbody>
</table>
Table 8

Regresssion of model volume vs actual values

results of regression of model volume versus actual volume when data from independent replications are combined, and when average of the 5 runs are used; for all platoons

<table>
<thead>
<tr>
<th>type of data</th>
<th>platoon number</th>
<th>b0</th>
<th>b1</th>
<th>R**2</th>
<th>s(b0)</th>
<th>s(b1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>all 5 runs combined</td>
<td>123</td>
<td>349.47228</td>
<td>0.79267</td>
<td>0.80913</td>
<td>28.55661</td>
<td>0.01717</td>
</tr>
<tr>
<td>average of 5 runs</td>
<td>126</td>
<td>412.41160</td>
<td>0.75583</td>
<td>0.86054</td>
<td>25.72670</td>
<td>0.01680</td>
</tr>
<tr>
<td></td>
<td>127</td>
<td>-6.27216</td>
<td>0.96049</td>
<td>0.86568</td>
<td>27.64479</td>
<td>0.01581</td>
</tr>
</tbody>
</table>

THE REPLICATIONS COMBINED VERSUS FIELD DATA:

The values obtained for individual simulation replications for a platoon are combined, and regression of these repeated values versus values computed from the field data are performed. Summary of the results are given in Tables 6-8.

As it is expected the slopes and y-intercepts are the same as when the average values were used. The R-Squared values decreased a little bit, however, the variance of slopes and y-intercepts decreased almost to a half of the average values.
Since it seems that we have repeated observations for a given value of field data parameter, the ANOVA tables were examined for possible lack of fit of the regression lines. Not surprisingly the answer was positive, and the analysis falsely indicated a lack of fit. Residuals were plotted versus time and predicted values in order to see possibility of a higher order model. The plots of residuals versus time, and residuals versus predicted variables did not show any definite trend. The residuals were clustered around the line of Y=0, and approximately making a horizontal band. Such a band is an indication of adequacy of the model.

Therefore, false detection of lack of fit was not due to inadequacy of the model, but rather using the lack of fit test, which is not fully justifiable for this because of strong correlations between the points. The connections between successive points are very important in this model and the lack of fit test does not take into account this connection.

4.6.2.4 VARIATION OF THE RESPONSE VARIABLES

Since several stochastic input variables are used in each model, the responses would change from one independent simulation run to another. Thus the results obtained from a single simulation run may not be very reliable. To
increase the reliability and confidence on the simulation responses one should make either a long run or several replications in order to make some statistical inferences about the responses and their variabilities.

Each simulation model is replicated five times and the variation of the response variables are graphically presented for each platoon. For each run independent random numbers are used.

The variation of average speed of the platoons versus time for all platoons are shown in Figures 65-67. Such a variation is expected, and the range of it is very reasonable.

Density of a platoon depends on relative location of the last car to the first car in the platoon. The position of the last car would depend on the reaction time of the last drivers. If the last driver has a longer reaction time, then he is going to follow his leader with a longer spacing. As a result the platoon will have a lower density. On the other hand if the last driver has lower reaction time, the density will be higher for the platoon. Since the reaction time of the drivers are assigned randomly, more density variation is expected. Figures 68-70 illustrate the density variations for different platoons.
Figure 65: Speed variation for platoon 123. Speed variation among independent replications of a simulation model for platoon 123. Units are ft/sec and seconds.
Figure 66: Speed variation for platoon 126. Speed variation among independent replications of a simulation model for platoon 126. Units are ft/sec and seconds.
Figure 67: Speed variation for platoon 127. Speed variation among independent replications of a simulation model for platoon 127. Units are ft/sec and seconds.
Figure 68: Density variation for platoon 123. Density variation among independent replications of a simulation model for platoon 123. Units are vpm and seconds.
Figure 69: Density variation for platoon 126. Density variation among independent replications of a simulation model for platoon 126. Units are vpm and seconds.
Figure 70: Density variation for platoon 127. Density variation among independent replications of a simulation model for platoon 127. Units are vpm and seconds.
The variation of volume computed from independent simulation runs are shown in Figures 71-73 for platoons 123, 126, and 127. A consistent pattern can easily be seen. A high density and a high speed yielded a high volume and vice versa.

Independent replications of the simulation runs and the variation of the responses among different runs indicate that the response variables are fairly stable, and there are no unreasonable changes among runs. The consistent patterns in all of the response variables is enhancing the validity of this simulation under a variety of traffic flow conditions; specially when the vehicles are forced to decelerate rapidly to a complete stop and start moving again after a while of being halted.
Figure 71: Volume variation for platoon 123. Volume variation among independent replications of a simulation model for platoon 123. Units are vph and seconds.
Figure 72: Volume variation for platoon 126. Volume variation among independent replications of a simulation model for platoon 126. Units are vph and seconds.
Figure 73: Volume variation for platoon 127. Volume variation among independent replications of a simulation model for platoon 127. Units are vph and seconds.
CHAPTER V

TRAFFIC WAVE STUDIES AT MICROSCOPIC LEVEL

5.1 INTRODUCTION

CARSIM is used for microscopic level investigation of the propagation and dissipation of traffic waves under various traffic conditions. The model gives the speed and location of every vehicle at each scanning time (one second). The slowdown, stopping, starting(moving), and recovering waves can precisely be located down the line of vehicles, and their speeds can accurately be computed at a given time or location.

For the first time, the propagation and dissipation of traffic waves at microscopic level is investigated. First, the effect of deceleration rate of the lead car on the propagation of slowdown and stopping waves is examined. Then, the effect of traffic volume on the propagation of slowdown waves and stopping waves is investigated. Later on, the effect of an incident is simulated and the dynamics of four different traffic waves is analyzed. Finally, the effect of the mainline traffic control on congestion alleviation is discussed in this chapter.
5.2 SLOWDOWN WAVES

The leader of a group of cars decelerates at various rates and makes the following vehicles to slow down. The location and the time the following vehicles slow down due to the deceleration of the lead car are recorded for all cars. A group of 200 cars, generated from an arrival volume of 1800 vph, all traveling at a constant speed of 80.67 fps on a 7 mile section of a freeway is used for this study. The leader of the platoon is asked to decelerate at a rate of 4, 8, 12, and 16 ft/sec/sec to a complete stop. The simulation is run for 850 seconds to allow all vehicles to come to a standstill position.

Five replications are made for each deceleration rate. For a given deceleration rate the average of these five independent runs is computed for every vehicle. The results are graphically presented using coordinates of time, distance, and identification of vehicles.

The location of the slowdown waves while propagating down the line of vehicles are shown in Figure 74 for the first 50 cars, and in Figure 75 for all cars in the section. For the first several vehicles the effects of various deceleration rates are different. This difference is increasing up to a point and then levels off and almost parallel graphs showing relatively constant differences are
obtained. At lower deceleration rates, 4 or 8, the following vehicles initiate deceleration at a location closer to the point the lead car starts slowing down than at the higher deceleration rates.

These graphs can be used to find the location a certain vehicle will be affected given the deceleration rate of the lead car. Similarly, one can find how far the effect of the lead car deceleration is propagated and how many cars are affected due to the deceleration. Knowing how many vehicles are affected and their location on the road is very useful in bottleneck traffic control.

Furthermore, a set of linear models are fitted for different rates by using regression analysis techniques. From these equations one can calculate the number of vehicles affected at a given location, or can find the location a certain vehicle will be affected. In these equations the lead car initiated deceleration when it was at 36382.6 feet from the beginning of the study section. The lead car reached this point after traveling 450 seconds on the study section which is when it began to decelerate. The relationship between the number of vehicles (N) and the location (D in feet) the following vehicles will slow down when the lead vehicle decelerates to a complete stop at a rate of 4, 8, 12, or 16 ft/sec/sec. are as following:

\[ D = 81.6 - 24.79N \]
Figure 74: Location of slowdown for first 50 cars. The location the slowdown waves effect the first 50 cars (ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, unit is feet.
Figure 75: Location slowdown for all cars. The location slowdown waves effect the following cars(ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, unit is feet.
\[ D = -268.8 - 25.196N \]
\[ D = -391.1 - 25.290N \]
\[ D = -452.9 - 25.307N \]
respectively. The R-squared values are: 0.996, 0.996, 0.996, and 0.995 respectively.

These equations should not be used for the first several vehicles in the platoon. Although, for the computation of these equations the slowdown data for all 200 vehicles are used, the use of these equations are not recommended for several cars immediately following the lead vehicle. This restriction is suggested because of the different propagation patterns observed in Figures 74 and 75. After approximately the first ten cars the fluctuations of the location of the slowdown versus identification (number) of vehicles show a consistent pattern. The differences between the graphs for the different rates are not showing an increasing pattern as it does for the first few cars.

The slope of these lines are very close and that is in the neighborhood of -25, but their y-intercepts are clearly different. The differences between the y-intercepts of these equations show the additional distance the slowdown wave will propagate because of the higher deceleration rate.
The time it would take for a slowdown wave to reach a certain vehicle is shown in Figures 75 and 76. As it can be seen, the lead car decelerates at various rates in different experiments at 451 seconds. The propagation is faster for the first several vehicles and then fluctuations show a constant pattern. From these graphs one can read when a vehicle will slow down and how much difference there is from one deceleration rate to another.

The difference between the time a 16 ft/sec/sec deceleration rate and a 12 ft/sec/sec deceleration rate will reach a certain vehicle, is less than the difference between a 12 ft/sec/sec and an 8 ft/sec/sec and much less than the difference between an 8 ft/sec/sec and a 4 ft/sec/sec. These differences remain fairly constant as shown by parallel graphs on Figures 76 and 77.

The number of vehicles required to slow down at a certain time due to deceleration of the lead car can be easily found from these graphs. The first vehicle decelerates at 451 seconds, and the following vehicles thereafter. To find out how many vehicles are affected by the deceleration of the lead car 3 minutes later, add 180 to 451 and read the number of vehicles corresponding to this value on the horizontal axis of the graphs.
Figure 76: Time of Slowdown for first 50 cars. The time the slowdown waves effect the first 50 cars (ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, unit is second.
Figure 77: Time of slowdown for all cars. The time the slowdown waves effect the following cars (ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, unit is second.
A set of equations to express the relationship between the time and the number of vehicles affected by the lead car's deceleration are found using regression analysis. These equations for slowdown wave propagation in time are listed below. The relationships between the number of vehicles (N) the time (T in second) a slowdown wave propagates to a certain vehicle when the lead car decelerates at a rate of 4, 8, 12, or 16 ft/sec/sec to a complete stop are as follows:

\[
\begin{align*}
T &= -3.86 + 1.7321N \\
T &= -8.20 + 1.7271N \\
T &= -9.72 + 1.7259N \\
T &= -10.49 + 1.7257N
\end{align*}
\]

respectively. The R-squared values are: 0.999, 0.999, 0.999, and 0.9999, respectively.

It is not recommended to use these equations for the first several vehicles (e.g. first 10) because of the different propagation patterns which are observed on Figures 76 and 77. For the first several vehicles one can read the time directly from the graphs. The slopes of these lines are so close that one may consider them parallel. The unequal y-intercepts for these lines indicate the difference in propagation times of the various deceleration rates. These differences remain practically constant as the number of vehicles increase.
For instance, the time the 8 ft/sec/sec rate and the 16 ft/sec/sec rate will reach the 100th vehicle are 164.51 sec. and 162.08 sec., respectively. This difference here is 2.43 sec. versus the difference in y-intercepts which is 2.29 seconds. The difference is negligible and one may assume that the differences are constant.

Another way of using these equations is finding how many vehicles are involved for a given period of time after the lead car initiates the deceleration. For example, five minutes ago the lead car decelerated to a complete stop due to an accident on the road with a rate of 8 ft/sec/sec. The number of vehicles influenced by the stopping can be computed as $N = \frac{308.20}{1.7271} = 179$ vehicles.

The propagation of the slowdown waves in time and location are given in Figures 78 and 79. For higher deceleration rates the slopes of the graphs are very steep at the beginning, however, for a deceleration rate of 4 ft/sec/sec there is not a rapid propagation. For all deceleration rates a consistent pattern is observed beyond the initial rapid deceleration.

The speed of propagation of a slowdown wave can be computed using these graphs. Four regression lines were fitted one for each rate using the information for all 200 cars. The equations for the propagation of speed of the slowdown waves are:
Figure 78: Propagation of slowdown wave for first 50 cars. Propagation of the slowdown waves down the first 50 cars (ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, units are ft and sec.
Figure 79: Propagation of slowdown wave for all cars. Propagation of the slowdown waves down the line of cars (ID) when the lead car decelerates at a rate of 4, 8, 12, and 16 ft/sec/sec (ratecodes 1 to 4, respectively). Volume is 1800, units are ft and sec.
D = 37.36 - 14.2946T
D = -377.81 - 14.5647T
D = -523.30 - 14.6267T
D = -596.61 - 14.6370T

for deceleration rates of 4, 8, 12, and 16 ft/sec/sec, respectively. The R-squared values are 0.9941, 0.9934, 0.9924, and 0.9920.

The slopes of the graphs are close, but the y-intercepts are different. All data points for all rates are used to find the slope of an average deceleration wave. The slope of the lines show that the propagation is faster at the higher rates than at the lower rates. Furthermore, large negative y-intercepts at higher deceleration rates indicate a deeper propagation. Figure 78 clearly shows the difference in location of a slowdown wave at various rates.

The slope of an average deceleration wave is -14.45. This means that an average slowdown wave moves opposite to the direction of flow at a rate of 14.45 ft/sec.

To find the location of a slowdown wave at a given time one can use these equations. These equations should not be used for the first several seconds right after deceleration of the lead car. Figure 78 should be used instead. Such a restriction is due to a higher slope right after initiation of the slowdown and fairly constant slope after that.
Figures 74-79 are developed using an arrival volume of 1800 vph, and a maximum deceleration rate of 16 ft/sec/sec for all vehicles at all densities. For lower volumes similar graphs can easily be developed.

5.3 STOPPING WAVES

The leader of the group of 200 cars is directed to decelerate at a rate of 8 and 16 ft/sec/sec, in two different experiments, to a complete stop. Traffic volume is 1800 vph and the study section is a 7 mile straight section of freeway with no traffic volume change opportunity. The leader is asked to decelerate at 450 seconds after the beginning of the simulation and to remain in a standstill situation until the end of the simulation time which is 850 seconds.

For each deceleration rate, five independent replications are made, and an average of these runs are used to find a relationship between the stopping wave propagation and time or distance. When a vehicle reaches the speed of zero, time and location for this vehicle is recorded.

Graphical presentation of the stopping waves traveling backward along the lines of vehicles are shown in Figure 80 for the first 50 cars and in Figure 81 for all 200 cars in the section. From these graphs one can find where the stopping wave will reach a certain vehicle.
**Figure 80:** Location of stopping for first 50 cars. The location the stopping waves effect the first 50 cars (ID) when the lead car decelerates at a rate of 8 and 16 ft/sec/sec (ratecodes 2 to 4, respectively). Volume is 1800, unit is ft.
Figure 81: Location of stopping for all cars. The location the stopping waves effect the following cars(ID) when the lead car decelerates at a rate of 8 and 16 ft/sec/sec (ratecodes 2 to 4, respectively). Volume is 1800, units is ft.
Relationships between the stopping location and the number of vehicles are computed using regression techniques. The equations for 8 and 16 ft/sec/sec deceleration rates are:

\[ D = 442.3 - 24.8723N \]
\[ D = 238.7 - 24.8583N \]
respectively. Note that \( D \) is measured from the point the first vehicle starts to decelerate. The graph shows the location from the beginning of the section. The first vehicle started deceleration when it was at 36382.6 ft. from the beginning of the section at 451 seconds. The R-squared values for these equations are 0.9999 and 0.9999.

The slope of these lines show the spacing of vehicles when they stop. The spacing in both cases are approximately 25 feet which is very reasonable. At 16 ft/sec/sec deceleration rate the slope is 0.014 less than the slope at 8 ft/sec/sec. This difference is practically negligible because it is only 1.4 feet in 100 cars.

Propagation of the stopping waves in time down the line of vehicles are shown in Figures 82 for the first 50 cars and in Figure 83 for all 200 cars. From these graphs one can find when a stopping wave will stop a certain vehicle, or at a given time how many vehicles are stopped.
Figure 82: Time of stopping for first 50 cars. The time the stopping waves effect the first 50 cars (ID) when the lead car decelerates at a rate of 8 and 16 ft/sec/sec (ratecodes 2 to 4, respectively). Volume is 1800, unit is seconds.
Figure 83: Time of stopping for all cars. The time the stopping waves effect the following cars(ID) when the lead car decelerates at a rate of 8 and 16 ft/sec/sec (ratecodes 2 to 4, respectively). Volume is 1800, unit is seconds.
The relationships between the time and the number of vehicles stopped are obtained by fitting a line through the points with the least square errors method. The linear equations for 8 and 16 ft/sec/sec deceleration rates are as follows:

\[ T = 15.04 + 1.7143N \]
\[ T = 12.33 + 1.7159N. \]

The R-squared values for these equations are 0.9998 and 0.9997, respectively. The y-intercepts of these lines are showing the times after the first car starts deceleration which is at 451 second.

The following vehicle will stop 1.71 seconds after its leader had stopped, according to these equations. The slopes of the lines are almost equal and the difference is negligible. It can be stated that regardless of the ratio of deceleration of the lead car, the following cars will stop at a rate of one car in every 1.71 seconds. The rate would effect the time the stoppage would reach the vehicle. The difference in propagation time of an 8 ft/sec/sec and a 16 ft/sec/sec deceleration rate of the lead car is 2.71 seconds. This difference remains almost constant when the number of vehicles are changed.

For example, in lane one of a freeway the lead car decelerates at 8 ft/sec/sec to a complete stop. In the
other lane with the same exact traffic the lead car uses a rate of 16 ft/sec/sec and starts deceleration at the same time the lead car in lane 1 does. The 100th car in lane one and lane two are stopped at 186.47 sec. and 183.92 sec., respectively. The difference in propagation time for the first 100 cars is 2.55 seconds.

The propagation of the stopping waves versus time is presented in Figure 84 for the first 50 cars and in Figure 85 for all 200 cars in the system. The lead vehicle started deceleration at 451 seconds. At the rate of 16 ft/sec/sec the stopping wave reaches a point sooner than the 8 ft/sec/sec. This time difference is almost constant and does not change down the line of vehicles.

Figures 84 and 85 show propagation of stopping waves along the line of vehicles as it actually happens in the simulation model. The rates of propagation at the two different rates are not much different. The stopping wave reaches the same location after a certain time interval. At a given time the waves are almost a constant distance apart.

The equations of these stopping waves are:

\[ D = 674.17 - 14.5041T \]
\[ D = 430.83 - 14.4818T. \]
Figure 84: Propagation of stopping wave for first 50 cars. Propagation of the stopping waves down the first 50 cars when the lead car decelerates at a rate of 8 and 16 ft/sec/sec (ratecodes 2 and 4, respectively). Volume is 1800, units are ft and sec.
Figure 85: Propagation of stopping wave for all cars. Propagation of the stopping waves down the line of cars when the lead car decelerates at a rate of 8 and 16 \( \text{ft}/\text{sec/seg} \) (ratecodes 2 and 4, respectively). Volume is 1800, units are ft and sec.
The R-squared values for fitting of a linear model are 0.9996 and 0.9996 for 8 and 16 ft/sec/sec deceleration rate, respectively.

When all the data from the two deceleration rates are combined, a linear model with a slope of -14.4774 is obtained. Thus, the average stopping wave travels backward at a speed of 14.4774 ft/sec when traffic volume is about 1800 vph.

The slopes of the two lines are very close, but the y-intercepts are different. This indicates that when the lead car decelerates at a higher rate, the stopping wave propagates faster at the beginning and then at a fairly constant rate thereafter.

So far, the propagation of the slowdown and stopping waves at traffic volume of 1800 vph were discussed. In the following section the effect of traffic volume on propagation of slowdown and stopping waves will be discussed.

5.4 EFFECT OF TRAFFIC VOLUME ON WAVE PROPAGATION

Traffic volume affects the speed of propagation of slowdown and stopping waves. In the previous graphs the volume was about 1800 vph and the effects of the deceleration rates were investigated. Here, traffic volumes of 1200,
1500, and 1800 vph and only a deceleration rate of 8 ft/ sec/sec will be examined. The leader of a group of 100 cars all traveling at a speed of 80.67 fps, on a straight section of a single lane of a freeway, is asked to decelerate at a rate of 8 ft/sec/sec to a complete stop. The lead car starts deceleration at 361 seconds from the beginning when the car had traversed 29122.4 ft. on the section.

5.4.1 VOLUME EFFECT ON PROPAGATION OF SLOWDOWN WAVES

The propagation of slowdown waves at three different volume levels are shown in Figures 86-88. At all three levels the initial rapid propagation can be seen in all graphs. From Figure 86 it can be seen that after initial rapid propagation the graphs are not diverging, but following similar fluctuation patterns. This graph shows that there are differences in the number of vehicles affected at a given location, however, these differences are not increasing by involving more cars. Furthermore, the graph shows that at higher volume the slowdown wave propagates deeper into the line of vehicles.

Linear models representing the location of a slowdown wave versus the number of the vehicles were fitted for the points on these graphs. The equations for 1200, 1500, and 1800 vph are:

\[ D = -51.67 - 25.1747N \]
\[ D = -138.06 - 25.1271N \]
Figure 86: Location of slowdown waves at different volumes. Effect of traffic volume on the location the slowdown waves reach the vehicles. Unit is ft.
Figure 87: Time of slowdown waves at different volumes. Effect of traffic volume on the time the slowdown waves reach the vehicles. Unit is seconds.
Figure 88: Propagation of slowdown waves at different volumes. Effect of traffic volume on propagation of the slowdown waves. Units are foot and second.
\[ D = -257.58 - 25.5031N \]

and R-squared values are 0.9963, 0.9943, and 0.9903, respectively. As in the previous section these equations should not be used for several vehicles immediately following the first car. The slopes are close, but the y-intercepts of the lines are different.

The relationship between the time and the number of vehicles affected is shown in Figure 87 for the three volume levels. From this graph, one can find how long it would take for a slowdown wave to reach a certain vehicle. Furthermore, the number of vehicles influenced by a known duration of road blockage can easily be read off the graph.

The linear models for 1200, 1500, and 1800 vph are as follows:

\[ T = -9.65 + 2.7491N \]
\[ T = -8.21 + 2.1320N \]
\[ T = -7.99 + 1.7172N \]

The R-squared values are 0.9991, 0.9991, and 0.9988, respectively. Once again, these equations should not be used for the car immediately following the first car. The negative y-intercepts are due to using regression analysis and the difference in propagation at the beginning and later on. The slope of these lines show that one vehicle will slow down every 2.75, 2.13, and 1.71 sec. when traffic volume is 1200, 1500, and 1800, respectively.
The slope of propagation of the slowdown waves at different volume levels are shown in Figure 88. The slope at a higher volume is higher than at a lower volume. The slowdown wave propagates faster when the traffic volume is increased. From this graph one can find when a deceleration wave will reach a certain location on the road. Likewise, the location of the slowdown wave at a given time can easily be found from the graph or from the equations representing the graph.

Three linear equations each representing a certain volume level are calculated using the least square method. The equation for volumes of 1200, 1500, and 1800 are:

\[ D = -132.58 - 9.1445T \]
\[ D = -227.15 - 11.7605T \]
\[ D = -365.97 - 14.7935T \]

and R-squared values are 0.9943, 0.9909, and 0.9837, respectively. Like the other equations in this chapter, these equations should not be used while the propagation is influenced by the initial deceleration. The duration of initial deceleration is usually less than 20 seconds.

A slowdown wave propagates faster at higher volumes than at lower volumes. For example, at a volume of 1200 vph a slowdown wave propagates backward 1778.6 ft. in three minutes, while at a volume of 1800 vph the propagation is
3028.8 ft. From these equations one can find the extent of a disturbance as a function of time for a given volume level.

5.4.2 VOLUME EFFECT ON PROPAGATION OF STOPPING WAVES

The stopping waves propagation at volume levels of 1200, 1500, and 1800 vph are represented in Figures 89-91. In Figure 89 the lines are very close to each other as it is expected. The small difference is due to unequal spacings the vehicles will have when they come to a standstill position. The relationship between the location and the number of vehicles stopped are computed using regression techniques.

The equations representing propagation of a stopping wave down the line of vehicles at 1200, 1500, and 1800 vph are as follows:

\[ D = 433.19 - 24.9721N \]
\[ D = 435.39 - 24.9135N \]
\[ D = 434.52 - 24.8181N. \]

The R-squared values for the lines are 0.9999, 0.9999, and 0.9999. The slopes of these lines show the spacing between vehicles when they stop. At a volume of 1800 vph the spacing is 0.15 feet less than at 1200 vph.
Figure 89: Location of stopping waves at different volumes. Effect of traffic volume on the location the stopping waves reach the vehicles. Unit is ft.
Figure 90: Time of stopping waves at different volumes. Effect of traffic volume on the time the stopping waves reach the vehicles. Unit is seconds.
Figure 91: Propagation of stopping waves at different volumes. Effect of traffic volume on propagation of the stopping waves. Units are feet and seconds.
The time a stopping wave makes a vehicle to stop is shown in Figure 90. The equation of these lines are computed as:

\[ T = 6.86 + 2.7460N \]
\[ T = 10.95 + 2.1222N \]
\[ T = 15.14 + 1.7058N \]

with R-squared values of 0.9994, 0.9996, and 0.9991, respectively.

Using these equations one can find how many vehicles are stopped when the road is blocked for a certain time period; or to find how long it would take to stop a certain number of vehicles due to the road blockage. The restriction on not using these equations for the vehicles in front of the platoon should be reminded here.

The relationship between the location of a stopping wave and the time it reaches that point is presented in Figure 91. The slope of the line shows how fast the stopping wave propagates backward down the line of vehicles. As it is expected, at higher volumes the slope is steeper than at lower volumes. The equation of the lines for 1200, 1500, and 1800 vph are:

\[ D = 494.77 - 9.0887T \]
\[ D = 563.52 - 11.7355T \]
\[ D = 653.48 - 14.5303T \]
The distance, $D$, is measured from the point the lead car starts deceleration. The R-Squared values are 0.9995, 0.9997, and 0.9991, respectively.

The relationships can be used to find how far the congestion has propagated at a given time. Likewise, one can use these equations to find when the traffic jam will reach a certain point on the road.

In the previous sections situations where the lead car came to a standstill position and remained stopped until the end of the simulation time were studied. These represented situations where the lane is blocked for a relatively longer period of time. However, in the following section propagation and dissipation of kinematic waves when the road is blocked by an incident for a short period of time will be discussed.

5.5 EFFECT OF AN INCIDENT ON WAVE PROPAGATION AND DISSIPATION

Four different kinematic traffic waves are identified as follows: slowdown wave, stopping wave, moving(starting) wave, and recovering wave. A slow down wave is evidenced by a speed reduction imposed on the following car due to the lead car deceleration (all vehicles were moving at the desired speed before they decelerate). A stopping wave is
identified by the location and the time the vehicles reach a speed of zero (not all vehicles are stopped). The stopped vehicles will move only after the predecessor moves. The point where a stopped vehicle accelerates moves upstream and this movement identifies the propagation of a starting (moving) wave. The propagation of this moving wave causes the front of the queue to shrink. The recovering wave is determined by finding the points where a vehicle reaches the desired speed after an earlier deceleration imposed by its lead car.

The effect of traffic volume and the deceleration rate of the lead car on propagation of slowdown and stopping waves were discussed in the previous sections. Here, only at the volume of 1200 vph and lead car deceleration rate of 8 ft/sec/sec, the propagation and dissipation of kinematic waves caused by a three-minute blockage of the roadway will be analyzed.

Due to an incident the road is blocked for three minutes and arriving cars are queued beyond that section. Afterward the vehicles are allowed to accelerate and reach a desired speed of 55 mph. While the cars in front of the queue accelerate to reach the desired speed and make a reduction in the number of vehicles in the queue, the arriving vehicles join the rear of the queue and increase
the number of vehicles in the platoon. Through this process the location of the bottleneck moves upstream. This bottleneck of stopped cars will disperse only when the moving wave propagates faster than the stopping wave, or the propagation of the stopping wave is stopped.

For this experiment vehicles are generated from an arrival rate of 1200 vph. All vehicles have a desired speed of 80.67 fps, and this speed is assigned to them upon generation. The lead car starts deceleration after traveling 14600.55 feet which takes place at 180 seconds. There are enough cars in the system that the slowdown wave or the stopping wave does not reach the last vehicle. The lead car decelerates to a complete stop and remains in a stall situation until 360 seconds (about 3 minutes), and then accelerate to the desired speed. There is no restriction on the number of arriving vehicles. The time and the location a vehicle slows down, stops, moves again, and reaches the desired speed is determined for all vehicles.

Five independent replications each representing a different group of drivers are made and the average values are computed for each vehicle. Figures 92-94 show the relationship between the time, identification of a vehicle, and the location of each wave. The index values of 0, 1, 2, and 3 represent stopping wave, slowing wave, moving (starting) wave and recovering wave respectively.
Figure 92: Road blockage effect on location of waves. The effect of a 3 minutes road blockage on the time (sec) a slowdown wave (index=1), a stopping wave (index=0), a starting wave (index=2), and a recovering wave (index=3) reach a certain vehicle.
Figure 93: Road blockage effect on time of waves. The effect of a 3 minutes road blockage on the location (feet) a slowdown wave (index=1), a stopping wave (index=0), a starting wave (index=2), and a recovering wave (index=3) reach a certain vehicle.
Figure 94: Road blockage effect on propagation of waves. The effect of a 3 minutes road blockage on propagation of a slowdown wave (index=1), a stopping wave (index=0), a starting wave (index=2), and a recovering wave (index=3). Units are feet and ft/sec.
At the point a starting wave reaches a stopping wave the queue of stopped cars is eliminated. Figure 92 shows that the distance between these two waves decreases and finally they merge. The slope of the starting wave is less than the slope of the stopping wave. This indicates that more vehicles will move than stop in a certain period of time, and the queue of the stopped vehicles will dissipate. When the slope of the moving wave is equal or greater than the stopping wave, there will always be a queue of stopped vehicles.

To eliminate the queue of stopped cars fast, an arrangement should be made such that these two waves intersect at an earlier point. There are two possibilities: one is to minimize the starting delay by moving as many vehicles as possible from a standstill position in a given time. There is not much one can do with this option because the drivers themselves want to get out of the traffic jam quickly anyway. The second option is to reduce the number of vehicles joining the end of the queue. This option will be discussed in detail in the next section. The sooner these two lines merge the faster the queue of stopped cars dissipates. However, when the queue of the stopped cars is dispersed the effect of the road blockage has not been totally eliminated yet.
The recovering wave ought to reach the last car affected by the slow down wave. It should be mentioned that the lines representing the slowdown wave and the recovering wave will not intersect, because the slowdown and recovery will not happen at the same time. The recovering wave should affect more vehicles than the slowdown wave reaches at a given period of time. Otherwise, the queue of the slow moving vehicle will not dissipate.

The slope of the line representing the recovering wave is almost equal to the slope of the starting wave. A similar relationship exists between the slowdown wave and the stopping wave. The slopes of these lines are computed using the least square technique.

The equations of the lines representing the slowdown wave, stopping wave, moving wave, and the recovering wave are respectively as follows:

\[ T = 179.08 + 2.65388N \]
\[ T = 190.69 + 2.70625N \]
\[ T = 357.37 + 1.83815N \]
\[ T = 371.92 + 1.84478N. \]

The R-squared values are 0.9989, 0.9998, 0.9999, and 0.9998, respectively. Where \( N \) is the number of vehicles and the y-intercepts are showing the time a certain wave starts to affect the cars. The lead car starts to deceler-
ate to a complete stop at 180 seconds and starts to move again at 360 seconds.

The equations for the starting wave and the recovering wave can be used for various road blockage durations. They can also be used for different volumes. However, the equations for the slowdown and stopping waves are exclusive for this volume level and cannot be used in different situations. For a different duration of the road blockage one has to simply shift the recovering and the starting waves to the proper starting positions.

From Figure 92 one can find how many vehicles are affected or stopped at a given time after the blockage occurred. Likewise, the time a certain vehicle will be affected by any of the waves can be determined from this graph.

When the road is blocked for a longer period of time, the equations can be used to find when the queue of the stopped cars will be dispersed or when the recovering will take place.

Figure 93 illustrates the location a wave will reach a certain vehicle. The stopping wave and moving wave affect the vehicle at the same location, therefore, the lines representing these two waves coincide. The slowdown and
re Recovering waves move forward instead of moving backward when they reach the last affected vehicles. This phenomenon is shown by different patterns at the end of the lines representing these waves.

One can use Figure 93 to find the location a vehicle will go through the disturbance or will recover. Likewise, at a given location the number of vehicles affected by each wave can be determined.

The equations expressing the relationships between the location a vehicle is affected and the number of vehicles for slowdown, stopping, starting, and recovering waves are as follows:

\[
D = 14549.18 - 25.03356N
\]
\[
D = 15029.94 - 24.84350N
\]
\[
D = 15029.94 - 24.84360N
\]
\[
D = 15566.01 - 22.88485N
\]

The R-squared values are 0.9978, 0.9999, 0.9999, and 0.9832 respectively. The equation for the stopping and starting waves is the same as expected.

To find the location of a wave at a given time one may use Figure 94. This graph shows how the waves propagate upstream (backward), and when and where they will dissipate. It illustrates that the moving wave which started about 3 minutes later, travels faster than the stopping
wave and makes all the stopped vehicles accelerate to reach the desired speed.

Moreover, it can be seen that the distance between the stopping wave and the moving wave, also between the slowdown wave and the recovering wave is decreasing over time. A complete recovery will occur sometime after the moving wave reaches the stopping wave. The sooner these two coincide the faster the recovery will take place. When the distance between the corresponding wave lines is decreasing, it is an indication of the dispersion of the queue and finally a total elimination of the blockage effect. However, if the lines do not tend to merge, the effect of the blockage will not be eliminated.

The relationship between the distance and the time for a slowdown wave, a stopping wave, a starting wave, and a recovering wave obtained from regression analysis are as follows:

\[
D = 16225.34 - 9.40595T
\]

\[
D = 16776.21 - 9.17101T
\]

\[
D = 19859.31 - 13.51437T
\]

\[
D = 20167.73 - 12.38477T
\]

The R-squared values are: 0.9991, 0.9929, 0.9999, and 0.9802 respectively. The steeper slopes of the lines representing the starting wave and the recovering wave rela-
tive to the other two indicate that the bottleneck will be eliminated and the disturbance will stop growing.

The disturbance lasts even after the physical removal of the road blockage, and its duration depends on the arrival rate of the vehicles. The effect of the 3 minutes incident lasted about 10 minutes. When more vehicles arrive than depart the bottleneck will move backward but will not go away. To obtain a complete recovery the arrival rate ought to be controlled. This can be achieved by diverting some of the traffic to other routes or by closing or metering the ramp. Although these control strategies reduce the number of arriving cars, there are still a lot of mainline traffic approaching the site. A control strategy that would accelerate the recovery phase will be discussed in the next section, which controls the mainline input by delaying and reducing the arrival rate.

5.6 A CONTROL STRATEGY FOR BOTTLENECKS

In the previous section it was shown that the moving wave should reach the last halted vehicle in order to eliminate the queue. The sooner this happens, the faster the queue of the stopped cars is dispersed. It was also discussed that we do not have much control over the rate that the moving wave reaches the vehicles. On the other hand,
the stopping wave and its propagation can be controlled. In this section a control strategy which guides mainline traffic before and through a bottleneck will be compared to a not-guiding strategy.

The bottleneck is caused by an incident that blocks the road for about three minutes. During this time vehicles arrive at the scene and queue. The road is cleared after three minutes and the stopped vehicles accelerate to their desired speed of 55 mph (80.67 fps). All vehicles will travel at a speed of 55 mph when there is not a bottleneck.

The arrival rate of the vehicles to the system is 1200 vph, all with a desired speed of 80.67 ft/sec. Vehicles will reach the bottleneck after traveling for a while in the system. The first vehicle reaches the bottleneck after 180 seconds from the start of the simulation. At this time it decelerates and stops until the time is 360 seconds, then it accelerates to the desired speed of 80.67. The following vehicles adjust their speed and location relative to the situation of the car in front of them. Some vehicles are required to stop, and others to decelerate.

A control strategy which guides the upcoming mainline traffic through the bottleneck will be compared with the traditional non-guiding strategy. Customerly, the traffic approaching the bottleneck is allowed to reach the queue of
the stopped vehicles and progress after the predecessor has moved. This approach is basically leaving the traffic alone and allowing it to suffer and recover by itself.

A procedure is suggested which guides the vehicles before reaching the bottleneck and through the bottleneck to alleviate the congestion in an efficient manner. According to this strategy, the arrival of the vehicles to join the queue of stopped cars are delayed by reducing the speed of upstream traffic. The vehicles are asked to slow down before they reach the actual bottleneck. Delaying the arrival would help to dissipate the queue of stopped vehicles faster, as a result the road will be cleared sooner.

This simulation model is used to investigate and compare the guiding and non-guiding strategies on a 3 mile section of a single lane traffic. The study section is straight with no entrance or exit. The maximum number of vehicles affected by the incident, found from the preliminary independent replications of the simulation model, was 231 cars. In all of the simulation runs 235 cars are generated and the propagation of the slowdown wave, the stopping wave, the starting wave, and the recovering wave are investigated. There are always more cars in the system than the number of affected cars.
Each control strategy is repeated five times, independently; and the average values of these five replications are used. The control policy is the same in all five runs. The simulation time is long enough such that all vehicles entering the system would travel till the end of the section. The program is run for 1200 seconds.

The simulation model is designed such that the user can specify the location of the speed reduction signs and the speed posted on them. Furthermore, one can specify how long this operational policy is used within a single simulation run.

In this experiment all vehicles in the system are asked to slowdown at 180 sec. from the beginning of the simulation till 360 seconds; which is the time the incident is cleared. During the 3 minute period, the vehicles are asked to travel at a speed of 51.33 fps (35 mph) rather than 80.67 fps (55 mph). The road is totally blocked during this period of time and the lead car has stopped. After 360 seconds has elapsed the lead car, is allowed to accelerate to the desired speed of 80.67 fps.

The location of the speed reduction signs are 0.5, 1.0, 1.5, 2, and 2.5 miles from the beginning. The number of the speed reduction signs and the suggested speed on each sign can easily be changed by the traffic controller. The
desired speed of the vehicles when time is less than 180 seconds is 80.67 fps, and between 180 seconds and 360 seconds the speed is 51.33 fps. The posted speed of 51.33 changes to 80.67 fps at different times for different locations. The change for the first station which is located 0.5 mile from the beginning of the section happens at time 540 seconds. For the second, third, fourth, and the fifth signs the times are 540, 540, 480, and 360 seconds, respectively.

For comparison, one time a simulation run is made when there was no guidance and another time when the traffic was guided. The average values for both strategies are computed and compared.

The criteria for comparisons are: the duration of the slowdown, the duration of stoppage, the time vehicles spend in the system, delay caused due to the congestion or slowdown, the number of vehicles affected by the incident, the overall speed, density, and throughput. The summary of the results are given in Table 9. and Figures 95-97.

Figures 95-97 represent the situation where the traffic is guided through the section, and Figures 90 - 92 the same situation when the traffic is not guided. It can be seen that the duration of the slowdown is reduced from 617 seconds to 476 seconds, and the duration of stoppage from 565 to 426 seconds.
Figure 95: Guiding effect on time propagation of waves. The effect of guiding traffic on the time (sec) a slowdown wave (index = 1), a stopping wave (index = 0), a starting wave (index = 2), and a recovering wave (index = 3) reach a certain vehicle.
Figure 96: Guiding effect on location propagation of waves. The effect of guiding traffic on the location (feet) a slowdown wave (index=1), a stopping wave (index=0), a starting wave (index=2), and a recovering wave (index=3) reach a certain vehicle.
Figure 97: Guiding effect on propagation of waves. The effect of guiding traffic on propagation of a slowdown wave (index=1), a stopping wave (index=0), a starting wave (index=2), and a recovering wave (index=3). Units are foot and ft/sec.
Table 9

Guiding Vs Not-guiding Control Strategies


<table>
<thead>
<tr>
<th>evaluation criteria</th>
<th>not-guided</th>
<th>guided</th>
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<tbody>
<tr>
<td>Max. no. of vehicles slowed down</td>
<td>231</td>
<td>187</td>
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<td>Average no. of vehicles slowed down</td>
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<td>180.2</td>
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<tr>
<td>Max. no. of vehicles stopped</td>
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<td>133</td>
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<tr>
<td>Average no. of vehicles stopped</td>
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<td>114.0</td>
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<tr>
<td>time the last stopped car moved</td>
<td>745</td>
<td>606</td>
</tr>
<tr>
<td>time the last car recoverd</td>
<td>797</td>
<td>656</td>
</tr>
<tr>
<td>time vehicles are in system (sec.)</td>
<td>64788.3</td>
<td>64082.5</td>
</tr>
<tr>
<td>delay to vehicles (sec.)</td>
<td>18644.4</td>
<td>17938.9</td>
</tr>
<tr>
<td>overall speed (fps)</td>
<td>60.32</td>
<td>60.85</td>
</tr>
<tr>
<td>overall throughput</td>
<td>110014.3</td>
<td>111738.5</td>
</tr>
</tbody>
</table>

When the traffic is guided not only the duration of the slowdown and stoppage decrease, but also the number of the vehicles affected by the disturbance is reduced. The maximum number of vehicles affected when traffic is not guided is 231, while it is guided the number is 187; the average numbers are 197.6 and 180.2, respectively. There is a drastic decrease in the number of stopped cars. The maxi-
number reduced from 209 when traffic is not guided to 133 when traffic is asked to slow down before reaching the bottleneck. The average number of stopped cars reduced from 186.2 to 114.0.

The average time a vehicle spends traveling through the section is reduced. This is in spite of a deliberate delay that is caused to upstream traffic by asking them to slow down. The average total time all vehicles spend in the system when traffic is not guided is 64788.3, but when the traffic is guided this time is reduced to 64082.5 seconds. This implies that the traffic reached the destination faster by slowing down before reaching the bottleneck.

More important than the time saved for the mainline traffic is the fact that the congestion is cleared earlier and the operation on the road is back to normal faster. When the effect of the kinematic disturbance is cleared earlier, the road can be used with full capacity. This important effect is obvious when the mainline traffic guidance is used in conjunction with ramp metering. Consider a situation where traffic volume is 1600 and by ramp closure it is reduced to 1200 which is the mainline through traffic. The effect of the 3 minute road blockage would last until the time is 797 seconds. However, when it is guided the effect lasts only until 656 seconds. The difference is
the elimination of the disturbance effect which occurs 141 seconds faster.

The delay caused to the motorist when the traffic is allowed to reach the bottleneck and recover without any guidance is 18644.4 seconds for all vehicles. On the other hand, total delay to the traffic by reducing the speed limit ahead of congestion is 17938.9 seconds.

The overall speed of traffic when it is not guided is 60.32 fps, but when it is guided the value is 60.85. It is important to realize that the traffic reaches the desired speed much earlier when it is guided than when it is not guided, Figure 98 illustrates this point very well. The speed is computed for all vehicles in every second and the average of these speeds is found at that particular time. The overall speed is the mean of these average speeds for all 5 runs.

The average speed by itself is not enough criteria for evaluation of traffic condition. Therefore, the throughput of traffic which is a product of volume and speed is also computed. The throughput is calculated every second and the average of these values are obtained, Figure 99 shows throughput versus time. The overall throughput is the mean of the average throughput over all runs. The overall throughput when traffic is
Figure 98: Guiding effect on speed of traffic. Effect of guiding traffic (index=1) versus not-guiding (index=0) on average speed. Units are ft/sec and seconds.
Figure 99: Guiding effect on throughput of traffic. Effect of guiding traffic (index=1) versus not-guiding (index=0) on average throughput. Units are ft/sec and seconds.
not guided is 110014.3 and it is 111738.5 when the traffic is guided.

Density of traffic in the section is computed by finding the distance between the first and the last car in the section and dividing it by the number of vehicles in the system. In Figure 100, the density computed every second is plotted versus the average speed for that particular time. There is an obvious loop when the traffic is guided as well as when it is not guided. When the traffic is guided the loop is larger then when it is not guided.

From comparison of the results for the two different operational policies, one can clearly see the advantages of guiding traffic over not guiding it. This "crude" experiment showed that by controlling the upstream traffic the congestion is alleviated faster and flow is improved. However, the results obtained here should not be generalized. Further research on optimization of this procedure and its feasibility are necessary.
Figure 100: Guiding effect on density of traffic. Effect of guiding traffic (index=1) versus not-guiding (index=0) on speed-density relationship. Units are ft/sec and vpm.
A car following model, CARSIM, is developed and tested under various traffic conditions. CARSIM can be used for freeway and street traffic under normal as well as stop-and-go conditions. It can simulate situations ranging from free flow to jam density. Furthermore, the dual behavior of traffic is taken into consideration and the user has the option to use CARSIM as a single regime or as a dual regime model. Moreover, variable reaction times are used for different drivers, and the start-up delay of the drivers are taken into account. In its present form the model uses passenger cars and trucks; other types of vehicles can easily be added.

CARSIM provides a safe following distance for all vehicles. The speed and the location of each vehicle is updated such that the non-collision constraint is always satisfied. In the mean time, the computed acceleration/deceleration values are within the boundaries of acceptable numbers.

The features of CARSIM can be summarized as:
1. It provides a marginally safe following distance for all vehicles.

2. It assigns different reaction times to drivers.

3. It can handle normal as well as stop-and-go traffic operations.

4. It takes into consideration the start-up delay of stopped vehicles.

5. It uses different reaction times for a driver in congested and non-congested traffic.

6. It reproduces the effects of severe kinematic disturbances.

7. It can be used as a single or dual regime traffic simulation model.

In validation of CARSIM it was observed that when a platoon is not in the steady-state condition, the average time headways and volumes computed by two different methods are not the same. There is a considerable difference when a platoon is in an acceleration or deceleration phase; but in the steady state condition the difference is not noticeable. The maximum difference occurs at the beginning of the acceleration phase.

The concept of using different maximum deceleration rates is introduced. Using two different maximum deceleration rates, one for before the disturbance and another for
after the disturbance provided trajectory plots very close to those obtained from the field data. This also reflected the slower reaction of drivers to acceleration than deceleration when going through a kinematic disturbance. The notion of making the maximum deceleration rate as a function of density is merely for computational purpose. Using a single deceleration rate for all densities provided a good fit for either before the disturbance or after the disturbance, but, not for both cases.

When a disturbance forces the vehicles to stop, a 2 second start-up delay for all drivers; or 1 second for the drivers with a very short reaction time and 2 seconds for the majority of them is suggested. The buffer space for vehicles within the disturbance seems to be less than the value used in other cases.

The validation of this model is performed at microscopic and macroscopic levels. For microscopic validation the trajectory and speed change patterns of four different platoons of vehicles covering a wide range of densities (free flow to jam density) are compared to the simulation results. Speed change patterns obtained from the simulation models replicate the real world situation very realistically. Trajectory plots generated by the simulation models for before a severe kinematic disturbance and after
that are very close to the plots from the field data. The simulation plots reproduce situations where vehicles are forced to stop for a while and then start moving again.

For macroscopic validation the overall performance of the simulation model is compared with that of three different platoons. The variation of flow parameters, the fundamental relations of traffic flow, relationships between simulation results and field data, and the effects of stochastic input values on response variables are investigated at the macroscopic level. Three flow parameters: speed, density, and volume for the platoons are computed and compared with the average values obtained from simulation models. The simulation model replicates very well the behavior of platoons going from a speed of about 80 fps to a speed of zero in less than one minute. The time a simulated platoon reaches the jam density is very close to that of the actual platoon. The platoon volume which is the product of speed and density is not recommended to be used for comparison purposes, especially when there is a breakdown of the flow.

After extensive validation, the model is used to study the propagation of traffic waves and the effects of mainline traffic control on congestion alleviation. The effects of the lead car's deceleration rates on the propa-
gation of the slow down and stopping waves are pronounced for the first group of 10 vehicles, but it becomes less important after the propagation reaches more vehicles. A set of equations and graphs are developed to find the queue length, speed of wave propagation, location of a wave, time, and the number of vehicles affected by the waves. The slopes of propagation of the slowdown waves are approximately -25 ft/vehicle, 1.73 sec/vehicle, and -14 ft/sec; and that of the stopping waves are approximately -24.9 ft/vehicle, 1.71 sec/vehicle, and -14.5 ft/sec when traffic volume is 1800 vph.

The effects of traffic volume on the slowdown and the stopping waves propagation are investigated at volume levels of 1200, 1500, and 1800 vph. At all three levels the initial rapid propagation is observed. The graphs and equations to present the relationship between the time, location, and the number of affected vehicles are given at different volume levels. The rates the vehicles will be affected by the slowdown waves are 2.75, 2.13, and 1.71 sec/vehicle at 1200, 1500, and 1800 volume levels, respectively. The speed of propagation of the slowdown waves are -9.1, -11.8, and -14.8 ft/sec; and that of the stopping waves are -9.1, -11.74, -14.53 at traffic volumes of 1200, 1500, and 1800 vph, respectively.
The effects of a road blockage on the propagation and dissipation of a slowdown, stopping, starting, and recovering waves are investigated at a traffic volume of 1200 vph. The equation and the graphs for the starting wave and the recovering wave can be used at different volume levels, or road blockage duration. However, those for the slowdown wave and the stopping wave is only for this particular condition. The slope of the slowdown, stopping, starting, and recovering waves are -9.41, -9.17, -13.51, and -12.38 ft/sec., respectively. At the point a starting wave reaches a stopping wave, the queue of stopped cars is dispersed. The recovering wave and the slowdown wave will not intersect but may get closer.

The disturbance may last even after the physical removal of the blockage. The effect of the 3 minute incident lasted about 10 minutes, because the arriving cars joined the end of the queue and physically blocked the road.

A control strategy which reduces the number of vehicles joining the rear of queue is suggested. The attempt is to make the recovering wave reach the stopping wave sooner. This can be done by delaying the arrival of upcoming mainline traffic to the bottleneck. A traffic management policy which guides the vehicles before reaching the bottleneck is compared with the common practice of allowing traffic to recover by itself (not-guiding).
The duration of the slowdown is reduced from 617 sec. to 476 sec., and the stoppage from 565 sec to 426 sec. The average number of vehicles affected by the incidents are reduced from 197.6 to 180.2. The average number of stopped vehicles decreased from 186.2 to 114.0. The average time a vehicle spent to travel the three mile section is reduced in spite of a deliberate delay that is caused to upstream traffic by asking them to slow down. Thus, under some conditions it seems that the traffic can reach its destination earlier by slowing down before reaching the bottleneck.

More important than the time saved for the mainline traffic is the fact that, the congestion is cleared earlier and the operation returns back to normal conditions faster. When the traffic was guided the congestion is eliminated 141 seconds faster. The delay caused to motorist when they were guided is less then when they were not. There is also some improvement on the overall speed and the throughput of traffic. The results from this control strategy should not be generalized. More research on the feasibility of it and its optimization is needed.
Appendix A

REVIEW OF CAR FOLLOWING STUDIES

A.1 INTRODUCTION

Understanding the interaction of vehicles when following each other and realistic transfer of this behavior into mathematical expressions is a crucial part of a microscopic simulation model. Most of the earlier models used some form of

\[ S = \text{constant} + U\cdot t + \frac{U^2}{2} \]

for the average spacing of a lead vehicle and the following car. This model is for uniform velocity for each vehicle in the stream, and provides no insight into the behavior of a line of traffic when one of the vehicles in the platoon accelerates or decelerates and the following vehicle attempts to maintain a desired spacing (Gerlough & Huber, 1975).

Early car following models were developed in the mid 1950's; since then there has been a lot of studies on this subject. Car following studies are an attempt to under-
stand the behavior of driver–vehicle units when they follow another vehicle on the road, in order to provide and maintain an efficient and safe operation for all users of the roadway system. Some car following models can be represented in the form of the following equation:

\[ \text{Response} (t+T) = \text{Sensitivity} \times \text{Stimulus}(t). \]

Basically, there are two types of car following models: linear and non-linear.

In linear car following models the response, acceleration or deceleration is directly proportional to the stimulus since the sensitivity is a constant. In most of the linear car following models the stimulus is the differential speed of the follower and the leader, this implies that in linear car following the response is independent of spacing between the two vehicles.

Non-linear car following models take into account the spacing between the following and the leading vehicles. The response not only is a function of differential velocity, but also a function of spacing of the two cars.

In the following three sections, some car following models and studies related to car following behavior of traffic flow in bottlenecks will be reviewed in a chronological order. The purpose is not only to comprehend the subject of car following better, but also to use the findings
in developing a car following model that would handle traffic flow from a free flow condition to the stop-and-go situation in a more realistic manner. It must be confessed that this review is not a complete summary of previous research. It contains most of the studies on freeway traffic flow that employed the microscopic approach. The intention is to build up a solid base for development of a car following model that benefits from valuable findings already reported and contains fewer assumptions.

A.2 STUDIES BEFORE 1965

Chandler, Herman, and Montroll (1958) studied car following and asymptotic stability of it by using a simple linear car following model.

\[
\frac{dU_n(t)}{dt} = \frac{\Lambda}{M} [U_{n-1}(t-T) - U_n(t-T)]
\]

Where Lamda is the sensitivity constant and it is the same for acceleration and deceleration mode. It was assumed that the vehicles had the same mass, M, so the ratio of Lamda over M is a constant. They assumed that the acceleration of the following car is the sum of 2 functions. One function for velocity differences, and another function for space headways. Regression analysis showed that the second function did not contribute significantly to the correlation coefficient so it was dropped. Note that when Lamda is
assumed constant the Q-K curve is a line instead of a parabola.

Herman, Montroll, Potts, and Rothery (1959) used Chandler's et al (1958) model and studied local and asymptotic stability of car following on a long stretch of highway with no passing allowed. They proved that constant spacing control is always unstable, but that relative velocity control is stable.

Greenberg (1959) assumed that traffic behaves like a continuous fluid and used the continuity equation to derive his logarithmic speed-density model. Traffic flow for a bottleneck section is fluid flow and can be analyzed by this model. However, care must be taken when applying the analysis to a bottleneck-controlled section. Behind the bottleneck the number of vehicles that enter the section is limited by the bottleneck and the stream is often non-fluid; an alternative type of analysis is necessary, the author stated.

Gazis, Herman, and Potts (1959) demonstrated that a follow-the-leader theory with a sensitivity inversely proportional to the distance of separation gave a law of vehicle interaction which leads to the Greenberg model. The car following model is in the following form

\[ x_{n+1}(t) = \lambda \left( \frac{x_n(t-T) - x_{n+1}(t-T)}{x_n(t-T) - x_{n+1}(t-T)} \right) \]
They supported the validity of this model by using experimental data.

Greenberg & Daou (1960) in an operational study of bottlenecks and shock waves in traffic flow in the The Holland Tunnel in New York used this model. The bottleneck was located at the onset of upgrade in the tunnel. The flow at bottleneck determined the flow rate for the entire tunnel. Queue was observed at the bottleneck and the end of queue represented a shock wave, path of a rapid slowdown, traveled back in the stream toward the entrance of the tunnel. The shock waves acted to limit the flow, the flow through the wave was much less than that at the physical bottleneck. After the shock wave passed the flow returned to a high level until another shock wave occurred. Thus, the shock waves allowed the tunnel to clear itself of congested vehicles on the exit side of the tunnel. The process started again.

To eliminate this accordion effect in the tunnel they introduced a procedure that prevented strong interaction between vehicles by controlling excessive volume. In order to prevent excessive volume they introduced gaps into the traffic stream at the entrance, causing the platooning on the traffic stream. When the number of vehicles entering the tunnel within one minute was greater than 21-23, traf-
fic was stopped for the remaining of the one minute period. On the average the traffic flow was increased by 80 vph/lane compared to the normal operation. The experiment showed that up to 1250 vph can pass through the bottleneck with platoons of 21-23 vehicles/min.

Edie & Foote (1960) in a study of the effect of shock waves on the Holland Tunnel traffic flow using optimum speed concept, created platoons of 10 vehicles and asked the drivers to drive at an average speed close to optimum. The lead car tried to maintain a constant speed of 20-25 mph. Traffic showed accordion-like action, with severe fluctuations in flow at more or less regular intervals when volume was high. The bottleneck was found to be at the foot of the upgrade. On regular peak traffic stream accordion-like action was observed as well. An overall increase in flow through the bottleneck was predicted if gaps were introduced into the traffic. An experiment was conducted by counting arrivals every 2 minutes, and when 44 vehicles arrived before the 2-minute interval expired, a police officer held traffic until the interval was up. This introduced a gap in the stream usually about 10 sec. The hourly volume was increased by approximately 6% (from 1176 to 1248 vph).
Helly (1961) developed a single-lane traffic flow simulation model for tunnel bottlenecks. The simulation results suggested that certain types of bottlenecks might be greatly alleviated by deliberately introducing gaps into the traffic stream. A theoretical model in the form of Markov process was advanced. The proposed model assumed that the drivers will seek to minimize both velocity difference and the difference between his actual headway and his desired headway. The combination of the two gave the final form of the model as:

$$\ddot{X}_n(t+R) = C_1(\dot{X}_{n-1}(t) - \dot{X}_n(t)) + C_2(X_{n-1}(t) - X_n(t) - D)$$

Where: $R$ is driver reaction time; $C_1$ and $C_2$ are velocity and headway control parameters; $C_1 > 0$, $C_2 > 0$; $D$ is desired headway for car $n$. Every driver was characterized by a maximum acceleration and a minimum deceleration, and a maximum and a minimum velocity.

Helly's study found that there were 2 basic types of bottlenecks. In bottleneck of the first kind the time headway does not depend on the position of the vehicles in the platoon. This condition was met whenever a driver chose his bottleneck headway independently of the behavior of the vehicle in front of him. However, in a bottleneck of the second kind the time headway of a driver depended on his position in the platoon. A driver reacted differently
if he was far back in a platoon than if he was close to the leader. His behavior depended on the bottleneck behavior of the vehicle in front of him.

The plot of standard deviations of time headway of a platoon in bottleneck versus the number of trucks (in an all truck platoon) showed an increase as the number of trucks increased. However, the increase was slower for small platoons. This supported the idea that the small platoon passes the bottleneck with little change in their disciplined regular car following spacing and velocity.

The fact that the time headway of the Jth vehicle in platoon increased with J, led the author to consider enhancing flow by deliberate platooning. The number of gaps introduced must be minimized to maximize the flow. It was suggested that the gap between platoons at the end of the bottleneck must be less than 5 seconds for any improvement in flow. A value of two seconds gave a maximum increase of 75 vph for platoons of 17 cars. The gap to be introduced between platoons at the control point should be 7.5 seconds.

Edie and Foote (1961) in a study of fluid models (Light-hill & Whitham) concluded that the flow-density curve did not reflect a behavior exactly like that suggested by fluid dynamics, but there were noteworthy similarities. Data
from The Holland Tunnel and The Lincoln Tunnel showed that preventing short period flows greater than the tunnel capacity would improve traffic flow through bottlenecks. In The Lincoln Tunnel continuous feed method increased the flow of traffic, versus the police control which had to let the traffic from 6 toll lanes of traffic enter the tunnel in rotation (the tunnel has a total of 6 lanes). Gaps up to 15 seconds frequently occurred which was suspected that these gaps contributed to loss in capacity under police control condition. In The Holland Tunnel congested flow was improved by the deliberate creation of gaps in traffic stream; supporting the previous study.

Newell (1961) proposed a non-linear car following model in which the velocity of nth car at time $t$ was some non-linear function of headway at time $t-T$. It is possible to incorporate all previous linear car following models into this model. This model has combined the sensitivity and stimulus into one function. The shock wave predicted by this model may or may not exist as mentioned by the author. This model was not validated by field data.

Herman & Potts (1961) in a study of linear car following rules, examined three different functions for the proportionality constant. A typical car following law is

$$\ddot{x}_{n+1}(t+T) = a[\dot{x}_n(t) - \dot{x}_{n+1}(t)]$$
When Alpha is constant the model is simple in form and gives good approximation, but it is independent of spacing of the cars. The second function was a step function in which two different values were used depending on the spacing of the two cars. The third function was reciprocal-spacing law which provides reasonable explanation of the experimentally determined relation between the flow and concentration of traffic. This third function gave better fit than the other two.

The constant function was used for study of local and asymptotic stability. For step function and reciprocal-spacing function it was not theoretically possible to evaluate the limits for the local and asymptotic stability. They found that when car spacing was over 200 ft there was small correlation between the cars. In this experiment the observed range of reaction time of drivers in a platoon was 0.42-1.01 seconds.

Edie (1961) proposed a car following model for noncongested traffic since Greenberg and Herman et al models became less realistic for low density flow.

\[
\dot{X}_{n+1}(t) = \frac{\lambda X_{n+1}(t)|\dot{X}_{n}(t-T) - \dot{X}_{n+1}(t-T)|}{h(X_{n}(t-T) - X_{n+1}(t-T))^2}
\]

Integration of this equation and application of boundary conditions yielded the following speed-density relationship:
The same data Greenberg used was used for validation of this model. The two models differ very little when normalized, however, the maximum flow does not occur at the same point on the curve. Edie believes that there is no obvious reason why the mean free velocity of a roadway should be established by the same variable controlling its mean jam density. Jam densities in a tunnel are high with as many as 200 to 250 vpm, yet the free velocity is lower than that of freeways, which has a jam density of only 150 to 200 vpm.

After looking at many sets of data and flow-density plots, Edie noticed a discontinuity and existance of some kind of change of state in the relationship. The discontinuity is in density range of 70-100 vpm. For the data from The Lincoln Tunnel, two model were fitted. For non-congested flow (less than 90 vpm) Edie's proposed model and for the congested flow Greenberg's model was used. This dual model fit was superior to single model fit. As it is mentioned by the author, the extreme of very light traffic, also the extreme of very dense traffic may not correspond to this proposed two model car following rule.

It has been established that there is a correlation between the response of a driver and relative speed of his car and the leading car. Therefore, the relative speed is

\[ K = K_m \cdot \ln(U_f / U) \]
used as the stimulus. For sensitivity different functions have been used.

Gazis, Herman, Rothery (1961) proposed a general expression for sensitivity which included all previous sensitivity functions. The car following model is:

\[ x_{n+1}(t+T) = \lambda [\dot{x}_n(t) - \dot{x}_{n+1}(t)] \]

Where

\[ \lambda = \frac{a \dot{x}_n^m(t+T)}{(x_n(t) - x_{n+1}(t))^l} \]

Substitution of \( \lambda \) in the car following equation and integration yields:

\[ f_m(U) = C f_l(S) + C' \]

For different values of \( m \) and \( l \) different speed density graphs can be plotted. They also showed that from the stopping distance formula one can get the stimulus-response equation. In this paper they supported Edie's observation of apparent discontinuity of the speed-density relation near the peak flow. Furthermore, they mentioned that the possibility of existence of a bimodel character of the flow versus density curve for a multilane highway may be greater than that for a tunnel where no passing is allowed. From the limited data they could not establish the superiority of one model versus the other, but they gave ranges of \( m \) and \( l \)
and 1. The range of \( m \) is between \(-1\) and \(2\), and the range of \( l \) is between \(1\) and \(3\).

Kometani & Sasaki (1961) employed the concept of braking distance to study the dynamic behavior of traffic on streets and highways. The equation is expressed as:

\[ x_n(t-T) - x_{n+1}(t-T) = C_1 u_n^2(t-T) + C_2 u_{n+1}^2(t) + C_3 u_{n+1}(t) + C_4 \]

Where \( C_1, C_2, C_3, C_4 \) are proportionality constants. Their experiment showed that the amplitude of the speed-time curve of the following car is greater than the lead car. They also found that if the speed variation of the lead car is a sinusoidal wave, that of the following car is superposition of an infinite number of sinusoidal waves.

Understanding of the propagation of disturbances in traffic stream and factors affecting the behavior of driver in car following situations are essential for modeling of traffic flow at microscopic level.

Wright & Sleight (1962) used an experimental platoon to study the effect of simple judgement aid on the following distance. The result of analysis indicated that it is possible to reduce error substantially by providing simple judgement aids. The use of visual and timing aids resulted in significantly lessening the tendency to follow at a greater than the required distance.
Michaels & Solomon (1962) studied the effect of speed change information on spacing between vehicles. Intervehicular spacing is important in the stability of car following patterns and thereby influences the volume of traffic that can be moved on the highway. Advance speed change information was given to the drivers of the following car. The effect of the use of advance speed change information transmission on the spacing between two vehicles varied with the speed at which the vehicles were moving. It had little effect when the speed was 36 mph, but resulted in a large reduction in spacing when the speed was 54 mph. The transmission of the information approximately 3 sec. before initiation of the speed change by the lead car permitted maintenance of the most uniform headway. Uniform headways in car following provides a comfortable ride for drivers and increases capacity for the roadway. However, it is not possible to provide such a uniform spacing in actual traffic conditions. Not only is there variation of distance headway in one lane, but also spacing is not the same in different lanes because of lane interaction.

Gazis, Herman, and Weiss (1962) investigated the density oscillations between lanes of a multilane highway; and reported the conditions for damping and undamping fluctuations for a 2 lane model. On the basis of this model danger of instability increases with the number of lanes.
Propogation of disturbances in a line of vehicles can be stopped when the arrival of vehicles are temporarily halted to the congested section of the road. This concept was used successfully by the Port of New York Authority in tunnel traffic operation.

Foote, Crowley, and Gonseth (1962) developed and used successfully a surveillance system which introduced gaps into traffic stream in order to maintain the traffic speed above the level where shock waves are generated. The efficiency of operation was increased by taking actions which minimized the loss in capacity due to shock waves. The previous experiments on this system were discussed before.

Bierly (1963) carried out an investigation of the effect of intervehicular spacing display on the following driver behavior in a car following situation. A significant reduction on the average absolute spacing error was observed. However, the spacing display alone did not affect the reaction time of the following driver. When the spacing and the relative velocity were shown, the following performance was significantly improved. Also the driver reaction time was reduced markedly; in other words, the driver sensitivity was increased.

Forbes (1963) studied time headways of the following drivers in different car following situations. The time
headway of 1.0-1.5 seconds range was observed for an experimental platoon of 3 cars when going into a slowdown and approximately twice that much when going out of the slowdown. More investigations were performed on this phenomenon and its contribution to the discontinuity reported by Edie was discussed. From analysis of several data sets, it was evident that the time headway between vehicles going out of a slowdown must be decreased, or the time headway between vehicles going into a slowdown must be increased, relative to each other in order to have the stoppage or slowdown to dissipate rather than continue to build up. When drivers are reasonably confident about the free flow on the highway they will maintain the speed and minimum response time within platoons. But when a slowdown, an expected bottleneck, poor visibility, or other confidence-reducing factors are reached, they will return to longer response times. The study suggested two different kinds of driver responses. One for open flow and another for expected congestion.

Daou (1964) looked at space headways within platoons in The Holland Tunnel. Vehicles with space headways greater than 200 feet were not considered a platoon member. Vehicles with a space headway less than 200 feet were divided into speed classes of 5 ft/sec, ranging from 15 to 60 ft/sec. The distribution of vehicle headways for each speed
class was plotted and fitted to the lognormal distribution. It was further noticed that the mean of the headway distribution was a function of speed. This implies that within a platoon, flow increases with speed.

Lerner, Abbott, and Sleight (1964) examined the effect of speed, trip duration, and day or night driving on following distance on the highway. Analysis of variance technique were used for the data analysis. The drivers were not asked to follow as if they were in congested traffic, but to follow the leader who was showing the way to a destination. At speed of 35 mph the mean of following distance for all subjects were 118 ft and standard deviation was 62 feet. While at 55 mph, it was 177 feet with standard deviation of 97 feet. The drivers kept longer distance than what the "rule-of-ten" gives. Duration of 2 hours driving had no effect; and most drivers maintained approximately the same following distance during day and night. Following distance by different drivers varied over a wide range, while that of the individual driver was relatively consistant.

Braunstein and Laughery (1964) found the rate of change of velocity of the leading vehicle had a definite effect on response latency for deceleration. The velocity change detection time increases with intervehiculer separation and decreased with increasing rate of change of velocity.
A.3 STUDIES DURING 1965-1975

Another approach to the analysis of car following is the study of human response process in car following situations. During this time period there were many investigations using this approach. Some of them along with the other developments will be reviewed in this section.

Michaels (1965) used the horizontal angle subtended by the lead vehicle, and the rate of change of the angle in the study of the overtaking process. The drivers adjust their speed such that they maintain the observed angular velocity at or very close to their threshold. The drivers perception of relative velocity is a function of headway. Large separations will produce a slow reaction and requires rather large speed variation for detection of speed changes. On the other hand, at short headways the driver is very sensitive to small change in relative velocity. The response time to deceleration of the lead vehicle varies with the magnitude of deceleration and average queue speed. For the most common region of 1-2 mph/sec/sec the response time varies from 0.75 to 2.0 seconds. An estimate of relative velocity range within which a driver would detect no change in that velocity are: 2.1, 3.8, 6.3 ft/sec for spacing of 100, 135, and 175 respectively.
The following driver may respond too little or too late to a change in velocity of the leading car, and over correct the error and amplify the disturbance. A small initial disturbance could grow as it propagates down in a long line of cars until one finally has a large disturbance generated from practically nothing (Newell 1965).

Newell (1965) proposed an alternative model in which delay was considered to be more a consequence of laziness or an intentional failure of drivers to respond to every stimulus, rather than some inherent limitation on reaction times. Two velocity-headway relations, one for during acceleration and the other one for during deceleration was proposed. The former had longer headways than the latter for the same velocity. This model was used for study of instability and shock waves in traffic flow.

Herman & Rothery (1965) reported a dissymmetry in performance of drivers in maximum acceleration and maximum deceleration. Small acceleration disturbances propagated more slowly than deceleration waves. They also reported that the reaction time of the second following car (3rd car in car-following situation) did not decrease when information about acceleration/deceleration and coasting of the leading car was given to the driver of the car. The average reaction time in The Holland Tunnel was 1.4, on test
track 1.6, on 3 car case 1.6 seconds. Therefore, the most significant aspect to be considered in car following is the car immediately ahead. For the linear car following model, giving information about acceleration, coasting and braking increased the sensitivity coefficient for each driver, and variance in the relative speed was substantially reduced. The stability was not improved even though reaction times were decreased. The nonlinear model was used for analysis of data, the result indicated that individuals used the information in order to drive with a smaller spacing than they did when no information was given.

Drew (1965) generalized Greenberg's model and proposed a general model.

Foote (1965) compared Q-K curves for the data taken between entrance portal and the foot of the upgrade with the data taken at the foot of upgrade in The Holland Tunnel. There was a significant shift in critical density, from 85 to 70 vph. The behavior of platoon before the bottleneck and at the bottleneck were compared. There is a distinct tendency for speed to decay within the platoon as they pass through the foot of the upgrade. The range of the differences in speed for the nth car in the platoon were from 7 to 14 ft/sec. depending on the vehicle. The cause of the shock waves were due to change of upgrade.
The decay in speed was gradual and the shock waves were generated as a result of a cumulative build up of small decelerations. Automatic control devices were used to insert gaps to reduce the number of vehicles entering the tunnel when the speed was less than 25 mph. The result was an increase in velocity.

Cosgriff, Rockwell, and Treiterer (1965) conducted a study on application of electronic devices as traffic aids. An automatic longitudinal control system for automobiles was developed. This system consists of a sensing system, a communication system, and an actuating system which deals with braking or acceleration. The performance of the conventional man-machine system was studied using Todosiev's model. The point at which the driver decides to change speed is called the decision point and the corresponding car following model is called the decision model (Barbosa, 1961).

Todosiev (1963) modified this model and obtained the action point model. He hypothesized that a driver, before changing vehicle acceleration, went through a perception-decision-action process. Here, maintaining of a constant acceleration or deceleration until another change occurs is considered an action. Analysis of action point model of car following revealed that the finite time required for a driver to detect slow relative motion of a lead vehicle is
a main factor causing erratic headway control. Thus, if headway control is to be improved, the driver's effective detection time must be decreased. Such a decrease can be achieved by presenting certain additional information to the drivers. Significant results were obtained when information was presented via a tactile aiding device, or finger, which was built into the head of a control stick. Sizeable reduction in velocity and headway variance were obtained from simulation studies.

They also found that, lower velocities were underestimated and higher velocities were overestimated. Acceleration change thresholds were in the region of 0.005g to 0.010g under ideal conditions. To improve car following performance headway and relative velocity were displayed for the following car. The analysis of headway and relative velocity variance showed that for greater car following distance (for headway) the headway is improved 69% and relative velocity is improved 169% over normal driving (no display). However, when the drivers were asked to maintain a short headway (closed headway versus far headway) there was not much difference between normal driving and display of relative velocity or headway.

Their analysis of data showed that there exists a critical density (approximately 55 vpm) beyond which no stable
relationship is observable. Beyond this critical density aids of some kind are needed to maintain stability in traffic flow. The distribution of spacing was tested using composite exponential and the Pearson Type III, and the new function, lognormal. The results indicated that the lognormal function approximates the observed distribution of spacings better than the other functions. Spacing which was found to be a major parameter in car following model can be influenced by electronic aids in order to induce stability into traffic flow in the range beyond critical density. This study also found that the changes in acceleration are the results of discrete decisions not a continuous change in acceleration. The expression for acceleration is given as follows (Research Staff, 1965).

\[
\dot{U}_2(t) = \frac{-2*(\text{relative velocity})^2(t-T)}{(\text{desired spacing} - \text{spacing})(t-T)}
\]

Due to the response lag time and arbitrary assignment of acceleration rate, they concluded that sufficient stability in the model was achieved when spacing between vehicles remained within 10 ft of desired spacing.

They used acceleration rate of -16 ft/sec/sec for ideal road conditions and a faultless braking system. Vehicles maximum acceleration rate is a function of their velocity. The function used by the Research Staff is:

\[
a_2 = U_2*e^{0.02}
\]
8 ft/sec/sec was used as the maximum value.

Torf & Duckstein (1966) divided driver perception latency in car following into 4 parts: detection time, decision time, subject reaction time, and car response time. Detection time plus decision time is called perception time. For a base velocity of 40 mph and gap spacing of 80 feet between cars they found that perception times for deceleration (deceleration were less than 2 ft/sec/sec) were longer than those for acceleration. This does not agree with the results of other research.

Lee (1966) generalized linear car following theory by introducing a memory function that defined the way in which the following driver processes the information concerning the relative speeds. The response of the follower depends not on what the relative speed was at a certain earlier instant, but rather on its time history. Different memory functions were examined, but no experimental validation was done.

Unwin & Duckstein (1967) examined the stability of reciprocal-spacing type car following models. They discussed the stability of following cars in the sense of Liapunov. This means that some changes in the internal condition only produces a small change throughout the solution. Thus, it is possible change the lead car's
behavior slightly without causing the following car to behave much differently. The system should be stable in the sense of Liapounov, regardless of the considered starting time. They also proposed a model, which exhibited asymptotic stability, by adding a feedback term proportional to the deviation of the spacing from desired steady-state spacing. And at the same time an attempt was made to justify this model by extending the Weber Ratio to a car following situation. The Weber Ratio hypothesizes that a human being responds to a change (delta S) of an applied stimulus S by an amount (delta R) of response R that is proportional to (delta S)/S.

Treiterer (1967) suggested that the stability of traffic flow and safety can be improved by longitudinal control. Longitudinal control can increase the traffic volume by reducing the reaction time of following driver. His investigation on highway traffic flow revealed that about 90% of the drivers have adopted marginal safety in car following when a 0.7 sec. (assumed value) reaction time is used as the minimum value. Only about 60% of time drivers implicitly accepted the recommendation of one car length per 10 mph speed for safe spacing. Maximum marginally safe flow as a function of reaction time has shown that when the stability condition of 2(Lamda)(Tau)<=1 was satisfied, only response times of less than 0.8 sec will result in considerable traffic flow increase.
Smeed (1967) introduced the strategy of arrival time control of vehicles to a point on the road, to reduce the total journey time for all drivers. Two conditions should be satisfied in order for the vehicles to reach their destinations earlier by starting later. He discussed that, when the two conditions are met all vehicles will arrive to their destinations earlier and the sum of journey time for all vehicles is lessend.

Pipes (1967) demonstrated the derivation of Greenshields model from models based on the rate of change of visual angle. A general model was proposed which included Greenshields' and Greenberg's model.

May & Keller (1967) studied, for the first time, car following models with non-integer exponents. They showed that macroscopic and microscopic theories of traffic can be reduced to the equation of the general car following model of Gazis et al (1961). Speed density relationships for different values of $l$ and $m$ were given. On $m-l$ plane, the area between lines $m=0.5$ and $l=2.5$ and $m=2.5$ and $l=3.5$ covered models of very good fit, but with a maximum flow rate of less than 1800 vph. Models with a maximum flow rate greater than 1800 vph appeared in the area with lower $m$ and higher $l$ values ($m$ ranges from from -1 to 3, and $l$ from -1 to 4).
Herman & Rothery (1967) conducted an experiment with a platoon of 11 vehicles on a test track facility to determine the speed of propagation of disturbances or kinematic waves; and the simultaneous measurement of traffic stream parameters in order to make direct comparison. Standard deviation of speed of vehicles increased for vehicles down the line. When speed of the lead car was approximately 15 mph the fluctuation was more and the 11th car came to a complete stop 3 times during a 4-minute run. The plot of calculated values of speed propagation of a disturbance with respect to the traffic stream, $k*du/dk$, versus speed of traffic did not lead to a relationship valid for the entire range of traffic. Note that the speed of propagation of disturbance with respect to the roadway is equal to sum of speed of traffic stream and $k*du/dk$; $dq/dk = u+k*du/dk$. The study of a plot of experimentally obtained propagation time versus platoon length revealed that there is a marked difference between acceleration and deceleration disturbances.

Edie & Bavarez (1967) in study of generation and propagation of stop-start traffic waves in tunnel indicated that the continuum theories based on fluid dynamics (Lighthill & Whitham, 1955; Richards, 1956) failed to explain the nature of periodic recurrence of stoppage waves or shock waves in a tunnel. For detail study of this recurrent phenomenon, contours over space and time of average flow, concentra-
tion, and speed for a section of a roadway about 600 feet long in the region of the bottleneck of The Holland Tunnel was constructed. They also observed that initial steady-state condition of 22-24 mph rapidly deteriorates into unstable state of even slower speed (4 mph). The slow speed condition propagates backward in space at about the stream speed, then traffic stream returns to another steady state at the initial speed of about 22-24 mph. All this occurred within a period of 3 min. They indicated that:

This result suggests that for this scale of measurement, small changes in flow may not propagate at a speed equal to the slope of the tangent to a steady-state q-k curve as suggested by hydrodynamic wave theories of traffic flow. Instead, they are carried along at about stream speed or only slightly less than stream speed right up to saturation flows, at which level they suddenly reverse direction.

They also reported mean values of 1.1 mph for deceleration and 1.0 mph for acceleration. The generation of stop-start waves resulted from arrival of a pulse of high level flow at a time when the density was in the order of 70-90 vpm. Vehicles found themselves too close together in time and slowed successively to lower and lower speeds in an effort to reduce their mean time headways. As a result the density rose up to 100-120 vpm.

Foote & Crowley (1967) developed an automatic system to control density of traffic to improve traffic flow in tunnel. The automatic control system would limit input flow
whenever 6 or more vehicles were observed in any minute traveling through the output section at speeds less than 30 fps. This strategy maintained a fluid traffic movement with density of less than 50 vpm and speed above 30 fps.

One way to increase the traffic flow on crowded highways is to decrease the time headway between the cars. However, smaller spacing between the cars increased the probability of rear-end collision and may cause chain reaction leading to shock waves in flow.

Gantzer & Rockwell (1967) hypothesized that the time it takes the driver to perceive a change in the car following environment could be decreased by presenting the driver with information before he visually perceives such a change. This might improve car following performance. In an experiment the following driver was informed by given red or green light whether he is within a established bandwidth. The result showed that a discrete light display presenting headway and relative velocity information can improve car following system performance. A 60% reduction of headway variance just by using a headway bandwidth display were obtained.

Fox & Lehman (1967) investigated stable and unstable behavior of traffic with regard to accident-producing situations through simulation of a single lane traffic.
The driver of following car is considered in velocity-detection mode when the threshold test quantity for that driver exceeded a present value for velocity threshold. Under threshold value he was in a distance-detecting mode. On velocity-detecting mode, a driver responded roughly in accordance with the Edie's model (1961), with some considerations. For example Alpha takes different values for acceleration and deceleration, it is also limited to 8 to 11 ft/sec/sec as a comfortable value. Each driver is assigned a certain velocity and attempting to keep his speed within +/-15% or so of this value. On distance-detecting mode a driver cannot detect relative velocity, but he can determine his action on the basis of desired velocity, spacing and so on. The brake light of the lead car is included in the model, it is on whenever his deceleration exceeds that caused by drag (T.E Handbook, 3rd ed. 1965 p.26).

According to their car following, a driver considers relative spacing and velocity between his car and the car two ahead. The reaction time, t, varies not only from driver to driver, but also varies depending on the detection mode of the driver. His reaction time in velocity-detection mode is smaller than distance detecting mode. Time between computation is 0.1 sec. on IBM-1620-II. A half min. of real time driving for a platoon of six cars requies
15 min of computer time. No actual validation is done because of lack of fine data. They did after-the-fact validation (investigation of parameter values suitable for representing typical car following phenomenon).

Rockwell, Ernest, and Hanken (1968) did a sensitivity analysis of empirically derived car following models through the use of piecewise linear regression; and showed that relation between following car acceleration; and following car speed, lead car speed, and headway is linear. They also observed that the following car responded weakly to changes in headway, but strongly to changes in speed of the lead car.

Gorden & Mast (1968) conducted a study to find out how well drivers can judge the distance required to overtake and pass by. They indicated that drivers do not estimate passing and overtaking distance accurately; they underestimate the distance required for maneuvering more than half of the time.

Rockwell & Treiterer (1968) proposed a model for car following study which indicated that the following car would duplicate the acceleration of the lead car after a short time lag. An acceleration control term expressed as $kx''(t)$ was added to the equation of the linear car following model. The study suggested that proper guiding of
vehicles movement at high traffic density can improve safety and capacity of the road.

Forbes & Simpson (1968) examined the drivers response time in freeway deceleration waves by taking air photo records of traffic in 1 sec intervals on congested and less congested sections of the John Lodge Expressway. Average time headways were longer downstream than they were upstream from the wave. They were also longer on congested sections than noncongested sections. Within platoon time headways of less than 1.0 sec. occurred frequently while between platoons the time headways might be in order of 3.0 to 4.0 sec. Longer time headways on congested sections were because of drivers uncertainty and anticipation of congestion ahead which were apparent on the lane signals, visible slowing ahead, on this section. Traffic sign and or signals and other drivers aid that reduces uncertainty will increase safety, if the drivers are alerted.

They indicated that the deceleration wave propagation response time will approach true driver sensitivity (min. perception-judgement-response) when time headways are very short. To operate safely with individual headway time of less than 1.0 sec., as reported here and previous studies, drivers must be alert and respond to reactions of more than one car ahead.
Bexelius (1968) reported that a driver observes at least the two nearest cars ahead. A model was proposed and the stability conditions were discussed. This is contrary to Herman et al (1965) conclusion that, the influence of more than one preceding car on car following is not significant.

Gorden (1970) concluded that most of the drivers in a medium freeway traffic escaped the obstruction (slow moving vehicles) by simply shifting lanes. Drivers start to react to an overtaken vehicle at a distance of 250 ft or less, lane shifting completed at a distance of about 100 ft from the car infront. In passing situations the drivers slow down in anticipation of being blocked and goes through a delay phase of 15-35 sec. in which the driver moves to a slightly closer position or matches pace with the car in front, and finally passes.

Treiterer (1972); Lee (1971) observed that when a platoon of vehicles is subjected to a kinematic disturbance, the traffic moved slower in the queue releasing condition than the queue forming condition for the same density values. Speed density graphs showed that the retarded traffic movement occurred immediately after maximum density. The traffic recovered at density of 73 vpm for one platoon and 59 vpm in the other one. When traffic has reached its point of recovery then the density stopped decreasing but
the average speed of the platoon kept increasing for some-
time and then a reduction in density occurred. This phenom-
non, which was called hysteresis phenomenon, was observed
on volume-density and volume-speed relationships, too.

There were two loops on the graphs, the hysteresis loop
and the energy-gain loop. The normal behavior of traffic
before maximum density and the retarded behavior of traffic
immediately following maximum density formed the hysteresis
loop. The normal behavior of traffic before the maximum
density, and the higher than normal performance of traffic
on its recovery path formed the energy-gain loop.

They observed that average platoon headway (for platoon
A) was 2 sec. in density region of 33-110 vpm. The aver-
age time headway then increased, first slowly, and then
sharply as traffic approached its maximum density. When the
platoon was released from the maximum density, the headway
decreased but maintained higher values when compared to
"before" condition. It reached the value of 2 seconds at
73 vpm and kept decreasing without pronounced variations in
density. The average headway then increased again with a
reduction in density after it reached a local minimum of
about one sec. and again assumed the before condition at 33
vpm. For platoon B the same observation but different den-
sity values were reported.
As a result of the observation, they proposed a multilinear speed density relationship for prior queue condition. In each region speed-density relationship is linear and the regions are connected to each other (no jump on fall). There were six regions.

Gorden (1971) experimented that when a slow moving vehicle (30-35 mph) was introduced into the traffic stream of a 2 lane highway, 10% of drivers responded with avoidance, 30% with car following, and 60% with mix of the two reactions. The data from this study and previous study (Gordon 1970) showed that driver lead distance pattern did not conform to the Herman et al reciprocal-spacing car following law. In both of the studies traffic volume was not high and the drivers had opportunities to change lanes.

Kapur (1971) used calculus of variables to optimize various objective functions for vehicle following dynamics. This approach gives some of the well-known vehicle following equations and some generalized vehicle following equations.

Heyes & Ashworth (1972) used a form of non-linear car following model to study stable car following situations. They suggested that a driver is able to judge his time headway when traveling in a medium-high density traffic stream, and his response is a function of that time. For
the time headway observations in Mersey Tunnel the lognormal distribution was reported to provide the best fit.

El-hosaini (1973) studied transport behavior at high density, and speed range of below 7 mph. The model for this range is based on compressible gas dynamics. It is assumed that the traffic behaves like a compressible gas under piston action. This analogy yielded the following model:

\[ v = c(2\text{LnK}/K)^{0.5} \]

Analysis of data revealed that most drivers at very high densities and very low speeds (below 7 mph) behave differently from Greenberg's model. Individual drivers tend to decrease their speed rapidly near jam density. This action is reflected on Q-K relationship with a curved-down end near jam density.

Eshler (1973); Rackoff & Rockwell (1973) simulated the effect of car following aids on traffic safety and the use of driver display and control aids to stabilize and improve traffic flow. In the simulation model the driver was either in a distance-detecting mode or in a velocity-detecting mode. A form of non-linear car following models were used. A driver would detect a change in vehicle relationship when the rate of spacing change exceeds his threshold for that quantity. The response time for each driver varies individually over time depending on his driving situation. When the rate of change goes over the
threshold for a driver and he goes into the velocity-detecting mode, his reaction time drops from a large value in the distance detecting mode to a smaller value. When the rate of spacing change remains below threshold, reaction time can build up once again to a maximum value. Maximum reaction time of 1.4 seconds was used to represent the unaided driver, while 1.2 and 1.0 sec. for aided drivers.

The car following aids were limited primarily to two general types: rate of spacing change aids (threshold), and reaction time aids. The results of simulation showed that these two are similar in their effect on the platoon flow measures of effectiveness, velocity variance, and headway variance. Both of them reduce these MOEs. Combination of these two aids is much more effective.

Treiterer (1970); Chandrangsu (1973) examined traffic flow models based on energy concepts. Mathematical models were tested using field data collected from aerial photography of I-71 southbound in Columbus area. The results indicated that work and energy concepts can be used to describe the movement of a vehicular platoon toward the congestion region. The concept was also used to describe traffic stream.

The total energy of traffic stream is the sum of potential (internal) energy and kinetic energy. For kinetic
energy Drew in his book used $aku^{2}$ and for internal energy the acceleration noise. The studies showed that if Drew's definition is used for kinetic energy, then the principle of conservation of energy was reported not to hold. They proposed coefficient of variation of speed as the indicator of internal energy. Inspite of the inapplicability of the principle of conservation of energy, the concept of kinetic energy and internal energy made important contributions to understanding of traffic dynamics.

Kohler (1974) investigated asymptotic stability of vehicle platoons using general car following model. The use of a distance warning indicator is suggested and used in a simulation study. A driver must be warned by the indicator the latest at the point at which a specified distance was underrun. The distance warning indicator was not designed on the basis of braking distance because vehicles are moving in platoon and their speed is controlled by other vehicles.

Evans & Rothery (1974) experimental study of detection of the sign of relative motion in car following situation showed that, the sign of relative motion can be explained in terms of spacing change divided by spacing, or by the ratio of average relative speed and spacing. Drivers underestimated the distance between his car and the car ahead.
Edie (1974) stated that the car following phenomenon can be approximated by a simple differential-difference equation. Even though the stimulus-response description would in practice be a function of dynamical properties of the car and of the psychological and physical characteristics of the drivers. The reason for this is that a driver in order to avoid an accident and get to the destination quickly, cooperates with the other drivers by using a simple average response to a limited number of stimuli.

A.4 STUDIES SINCE 1975

Treiterer & Research Staff's (1975) study of safety in dense traffic concluded that high density traffic is at its best marginally safe (spacing=vt) assuming almost ideal conditions (11% unsafe driving at a reaction time of 0.7 sec.)

The degree of safety in car following for a platoon of vehicles and a change in traffic flow were the two factors considered for investigation of propagation of disturbances and study of instability of traffic flow by Research Staff. The study indicated that instability arises from the variance in the reaction of the drivers to changes in velocity by the leading vehicles. It was found that instability can occur for no apparent reason. The sensitivity factor pro-
posed by Gazis et al (1961) was used. The condition for the stable flow which is $2(\lambda)(\tau) \leq 1$ was checked and the distribution of its values for entering, leaving, within disturbance, and for uniform flow was obtained. For entering disturbance quite a number of $2(\lambda)(\tau)$ values exceeded unity. For the others the distribution was on both sides of unity.

Also they used this data to evaluate seven speed-density models, and concluded that multi-regime models gave better fit than single-regime models. They also observed that traffic stream does not recover at the same rate with density as it entered jam condition. The data collected from I-70 (for small platoon, size 7) supported the hysteresis phenomenon observed in a totally different data collected from another interstate freeway (I-71). The only difference was that the A loop was reversed. The traffic entered the disturbance at a lower speed than the speed gained in the recovery phase. The situation on I-70 was a typical example of recycling around the A loop which resulted in a stop-and-go operation with higher density. The condition on I-71 which was a single disturbance, and the one on I-70 which was multiple disturbance, both, supported the notion of hysteresis phenomenon on the traffic flow.
Ceder & May (1976) supported the two regime approach proposed by Edie in 1961. The traffic flow model was not the same for the entire density range but had one model for free flow regime and another for congested flow regime. They used 50-60 vpm as the discontinuity range, and found m and l values for each model. The two-regime model provided only slightly better representation of the data set although it did support the visual appearance of the two regime phenomenon.

Ceder (1976) replaced this two regime generalized car following model with a new model. This model did not have the complicated and complex function of the generalized model. The sensitivity function of this model is in the following form \( a(A^{-(S_p/S)})/S^2 \) where S is spacing and A is a weighting factor. The comparison of this model with their previous model yielded that the proposed model fits better, it is simple and clear to understand.

Ceder (1977) in the time sequence analysis of speed-concentration-flow data (5-min ave) on urban freeway in Los Angeles revealed that on the Q-K curves 5 phases can be identified. These phases are very similar to those found by Treiterer, In view of the two regime traffic behavior data points were separated in two groups. The analysis showed that there is strict distinction between the free flow regime and the congested flow regime.
Kohler (1979) carried out a simulation study to evaluate the effect of distance control strategies on car following. As the traffic volume increases the number of vehicles with following space influenced by the presence of other vehicles increases too. The fast moving vehicles approach the slow moving vehicles and follow them or pass them if possible. The driver of following car was warned if the spacing was less than the required distance for braking. Measurements taken from a 4 lane highway showed that a portion of vehicles with too small a distance both in approaching of a slow vehicle or following it. Using car following equation of Gazis-Herman, the author found the criteria for asymptotic stability of car following. This criteria which is a function of speed is used for warning of the drivers of the short spacing. This strategy increased the safety and reduced the rear-end-collision. It also decreases the capacity of the roadway when distance warning strategy was compared to no warning strategy; in a simulated condition. Maximum Q for distance warning situation was 1200 vph and for no warning situation the volume was 1440 vph.

Ceder (1979) reported the possibility of existence of clockwise looping fluctuations on the plot of spacing versus relative speed of a pair of cars in car following situation. Similar analysis by Todosiev(1963) showed the existence of multiple cycles in the phase plane. These studies
showed that drivers tend to cycle around an approximate equilibrium spacing.

Allen, Hall, and Gunter (1985) discussed speed-flow relationships when traffic flow moved between the congested and uncongested regimes. The transition from the congested to uncongested flow was identified by a 15 Km/h speed change for the data collection station.

Gunter and Hall (1986) altered the previous results and used speed-occupancy to identify critical operations. They concluded that the identified transition toward both regimes do not occur at a constant flow rate as it was mentioned in the previous paper. Breakdowns and recoveries tend to occur with a 200-400 passenger car units per hour flow change.

Hall, and Gunter (1986) supported the idea of having different speed-flow relationships for each lane, and various locations on a given lane. They indicated that discontinuous models are not convincing, and proposed an inverted V shape speed-flow relations. The inverted V shape model for shoulder lane was not as good as it was for the median and middle lanes; and did not have a sharp peak on the left hand side of the speed-flow curve.
Appendix B

DISCONTINUITY IN TRAFFIC FLOW MODELS

The assumption that the relationship between speed and density is a single straight line, Greenshields' model, helps to explain the overall behavior of traffic flow in an oversimplified manner. However, this explanation leaves unanswered some of the most important questions about the behavior of traffic when suffering from a kinematic disturbance. In fact all of the single regime models have this deficiency. An abrupt change of operation from uncongested to congested or vice versa cannot be explained with a smooth speed-density curve or line. There have been many observations and investigations about dual mode, or discontinuity, or sudden jump and fall on speed-density-volume relationships. In the following sections some of the research will be discussed in order to help to understand behavior of traffic over a wide range of density. This knowledge will be used in the development of a realistic car following and simulation model.
Greenberg (1959) used the continuity equation to describe traffic flow characteristics and assumed that the traffic would behave like a continuous fluid, except at the lowest density. Traffic flow for a bottleneck was assumed to be fluid flow too. However, it was mentioned that behind the bottleneck the traffic stream is often nonfluid and care must be taken in applying the fluid flow analysis. An alternative type of analysis may be necessary.

Edie (1961) was the first one to point out the discontinuity in the region of maximum traffic flow. A dual model rather than a single model was proposed for the first time. The model for uncongested flow was different than the model for the congested flow region.

Gazis et al (1961) supported Edie's observation of an apparent discontinuity of flow near the maximum flow. They indicated that the possibility of the existence of a bimodel character of flow versus concentration for a multilane highway may be greater than the possibility for tunnel traffic.

Underwood (1962) divided traffic flow into 3 zones: normal flow, unstable flow, and forced flow. For normal flow, a straight line would represent the relationship between speed and volume. However, this relationship will not apply as the density increases. When density reaches about 50-60
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Vpm the flow would break down about 50% of the time. For an unstable flow zone there is not a definite relation between speed and volume; it is a transition state and a zone of separation between the two zones.

Ryan & Breunig (1962) reported that the change from stable flow (non-critical) to unstable flow (critical) is fast and traffic flow failure occurs suddenly without a definite warning sign. Their analysis of freeway data showed the relationship between Q, K, and U are linear in the non-critical flow region, but not in the area of critical flow. A speed of 40 mph was suggested as the boundary between critical and non-critical flow; and the linearity ends at this boundary.

Forbes (1963) investigated the differences in time headways when an experimental platoon was going into a slowdown with those of coming out of the slowdown. The range of time headways of 1.0-1.5 seconds for going into a slowdown and approximately twice the values were reported for leaving the disturbance. The contribution of the differences to the discontinuity reported by Edie was discussed, and the drivers responses were separated for free flow and congested flow conditions.

It was suggested that, in order to dissipate the slowdown or a stoppage of traffic flow, the time headway
between vehicles coming out of the slowdown should be decreased; or for the vehicles going into the slowdown it should be increased.

Newell (1965) considered the delay in response of a driver to a stimulus to be more of a consequence of laziness or intentional failure than some inherent limitation on reaction time. Two different velocity headway relations, one for during acceleration with longer headways, and the second one for deceleration with shorter headways were proposed.

Herman & Rothery (1965, 1967) reported dissymmetry in performance of vehicles in maximum acceleration and maximum deceleration. Acceleration disturbances were propagated slower than the deceleration waves.

Welding (1965) gave several possible reasons about the departure of traffic behavior from the fluid theory of Lighthill & Whitham especially in bottlenecks. The theory was not held accountable for the falling of a bottleneck capacity below its normal capacity. A relaxation of practical evidence of a complete Q-K curve was called. Time series analysis is suggested for investigation of traffic flow along a road which has a constriction (a bottleneck).
Franklin (1967) in studying the propagation of disturbances in traffic stream on tracks, observed that at certain densities steady flow breaks down. This critical concentration separates the flow into two types. Beyond this density the speed of vehicles falls in such a way that the flow decreases linearly with concentration. As density increases from the lower values, a point is reached at which the spacing becomes too small and drivers begin to hold larger than average spacing. This is when bunching (queuing) occurs and the flow begins to fall. This study showed that the disturbance (acceleration or deceleration) propagates at a speed between 8 and 11 mph; and suggested the possibility of not having a single regime, but 3 or 4 regimes of flow.

Edie & Bavarez (1967) studied stop-start traffic waves and indicated that the continuum theory based on fluid dynamics failed to explain the nature of periodic recurrence of stoppage waves or shock waves in tunnel traffic.

Fox & Lehman (1967) considered the driver of a following car either in a velocity-detecting mode or distance-detecting mode. The reaction time varies not only from driver to driver, but also with the detection mode. The reaction time was smaller when the driver was in the velocity detecting mode.
Forbes & Simpson (1968) reported that average time headways were longer downstream from a wave than they were upstream. They were also longer in the congested section than non-congested section, because the drivers were expecting congestion ahead.

Drake et al (1967) compared seven different models of speed density relationship of which 3 were discontinuous. In some respects the discontinuous models gave better conclusions about the estimated values. Edie’s model and the two regime model yielded the highest maximum flow and the highest estimate for jam density; and the two lowest values for the standard error of estimate. On the other hand they had the two lowest R-Squared values.

Leutzbach (1967) investigated traffic behavior in a bottleneck and compared the observed density patterns with those obtained from the theory of continuity (Lighthill & Whitham). There were discrepancies between observed and theoretical density patterns. The observed points on the speed density plane were situated in two distinctly separated areas of the K-U plot. There was no definite answer as to whether the theory of continuity could be used to describe actual traffic in bottlenecks.

Mika et al (1969) in an analysis of data from the John Lodge Freeway observed the phenomenon of rapid change of
speed and density versus time when traffic suffered from a kinematic disturbance. When speed and density was plotted as a function of time, an oscillatory mode of flow was observed. The onset of oscillatory behavior was abrupt, approximately at lane occupancy of 15%, and its amplitude varied from location to location by a factor of two. The frequency of the oscillation was 1/4 cycle per minute or a period of 4 minutes. The disturbance propagated upstream with a speed of about 16 mph, this is the same speed as would be predicted for propagation of shock waves from the theory of Lighthill & Whitham.

They also indicated that a simple linear relationship for K-U, and a parabolic for Q-K is difficult to justify when traffic is not in a steady state mode (lane occupancy of above 15%). The change from steady state mode occurs when lane occupancy exceeds a certain value. This value is different for the shoulder, center, and median lane.

El-Hosaini (1973) proposed a model for very high density and low speed (less than 7 mph) traffic assuming that traffic behaves like a compressible gas under piston action. The tail of the speed-density curve showed a rapid decrease in the vicinity of jam density. In this region the traffic was treated differently from congested flow.
Rackoff & Rockwell (1973); Eshler (1973) categorized the drivers into distance-detecting mode and velocity-detecting mode. If the rate of spacing change threshold was exceeded, the driver was considered in the velocity-detecting mode; otherwise in distance-detecting mode.

Athol (1965, 1973) reported a breakdown of traffic flow at less than maximum volume. When traffic operation was changing from uncongested mode to congested mode; changes of speed in the order of 10 mph were observed in a short period of time.

Hillegas et al (1974) investigated the discontinuity in the traffic flow relations and reported a distinct and discontinuous range of linear (un-congested) and non-linear (congested) flow regimes. A critical density value that separated the two operational states was 51.3 vpm. They also noticed that the flow density function is not the same for recovery and breakdown processes.

Treiterer and Research Staff’s (1968, 1969, 1970, 1972, 1973, 1975) investigation of dynamics of traffic flow revealed that a single regime linear speed-density model is not the best representitive of the traffic behavior on a freeway. A unique and accurate data set was obtained through aerial photograpy and was analyzed extensivly. Earlier investigations indicated that the multiregime linear
models provide better statistical fit to the K-U curve than a single regime linear model. The discontinuity points clearly could be identified. Different regions of flow were describe by linear models computed for the speed-density relation of that region.

They investigated the behavior of several platoons of vehicles going through a kinematic disturbance, and found that a three regime linear model would best describe the traffic as it goes from stable flow to disturbed flow, and finally to a compressed flow condition. The rate and the function when a platoon progresses from stable flow to a jam density situation are different from the rate and the function at which the platoon recovers from jam density to free flow condition.

When plots of speed-density-flow were made using separate data for the "before and within" and "within and after" disturbance situations, two loops were observed on the plots. This is attributed to the the existence of a phenomenon called hysteresis phenomenon in traffic flow (similar to hysteresis loop formed by loading and unloading of plastic materials).

Their further investigation of queue forming and queue releasing conditions supported the existence of hysteresis phenomenon in traffic flow. On the speed density curve for
different platoons there was a point at which a rapid speed change occurred while the density did not change proportionally. This point was not the same for all platoons. In the recovery phase when density reached a certain level a quick increase of speed was observed while the density remained almost constant. The point was located in the range of 50 to 70 vpm depending on the platoon and the condition of traffic.

This kind of abrupt change was observed in speed-density-volume curves, supporting the idea that speed-density-volume relations are not a smooth linear or a smooth parabolic curves; rather they have discontinuities.

Further investigation (Treiterer & Myers 1974) of hysteresis phenomenon was carried out by examining the acceleration-deceleration asymmetry. For individual vehicles the deceleration rates, at a given speed change value, are in general substantially greater than acceleration rates corresponding to the same amount of speed change. The difference is especially pronounced at higher speed change values. Moreover, the acceleration rates remain basically the same while the deceleration rates increased with increasing the speed change. Thus the drivers behavior during acceleration is different from that of during deceleration.
They separated the data into acceleration and deceleration phases, corresponding to a platoon leaving the disturbance and approaching the disturbance. For the separated data various speed-density models were fitted. Only Underwood’s model represented closely the speed-density relation for traffic leaving the kinematic disturbance (acceleration phase). However, for traffic approaching the disturbance (deceleration phase) Underwood’s, Greenberg’s, and a proposed model of the following form: \( A + B K + C K^2 \) provided adequate fit (R-Squared higher than 95%). Underwood’s model for acceleration and deceleration phases did not have the same parameters values. The plot of speed-density relation for recovery phase made two loops with that of breakdown. The point of intersection was in the neighborhood of 60 vpm. Traffic may recycle around one loop in a stop-and-go situation, or may go from one loop to another loop when suffering from a single kinematic disturbance.

Ceder & May’s (1976); and Ceder’s (1976) analysis of data supported Edie’s duel model approach. They used 50-60 vpm as the discontinuity range.

Ceder’s (1979) investigation of speed-density-flow data revealed that on the Q-K curve 5 phases can be identified, supporting Treiterer’s conclusions. The data points were separated into two groups to represent a two regime model;
one for congested and the second one for free flow. The distinction between the regimes was reported.

Roess, Linzer, McShane, and Pignataro (1980) reported observation of speed flow relationship with a considerable range of volume over which speed is relatively stable, or insensitive to volume, followed by a rapid deterioration of speed as volume approaches capacity (2000). They used speed as a secondary defining parameter and used density as the principal parameter defining level of service.

In summary, from the studies cited in this section one can conclude that the discontinuity in traffic flow relationship, or a jump in K-U curve, or dual mode behavior of traffic flow, or the hysteresis phenomenon should not simply be overlooked in microscopic simulation models. A microscopic simulation model should take into consideration the dual behavior of traffic especially if the model is used to study traffic flow at higher densities. So far the car following algorithms of freeway simulation models have not considered the effect of the dual mode behavior in model building. A new car following model has been developed to take care of this shortcoming of the previous models.

In the following sections the existing freeway simulation models will be briefly discussed. Later on, the development of the new model will be given in detail along with the features and the capabilities of the new model.
Early traffic simulation models earned a low level of credibility because they suffered from lack of realism. Two reasons were given (Lieberman, 1980): First, traffic phenomenon were very difficult to define and model because of their fine grained nature. Second, the human behavior and decision are hard to predict.

Present simulation models are more realistic, but their efficiency may be of concern. With the advancements in computer technology the criteria for efficiency is more related to the cost of human effort than the computer resource requirement. Comparing CPU time of different models for solving of identical problems does not really tell you much about the efficiency of the models. The level of details of computation and the amount of work needed to run the program has to be weighted in evaluation of the efficiency of a model. Lieberman (1980); and Radelat (1981) stated that the emphasis should be on human efficiency rather than machine processing efficiency.
Other concerns about a simulation model are documentation, update, support and maintenance, and compatibility with the need of users and the situation. Lieberman (1980) reported that many of the models are poorly organized and poorly documented because they were developed with realism as the primary goal. Also, a simulation model should be properly maintained and supported. The new findings should be incorporated into the model and the updating of the model should be a major concern for the developer.

Furthermore, a simulation model should be compatible with the need of a potential user, and the situation it would be applied. This would bring an undesirable characteristic of being limited to only a special application. A general purpose model on the other hand, might be too general and may not be able to simulate the situation accurately.

To resolve this conflict an integrated simulation system with several specific purpose models may be developed. Feasibility of development of an integrated simulation system to improve human and computational efficiency was investigated by the Office of Research of FHWA in 1975. An integrated traffic simulation system called TRAF was developed which can be used for evaluation and development of traffic control and traffic management policies (Lieberman, 1980).
The objectives of TRAF integrated model was to create a single source of traffic simulation program, with consistent input and output requirement and a single programming language, usable in macroscopic and microscopic levels, well documented, and efficient. The creation of TRAF does not involve new model development, but the enhancement of the best existing traffic simulation models.

The existing base models that are being integrated into TRAF are:

1. The NETSIM model. It is a microscopic network simulation model; extension of the UTCS-1 model, that in turn was incorporated and expanded version of DYNET model and TRANS model. NETSIM is one of the most widely used traffic simulation models for urban streets and arterials.

2. The FREFLO model. A macroscopic model for simulation of freeway traffic. FREFLO is the expanded, refined, and modified version of the MACK model (Goodwin et al, 1974). In this model the aggregate measures of traffic are calculated for sections of a road several hundred feet to a mile or so long.

3. The NETFLO model. A macroscopic model for urban network with three submodels—level I, level II, level III—with different levels of details.

4. The ROADSIM model. A microscopic model for two-lane, two-way rural roads. The MIR model which is renamed ROADSIM is basically the TWOWAF model.
5. The TRAFFIC model. It is an equilibrium traffic assignment model added to the collection of TRAF models.

FRESIM a microscopic traffic simulation model for freeway corridors traffic operation which is the freeway part of the INTRAS model was originally included in TRAF. However, in phase II of TRAF development (1983 version) it was dropped from the set of TRAF models.

In phase II of TRAF development the NETSIM, the NETFLO, and the TRAFFIC models were extensively validated.

The INTRAS model, of which part of it was used in phase I of TRAF, is a highly complex, multi-purpose, and a complete and updated microscopic simulation model. The integrated traffic simulation, INTRAS, model is a stochastic, microscopic model especially developed for freeway incident detection. The simulation procedure of the UTCS-1 is adopted and extended to the simulation of traffic on the freeway and parallel streets. INTRAS is a vehicle-specific time-stepping simulation designed to represent realistically the traffic and the traffic environment. INTRAS represents the roadway system by a set of links and nodes. Vehicles traveling on the links will be moved at one second intervals. The driver-vehicle pair is considered as an individual entity having its own characteristics. There are 5 types of vehicles. INTRAS has also a comprehensive freeway incident simulation procedure.
Another integrated traffic simulation model is the TRAFLO model (Lieberman, 1982). However it is a macroscopic model. TRAFLO is a system of integrated component traffic simulation submodels plus an equilibrium traffic assignment model. The models included in TRAFLO are: NETFLO, FREFLO, and TRAFFIC models. TRAFLO is the macroscopic part of the TRAF integrated system. Payne (1979) discussed the way traffic incidents are handled in FREFLO. It is either by reducing the number of available lanes, or by constraining the traffic volume past the incident site.

Other than the above mentioned integrated models, there are many other specific-purpose or general-purpose simulation models developed long before the integrated models. These models will not be surveyed here since there have been several excellent reviews before. Only these reviews will be outlined to carry through the discussion of model development.

In 1972, Goodwin examined in detail nine microscopic traffic flow simulation models for highways. The main components of each model were presented and summary of the features of each model were listed. Descriptions of most of the algorithms were given and tabulation of some of the parameters used in the models were included. Under identical conditions, the car following rules of these models
were tested. It was found that there are differences between the car following algorithms. Each model had something to offer, but, there were no conclusive results on which model was better. The models compared are:

1. Arizona Transportation and Traffic Institute Model (Richards, et al 1965)
2. The Connecticut Department of Transportation Expressway Model (Leland, 1970)
3. Midwest Research Institute Freeway Model (Kobett, 1968)
5. Northwestern University Lane Changing Model (Worral et al, 1969)
7. System Development Corporation Freeway Model (Warnshuis, 1972)
9. Texas Transportation Institute Freeway Merging Model (Buhr, et al)

This publication is a good condensed resource about the nine models and contain an excellent comparison of the car following rules of these models.
In 1974, Hsu & Munjal published a detailed discussion and critical review of 15 traffic flow simulation models associated with various aspects of highway vehicular traffic dynamics. The 15 models include the above mentioned 9 models plus:

1. Mikhalkin Freeway Simulation Model (Mikhalkin, 1971)
2. Georgia Model (Wildermuth, 1971)
3. SCOT Corridor Model (Wicks, 1972)
4. Priority Lane Model (May et al, 1972)
5. Aggregate Variable Models (Payne, 1972)
6. Aerospace Corporation Freeway Simulation Model (Harju, 1972)

There is no need to repeat the discussion or review the models here. These models are important because new models are benefited from certain features of them. For further information the original models or these reviews should be consulted.

In 1981, May described the existing models for freeway corridor analysis in five categories. He did not include the 15 models reviewed by Hsu & Munjal. A brief description of each model along with development and application of each category was given.

Also, in 1981, Gibson classified 104 distinct computer models into intersection models, arterial models, network
models, freeways, and corridor models. A brief description of some of the models were given by the author. The freeway models class contained 18 different models.

Finally, in 1982 Byrne et al gave detailed information about 10 mostly used simulation models in the Handbook Of Computer Models For Traffic Operation Analysis; and a synopsis of over 100 computer models in the technical appendix of the handbook.

Available models are not developed to handle at fine levels recurrent bottlenecks on the freeways. They may not represent realistically the behavior of traffic in the bottlenecks which is different from that of normal traffic. Furthermore, the dual behavior of traffic has not been taken into consideration, rather a single model for the entire range of density is used. A reliable and realistic simulation model will be developed to take into consideration the dual mode behavior of traffic and its behavior in the bottlenecks. This model will be used for simulation of traffic flow through a bottleneck.
Appendix D

PARTIAL VERIFICATION OF CARSIM

Since CARSIM is new and has not been used before, its ability to perform as intended had to be examined. As a part of the validation effort, acceleration and deceleration patterns, velocity patterns, trajectories of vehicles, and vehicles spacings were investigated. Details of partial verification are discussed in the following sections. More discussion is given in sensitivity and validation sections.

In calibration of the INTRAS car following model Long Island data set and mainly The Ohio State data set were used. The Ohio State University's data indicated that ten feet should be added to $L+kV$ for better representation of intervehicle spacing, also it indicated a $k$ value of 0.93 and lag time of 0.2 or 0.3 seconds.

As a part of validation of INTRAS simulation model trajectories of a platoon of 23 vehicles were plotted using simulation results and actual O.S.U. data. The actual data
show the behavior of the platoon through a kinematic dis­
turbance in which traffic flow goes from lower density and
higher speed to very high density and very low speed and
back to lower density and higher speed. For O.S.U. data
this time is normally over 100 seconds, however, in the
trajectories plots generated by INTRAS, the graph is only
given for the first 50 seconds, which covers the first
phase of going thru a kinematic disturbance. The second
phase which is recovering from very high density and very
low speed is not shown on the plot. This phase is as
important as the first phase and it may be even more impor­
tant.

In the validation of CARISM a complete phase of going
thru a kinematic disturbance is used. Four different pla­
toons of vehicles all suffering from a kinematic distur­
bance are selected from the O.S.U. data and the results of
simulation for these situations are compared with the actu­
al data. The platoons used for this purpose are 123, 123X,
126, and 127. More on the results of validation will be
given in the model validation section. Here only those
related specifically to car following will be discuceed.
The following validation procedures are specifically for
the car following algorithm(CARSIM).
Various artificial disturbances are induced to a platoon of 15 vehicles and their effects at microscopic and macroscopic levels are discussed in the subsequent sections.

D.1 MICROSCOPIC LEVEL

At this level the effect of the disturbances on individual vehicles trajectories, speed, and acceleration is discussed. The propagation of the disturbance along the line of vehicles at high and low volumes are presented. The disturbances are: a regular disturbance, an emergency stop, and a stop-and-go situation.

D.1.1 REGULAR DISTURBANCE

The leader of the platoon is asked to decelerate at a rate of 6 ft/sec/sec for 6 seconds; following with a 0 ft/sec/sec deceleration for 3 seconds, and finally an acceleration of 6 ft/sec/sec for 6 seconds; the regular (-6, 0, +6) disturbance. All vehicles were traveling at a speed of 80.66 ft/sec (55mph) before the disturbance. In validation of the car following algorithm of INTRAS this disturbance was used.

Figure 101 shows the effect of the disturbance on the trajectories plot of the platoon when traffic volume is 1200 vph. Since traffic volume is lower, the effect does
not reach the fifth vehicle in the platoon which is traveling with a larger space headway.

However, when the traffic volume is 1800 vph, the disturbance reaches far beyond the first six cars. As it can be seen from Figures 102 and 103, at least the first 10 vehicles are affected by the regular disturbance (-6, 0, +6). This is an indication of the fact that in dense traffic, a disturbance propagates deeper down the line of vehicles.

The effect of the disturbance on speed patterns are shown in Figures 104-106. When traffic volume is 1200 vph only the first 4 vehicles suffered from the disturbance, Figure 104. As volume increases to 1800 vph more cars are affected, Figure 105. In fact over 10 cars are forced to decelerate and to accelerate, Figure 106.

Figures 104-106 show two important points. First, the drivers would make higher speed reduction in a dense traffic than in a light traffic due to shorter headways. Figures 104 and 105 show the reaction of the same drivers at two different volume levels. Second, the effect of a small disturbance would dissipate along the line of vehicles with a higher rate at low volume traffic. Figure 106 shows how the effect of the disturbance is deteriorated through the platoon. The 10th vehicle in the platoon is affected very
Figure 101: Trajectories of first 6 cars (regular disturbance, low volume). Effect of the regular disturbance on trajectories plot for the first six cars of a 15-car platoon. Traffic volume is 1200, units are feet and seconds.
Figure 102: Trajectories of first 6 cars (regular disturbance, high volume). Effect of the regular disturbance on trajectories plot for the first six cars in a 15-car platoon. Traffic volume is 1800, units are feet and seconds.
Figure 103: Trajectories of cars (regular disturbance, high volume): Effect of the regular disturbance on trajectories plot for every third car in a 15-car platoon. Traffic volume is 1800, units are feet and seconds.
Figure 104: Speeds of first 6 cars (regular disturbance, low volume). Effect of the regular disturbance on speed change patterns for first 6 cars in a 15-car platoon. The volume is 1200, units are ft/sec and seconds.
Figure 105: Speeds of first 6 cars (regular disturbance, high volume). Effect of the regular disturbance on speed change patterns for first 6 cars in a 15-car platoon. The volume is 1800, units are ft/sec and seconds.
Figure 106: Speeds of cars (regular disturbance, high volume). Effect of the regular disturbance on speed change patterns for every third car in a 15-car platoon. The volume is 1800, units are ft/sec and seconds.
little compared to the 3rd vehicle. The disturbance does not reach vehicle 13 or after.

The consequence of the regular \((-6, 0, +6)\) disturbance on acceleration or deceleration of the following vehicles are presented in Figures 107-109. Comparison of Figures 107 and 108 revealed that the magnitude of acceleration and deceleration decreases along the platoon of vehicles. For example, the 4th vehicle decelerates and accelerates with a lower rate than the 2nd vehicle.

Furthermore, when the volume is 1200 vph the drivers react later compared to the condition when the volume is 1800 vph. The deceleration rates for the first 4 cars in both volume levels are almost the same. It is a little less for congested traffic, because the drivers are alert and expecting a congestion, but, at the 1200 volume level they are not alert to the adverse traffic conditions.

Figures 101-109 reproduce the desirable patterns and the non-oscillatory behavior expected from a reasonable car following model.
Figure 107: Acceleration of first 6 cars (regular disturbance, low volume). Effect of the regular disturbance on acceleration/deceleration patterns for the first 6 cars in a 15-car platoon. The volume is 1200, the units are ft/sec/sec and seconds.
Figure 108: Acceleration of first 6 cars (regular disturbance, high volume). Effect of the regular disturbance on acceleration/deceleration patterns for the first 6 cars in a 15-car platoon. The volume is 1800, the units are ft/sec/sec and seconds.
Figure 109: Acceleration of cars (regular disturbance, high volume). Effect of the regular disturbance on acceleration /deceleration patterns for every third car in a 15-car platoon. The volume is 1800, the units are ft/sec/sec and seconds.
D.1.2 EMERGENCY STOP

The lead car in the platoon of 15 vehicles is directed to stop; as if it were an emergency situation. The maximum deceleration rate is 16 ft/sec/sec.

Figures 110-113 exhibit the trajectories of vehicles as they decelerate in response to the emergency deceleration of the lead car. The following vehicles decelerate at smaller rates than the lead car. When vehicles are stopped the spacing between them is less than when they were moving.

When volume is increased from 1200 vph to 1800 vph, the time a certain vehicle in the platoon will come to a complete stop is reduced. For instance the 15th car came to a complete stop 8 sec. earlier when the volume was 1800 vph. This indicates that the following simulated driver reacts to the traffic situation and decelerates accordingly. Thus, the time the leader of the platoon starts deceleration does not set the deceleration time for every vehicle in the platoon.

Figures 114-117 demonstrate the speed pattern developed under an emergency condition. Two important observations should be mentioned. First, the slope of the lines are different which is an indication of slower deceleration by
Figure 110: Trajectories of first 6 cars (emergency stop, low volume). Propagation of a disturbance, caused by an emergency stop of the lead car, for the first 6 cars in a 15-car platoon. The volume is 1200, the units are feet and seconds.
Figure 111: Trajectories cars (emergency stop, low volume). Propagation of a disturbance, caused by an emergency stop of the lead car, for every third car in a 15-car platoon. The volume is 1200, the units are feet and seconds.
Figure 112: Trajectories of first 6 cars (emergency stop, high volume). Propagation of a disturbance, caused by an emergency stop of the lead car, for the first 6 cars in a 15-car platoon. The volume is 1800, the units are feet and seconds.
Figure 113: Trajectories of cars (emergency stop, high volume). Propagation of a disturbance, caused by an emergency stop of the lead car, for every third car in a 15-car platoon. The volume is 1800, the units are feet and seconds.
following vehicles. The time interval between starting of deceleration by any car and that of the lead car is much shorter than the time interval when the same car comes to a complete stop. For example, for 1800 vph, the 15th car starts deceleration 13 sec. after the first car, but came to a complete stop 30 sec. after the first car had stopped. When volume is 1200 the corresponding numbers are 28 seconds and 38 seconds.

The second observation is that, as the volume increases the following vehicles react sooner but not stronger to the deceleration of the leader. The graph shows the time of initiation of deceleration for every car. The negative slope of the lines at 1800 volume level is larger than at the 1200 volume level, as it is expected for higher densities.

Figures 118-121 illustrate the deceleration of the following car when the lead car stops in an emergency situation. The following car decelerates at a slower rate than the leading car. Moreover, the drivers slow down faster at the early phase of deceleration. This non-oscillatory and damping pattern is very desirable and replicates real world situations.
Figure 114: Speeds of first 6 cars (emergency stop, low volume). Speed change pattern for the first 6 cars in a 15-car platoon when the lead car makes an emergency stop. Volume is 1200 and units are in ft/sec and seconds.
Figure 115: Speeds of cars (emergency stop, low volume).
Speed change patterns for every third car in a 15-car platoon when the lead car makes an emergency stop. Volume is 1200 and units are ft/sec and seconds.
Figure 116: Speeds of first 6 cars (emergency stop, high volume). Speed change patterns for the first 6 cars in a 15-car platoon when the lead car makes an emergency stop. Volume is 1800 and units are ft/sec and seconds.
Figure 117: Speeds of cars (emergency stop, high volume). Speed change patterns for every third car in a 15-car platoon when the lead car makes an emergency stop. Volume is 1800 and units are ft/sec and seconds.
Figure 118: Deceleration of first 6 cars (emergency stop, low volume). Deceleration patterns for the first 6 cars in a 15-car platoon when the lead car makes an emergency stop. Volume is 1200, and the units are ft/sec/sec and seconds.
Figure 119: Deceleration of cars (emergency stop, low volume). Deceleration patterns for every third car in a 15-car platoon when the lead car makes an emergency stop. Volume is 1200, and the units are ft/sec/sec and seconds.
Figure 120: Deceleration of first 6 cars (emergency stop, high volume). Deceleration patterns for the first 6 cars in a 15-car platoon when the lead car makes an emergency stop. Volume is 1800, and the units are ft/sec/sec and seconds.
Figure 121: Deceleration of cars (emergency stop, high volume). Deceleration patterns for every third car in a 15-car platoon when the lead car makes an emergency stop. Volume is 1800, and the units are ft/sec/sec and seconds.
D.1.3 STOP-AND-GO

The third kind of disturbance is induced by asking the lead car to decelerate to a full stop at maximum allowable rate, wait for 9 seconds, and start moving again. The lead car employs a 16 ft/sec/sec deceleration rate.

Figures 122-125 presents the way stop-and-go disturbances are propagated through the line of vehicles. CARSIM handles stop-and-go situations in a very realistic manner. The order the vehicles move and their spacing before and after disturbances are very similar to the actual trajectory plots. A few remarks about these graphs are imperative.

First of all, the following vehicles come to a complete stop at a slower rate than the leading cars. A smoother curve near zero density clearly illustrates this point at both volume levels.

The second remark; the spacings between vehicles before the disturbance and after it are not the same. Before the disturbance vehicles are either decelerating according to the car following rule or traveling at a speed limit of 80.66 fps. However, after the disturbance the vehicles use the same acceleration rates to reach the speed limit. This is the reason why the spacing between vehicles after going thru a kinematic disturbance is the same for the first six
Figure 122: Trajectories of first 6 cars (stop-and-go condition, low volume). Trajectories for the first 6 cars in a 15-car platoon in a stop-and-go situation. Volume is 1200, the units are feet and seconds.
Figure 123: Trajectories of cars (stop-and-go condition, low volume). Trajectories for every third car in a 15-car platoon in a stop-and-go situation. Volume is 1200, the units are feet and seconds.
Figure 124: Trajectories of first 6 cars (stop-and-go condition, high volume). Trajectories for the first 6 cars in a 15-car platoon in a stop-and-go situation. Volume is 1800, the units are feet and seconds.
Figure 125: Trajectories of cars (stop-and-go condition, high volume). Trajectories for every third car in a 15-car platoon in a stop-and-go situation. Volume is 1800, the units are feet and seconds.
vehicles in both volume levels, Figures 122 and 124. This equal spacing is true only for the first several vehicles, but not for all vehicles in the platoon. Figures 123 and 125 show that the traffic volume influences the spacing even after the disturbance. For example, spacing between the 10th and 13th vehicle in 1800 vph level is much greater than at 1200 vph level. This is due to the fact that vehicle 13 in 1200 vph level did not suffer from the disturbance as much as vehicle 13 in 1800 vph volume level, as it is obvious from the trajectory plots.

The third remark, the disturbance propagates longer as the traffic volume increases. Figures 123 and 125 illustrate this point clearly. Examination of the trajectories of the two plots disclosed that when the volume is 1200 vph vehicles 10-15 went through the kinematic disturbance without coming to a full stop. At lower volume levels, the effect of the disturbance is decreasing along the line of vehicles and finally is diminished. However, at 1800 traffic flow level all vehicles in the platoon were forced by the disturbance to come to a standstill. At higher flow levels the effect of the disturbance compelled even the 15th driver to stop the car for several seconds.

The fourth remark, duration of the disturbance is longer at higher volume, Figure 125 than lower volume, Figure 123.
The leaders of the platoons at both flow levels, started decelerating at the same time. At the lower flow level, at 80 seconds the vehicles have gone through the disturbance and are traveling at moderate speed. However, at the higher flow level at the same time, some vehicles are still in a stopped position.

Figures 126-129 represent the results of the disturbance on speed of following vehicles. Vehicles did not use the same deceleration rates because it is determined individually for each vehicle from the car following algorithm. However, they use the same rates when accelerating from a standstill position, Figures 126, 128 and 129.

Comparison of Figures 127 and 129 reveal the effect of volume on the number of vehicles affected by the disturbance and the severity of the affect. At higher flow level, all vehicles are stopped, but at lower flow level only some are forced to halt and others made some speed changes.

Another observation from Figures 126-129 is the time each vehicle is affected by the disturbance, and how long the vehicle is under the influence. At higher flow level the influence lasts longer than the lower flow for the same vehicle.
Figure 126: Speeds of first 6 cars (stop-and-go condition, low volume). Speed change patterns in a stop-and-go situation for the first 6 cars in a 15-car platoon. Volume is 1200, the units are ft/sec and seconds.
Figure 127: Speeds of cars (stop-and-go condition, low volume). Speed change patterns in a stop-and-go situation for every third car in a 15-car platoon. Volume is 1200, the units are in ft/sec and seconds.
Figure 128: Speeds on first 6 cars (stop-and-go condition, high volume). Speed change patterns in a stop-and-go situation for the first 6 cars in a 15-car platoon. Volume is 1800, the units are ft/sec and seconds.
Figure 129: Speeds of cars (stop-and-go condition, high volume): Speed change patterns in a stop-and-go situation for every third car in a 15-car platoon. Volume is 1800, the units are ft/sec and seconds.
Figures 130-133 show the deceleration and acceleration patterns at 1200 and 1800 flow levels. It can be seen that the effect of disturbance is not amplified but is propagated. This is a very desirable feature in a car following model.

Figures 130-133 exhibit a decreasing trend in the amplitude of the disturbance along the line of vehicles in the platoon. The reduction is less at higher flow level as it is in the real world traffic. It should be mentioned that all acceleration and deceleration values employed by the following vehicles in response to severe disturbance are very reasonable.

In summary CARSIM performed very well under various undesirable conditions and reproduced the behavior of actual traffic in a very realistic manner.
Figure 130: Acceleration of first 6 cars (stop-and-go condition, low volume). Acceleration/ deceleration patterns in a stop-and-go situation for the first 6 cars in a 15-car platoon. Volume is 1200, and the units are ft/sec/sec and seconds.
Figure 131: Acceleration of cars (stop-and-go condition, low volume). Acceleration/deceleration patterns in a stop-and-go situation for every third car in a 15-car platoon. Volume is 1200, and the units are ft/sec/sec and seconds.
Figure 132: Acceleration of first 6 cars (stop-and-go condition, high volume). Acceleration/deceleration patterns in a stop-and-go situation for the first 6 cars in a 15-car platoon. Volume is 1800, and the units are ft/sec/sec and seconds.
Figure 133: Acceleration of cars (stop-and-go condition, high volume). Acceleration/deceleration patterns in a stop-and-go situation for every third car in a 15-car platoon. Volume is 1800, and the units are ft/sec/sec and seconds.
The effects of the kinematic disturbances on aggregate traffic parameters, such as average speed, average density, volume, and the average headway are discussed at low and high traffic volume levels when traffic is subjected to two different kinematic disturbances. The volumes are 1200 vph and 1800 vph. The disturbances are a regular disturbance, and an emergency deceleration to near a complete stop.

The regular disturbance is when the lead of the platoon decelerates at a rate of 6 ft/sec/sec for 6 sec., followed by a 0 ft/sec/sec for 3 seconds, and accelerates at a rate of 6 ft/sec/sec for 6 seconds.

The emergency deceleration is induced by the lead car which decelerates for 5 seconds at maximum deceleration rate of 16 ft/sec/sec and then accelerates according to the car following logic to reach a speed of 80.66fps.

D.2.1 REGULAR DISTURBANCE

The average speed of the platoon when suffering from a kinematic disturbance is shown in Figure 134 and 135 at low and high volume levels. The average speed dropped a maximum of 9.75 fps at 1800 volume level, but only 6.33 fps at 1200 volume level. This indicates that the disturbance forced more vehicles to decelerate at a higher volume then at a lower volume.
Figure 134: Average speed of platoon (regular disturbance, low volume). Effect of the regular disturbance on the average speed of a platoon of 15 cars. Volume is 1200, and the units are in ft/sec and seconds.
Figure 135: Average speed of platoon (regular disturbance, high volume). Effect of the regular disturbance on the average speed of a platoon of 15 cars. Volume is 1800, and the units are ft/sec and seconds.
The time for the platoon to recover from the disturbance and get back to the speed before the disturbance is longer at high volume traffic than low volume. At high volume it takes 33 seconds for the platoon to reach an average speed of 80.66 fps, Figure 135. However, at low volume traffic only 21 seconds, Figure 134. Thus, the disturbance not only propagates more at higher volume, but also it takes a longer time to dissipate. The major difference is in the acceleration phase; it takes almost twice as long for the platoon to reach the speed of 80.66 fps at higher volume (22 seconds compared to 12 seconds).

The average density of the platoon drops 6.79 units at a higher volume, but 3.05 units at lower traffic volume; Figures 136 and 137. The average density reaches constant values after 15 seconds for both volume levels. The constant density is reached when the leader of the platoon accelerates to the speed of 80.66 fps; and remains constant because the spacing between the first and the last car does not change anymore.

Plot of trajectories of vehicles revealed that this disturbance did not propagate up to the last vehicle in the platoon. While the other vehicles are affected by the disturbance and their spacings changes, the spacing between the first and the last vehicle does not change after the
Figure 136: Density of platoon (regular disturbance, low volume). Effect of the regular disturbance on the density of a platoon of 15 cars. Volume is 1200, and the units are ft/sec and seconds.
Figure 137: Density of platoon (regular disturbance, high volume). Effect of the regular disturbance on the density of a platoon of 15 cars. Volume is 1800, and the units are ft/sec and seconds.
lead car reaches the speed of 80.66 fps. Since the density of the platoon is computed only from the space headway of the first and the last vehicle, Therefore, it does not represent the changes that are occurring within the platoon. Procedures to improve computation of platoon density are suggested which puts less emphasize in the vehicles in the two end of the platoon than vehicles in the middle.

Average headways are computed using two different procedures: a simple procedure, and an exact method.

The simple procedure is based on traffic volume which is the product of the speed and the density computed for every update time interval (1 second). The average time headway is simply the reciprocal of the volume: the equation is average headway = 3600/traffic volume. Can this simple procedure be used for average headway computation? The reason one should be concerned is the fact that the speeds are computed from the length measurements (versus the point measurement); an the average speed and the density are computed for a very short period of time (1 second).

The exact procedure is computing individual headways from spacing between vehicles, for all vehicles, then finding the arithmetic mean of them:

average headway = (h1+h2+h3+...+hn)/n
This procedure is exact because in every update time interval all headways are determined based on the actual spacing between vehicles and their actual speeds.

These two procedures will be compared and the effect of the kinematic disturbance on the average headways computed from the two methods will be discussed. Figures 138 and 139 exhibit the difference between the average headways at low and high traffic volumes.

When the traffic flow is in a steady state the average headways computed from both methods are equal, both at low and high volume levels the difference is less than thousandth of a second. On Figures 138 and 139 the steady states correspond to the constant headways. However, when the vehicles are in acceleration or deceleration mode, the average headways from the two procedures show considerable differences. As traffic approaches the steady state the difference decreases. Maximum difference occurs at the beginning of the acceleration phase. At high volume the average headway computed from the simple procedure is 0.04431 seconds higher than the average headway from the exact method, this happened at time 55; and at low traffic volume it is 0.08799 seconds higher at time 53. The average headways computed from the simple method is always higher than the exact method.
Figure 138: Headways of platoon (regular disturbance, low volume). Average headways computed using a simple and an exact method when a platoon of 15 cars going through a regular disturbance. Volume is 1200, units is seconds.
Figure 139: Headways of platoon (regular disturbance, high volume). Average headways computed using a simple and an exact method when a platoon of 15 cars going through a regular disturbance. Volume is 1800, unit is seconds.
The general shape of the graphs is the same; there is a peak after an initial constant value and there is a rapid decrease to a lower constant value. At the early stage of deceleration the average headway increases because the volume decreases due to the speed reduction. Once the deceleration phase is over the decrease in average headways starts and lasts until the vehicles reach a constant speed. The amount of decreases at both volume levels are 0.287 seconds. The decrease is expected to be the same at all volume levels as long as the disturbance does not effect the last car in the platoon. The proof of this lies in the definition of average headway at a steady state traffic condition.

Traffic volume is also computed by two different procedures corresponding to the headway computation methods. First, volume is a product of speed and density. Second, volume is reciprocal of average time headway which is computed as the arithmetic mean of individual headways. Figures 140 and 141 show the volumes and their differences at various phases.

Higher volume levels are reached after the disturbance due to higher densities. When traffic is in a steady state, both at high and low densities, the volume is constant and the same for both procedures (the difference is
Figure 140: Volumes of platoon (regular disturbance, low volume). Traffic volume computed using a simple and an exact method when a platoon of 15 cars going through a regular disturbance. Volume is 1200, unit is vph.
Figure 141: Volumes of platoon (regular disturbance, high volume). Traffic volume computed using a simple and an exact method when a platoon of 15 cars going through a regular disturbance. Volume is 1800, unit is vph.
less than 0.5 car). However, when vehicles are going through a speed change, different values are obtained from the two procedures. Maximum volume difference is 47.31 which occurs at 54 seconds at low volume. At high volume, the difference is 51.32 and it happens at 55 seconds. The volumes computed from average headways are always higher than the one computed from the product of speed and density.

Maximum volume after the disturbance is 2358 vph at high volume traffic and 1538 vph at lower volume. It should be mentioned that in real world situations maintaining such a high volume for a long period is not possible. However, for a platoon of vehicles traveling as a group this volume is not unreasonable.

D.2.2 EMERGENCY DECELERATION

The leader of the platoon of 15 cars decelerates at a rate of -16 ft/sec/sec for 5 seconds, and accelerates to a speed of 80.66 fps according to the car following model.

The average speed and the average density of the platoon showed a pattern very similar to that of the regular disturbance, Figures 142-145. The difference in magnitude and duration are summarized in Table 10.
Table 10
traffic parameters
Traffic parameters when a platoon of 15 cars suffers from kinematic disturbances at high and low traffic volumes.

<table>
<thead>
<tr>
<th>traffic parameters</th>
<th>regular disturbance</th>
<th>emergency deceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VOLUME</td>
<td></td>
</tr>
<tr>
<td></td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>minimum average speed; fps</td>
<td>74.33</td>
<td>70.91</td>
</tr>
<tr>
<td>time to decelerate to the min speed; sec.</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td>time to accelerate to speed of 80.66; sec.</td>
<td>12</td>
<td>22</td>
</tr>
<tr>
<td>total recovery time; sec</td>
<td>21</td>
<td>33</td>
</tr>
<tr>
<td>maximum density vpm</td>
<td>27.96</td>
<td>42.89</td>
</tr>
<tr>
<td>increase in density</td>
<td>3.05</td>
<td>6.79</td>
</tr>
<tr>
<td>time to reach maximum density</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>maximum headway difference from the two methods</td>
<td>0.08799</td>
<td>0.04431</td>
</tr>
<tr>
<td>decrease in headway after the disturbance</td>
<td>0.287</td>
<td>0.287</td>
</tr>
<tr>
<td>maximum volume difference from the two methods</td>
<td>47.31</td>
<td>51.32</td>
</tr>
</tbody>
</table>

Comparison of the values in the table reveals that a
Figure 142: Speed of platoon (emergency deceleration, low volume). Effect of the emergency deceleration on the average speed of a platoon of 15 cars. Volume is 1200, and the units are in ft/sec and seconds.
Figure 143: Speed of platoon (emergency deceleration, high volume). Effect of the emergency deceleration on the average speed of a platoon of 15 cars. Volume is 1800, and the units are in ft/sec and seconds.
Figure 144: Density of platoon (emergency deceleration, low volume). Effect of the emergency deceleration on the density of a platoon of 15 cars. Volume is 1200, and the units are in ft/sec and seconds.
Figure 145: Density of platoon (emergency deceleration, high volume). Effect of the emergency deceleration on the density of a platoon of 15 cars. Volume is 1800, and the units are in ft/sec and seconds.
short but severe kinematic disturbance propagates much faster and deeper along the line of vehicles than a regular disturbance. As a result the 5 second emergency deceleration reduced the average speeds more than the 6 seconds deceleration combined with the 3 seconds constant speed, at low and high traffic volume.

The time to recover from the emergency deceleration is longer, 17 and 27 for low and high volume compare to 12 and 22 in the regular disturbance situation.

The average density increased more, 4.80 and 11.05, but not at a fast rate. It took 16 seconds to reach the maximum density at both volume levels.

Headways computed from the two procedures were different at non-steady state conditions, Figures 146 and 147. Maximum difference between headways was higher than that for the regular disturbance case. At lower volume this difference was more than that at higher volume for both disturbances. The decrease in the average headways before and after the disturbance were 0.425 seconds for both volume levels. This is expected since the disturbance does not effect the speed of the last car in the platoon. As long as the disturbance does not reach the last vehicle in the platoon the decrease in average headway is independent of traffic volume. This can be simply proven as:
\[ H_b = \frac{3600}{Q_b} = \frac{3600}{K_b} U_b, \quad \text{for before,} \]

and

\[ H_a = \frac{3600}{Q_a} = \frac{3600}{K_a} U_a, \quad \text{for after} \]

for after the disturbance. Since the steady state speed before and after the disturbance is the same, we use \( U \) for both before and after. From these equations one can write:

\[ H_b - H_a = \frac{3600(1/K_b - 1/K_a)}{U} \]

where \( 1/K \) gives the average space headway between vehicles, let us call them \( S_b \) and \( S_a \) thus

\[ H_b - H_a = \frac{3600(S_b - S_a)}{U} \]

The average steady state spacing after the disturbance \( S_a \) is equal to:

\[ S_a = S_b - \frac{d}{n} \]

where \( d \) is the distance the lead vehicle falls behind due to the disturbance; and \( n+1 \) is the number of cars. Substituting \( S_a \) in the above equation results in

\[ H_b - H_a = \frac{(3600/U)(d/n)}{U} \]

For example, in case of the regular disturbance the lead car falls behind 324 feet; computed as

\[ \frac{1/2(6)(6*2)+2(36)+1/2(6)(6*2)}{324} = 1/2(6)(6*2)+2(36)+1/2(6)(6*2) = 324 \]

\[ H_b - H_a = \frac{3600/55}{(5280*14)} = 0.287 \text{ sec.} \]

The results from simulation confirms the computation.
Figure 146: Headways of platoon (emergency deceleration, low volume). Average headways computed using a simple and an exact method when a platoon of 15 cars going through an emergency deceleration. Volume is 1200, the units are in seconds.
Figure 147: Headways of platoon (emergency deceleration, high volume). Average headways computed using a simple and an exact method when a platoon of 15 cars going through an emergency deceleration. Volume is 1800, the units are in seconds.
The volume computed by the two procedures are different as much as 66.08, and 62.57 at low and high volume traffic Figures 148 and 149. Maximum volume difference occurred at the time when maximum headway difference happened, or one second later. At high volume, when vehicles are decelerating the difference between the volumes computed from the two procedures is very small; see Figure 149.
Figure 148: Volumes of platoon (emergency deceleration, low volume). Traffic volume computed using a simple and an exact method when a platoon of 15 cars going through an emergency deceleration. Volume is 1200, the units are in seconds.
Figure 149: Volumes of platoon (emergency deceleration, high volume). Traffic volume computed using a simple and an exact method when a platoon of 15 cars going through an emergency deceleration. Volume is 1800, the units are in seconds.
Appendix E

BOTTLENECK TRAFFIC OPERATIONS

A bottleneck is a local area of highway with high density and low speed traffic operation. It can be generated by an undesirable geometric design features of the freeway, incidents, poor operational policy, a slow moving vehicle blocking the fast moving vehicles, or any abrupt deceleration propagating backward into the upstream traffic. Bottlenecks have a disrupting effect on traffic, they increase the accident potential, cause higher density, lower speed and volume, reduce the level of service.

Bottleneck control can eliminate, or reduce considerably, the detrimental effects and can provide a safer and more efficient operation. There have been numerous investigations on the effectiveness of ramp control to eliminate the formation of bottlenecks; Ramp metering has its merits and drawbacks but it is not our intention to discuss it here.

Early works on traffic control emphasized obtaining maximum flow which was identified as the vertex of the tradi-
tional Q-K curve. Traffic operation cannot be maintained at this peak level because the maximum flow point is not a stable traffic flow condition. In recent years, the emphasis has shifted towards preventing congestion and operating at a stable level less than the maximum flow keeping the operation mode on uncongested side of Q-K curve.

In the following sections the effect of incidents on freeway quality of service and procedure to improve traffic operation will be discussed.

E.1 EFFECT OF INCIDENTS

A study of the characteristics of random freeway incidents (accident and disability) in a 3.2 mile section of the John Lodge Freeway (6 and 8 lanes) in Detroit, by DeRose (1964) over a period of 255 days reported an occurrence of 9.8 incidents per million vehicle miles. The average number of incidents per weekday was 3.64 and for Friday was 4.24. The average duration of an incident was 5.24 minutes. The duration of 50% of all incidents were 3 minutes or less. On the other hand 12% of the incidents lasted 10 minutes or more. The duration was terminated when the incident was moved to the shoulder lane, or when the freeway traffic resumed movement.
Another study by Goolsby (1971) on a 6.5 mile section of the Gulf Freeway in Houston (5 lanes) over a period of 2 years reported approximately 4.5 e lane blocking incidents each weekday during daylight. The average noninjury accident affected the traffic for approximately 45 minutes. This time was divided into the following steps: a) detection and reporting of an accident to police approximately 1 minute (using television); b) time till the police arrives approximately 12 minutes; c) clearing the accident approximately 7 minutes; and d) investigation of the accident 25.6 minutes. It is reported that approximately 47% of the incidents were noninjury accidents and 48% were stalls. For stalls, it took 9.4 minutes for the police to arrive and 8.9 minutes to remove the stalled vehicle.

The study also reported that, one lane blockage by a minor accident or stall reduced flow by 50% even though the physical reduction was only 33%. An accident that blocked two lanes reduced the flow by 79%. When the accident was on the freeway shoulder a reduction of 33% of normal flow was reported due to "gapers-block". Note that incident reporting time in this report was 1 minute because of television surveillance. Reporting time by motorists was found to be about 5 minutes.
Average time the accident effects freeway traffic can be reduced by moving the involved vehicle off the freeway as soon as possible. Sixteen accident investigation sites were located along a 6 mile section of the Gulf Freeway and accidents were moved from the freeway to these sites for police investigation. Pittman and Zenheiser (1973) reported that when the accidents were moved from the freeway to one of these sites, the freeway traffic was only affected for 16 minutes. The police investigation took place off the freeway at the designated accident investigation site. By doing this the 25 minute average investigation time was eliminated and operation returned to normal more rapidly; the average accident affected freeway traffic for 41 minutes. Furthermore, there was a reduction of the percentage of secondary accident.

E.2 PLATOONING

Any incident that induces a kinematic disturbance to traffic flow would cause the operation to shift from an uncongested and stable condition to a congested and less stable mode. Even after removing the physical blockage off the freeway, the disrupting effect of the incident will still influence the traffic operation. One sign of congestion is the clustering or platooning tendency of vehicles with high density. Montroll (1961) defined cluster as
a line of vehicles with separations of less than 100 feet from each other in a traffic with speeds in the order of 25 mph. The mean size of a cluster increases from 2.6 to 4.2 when density increases from 50 to 60 vpm.

Models of bunch size or cluster size were compared by Taylor, Miller, and Ogden (1974). They proposed a more general model (2 parameters Miller) which provided a much closer fit to the observed data from two-way roads. They also investigated the notion of an exponential inter-bunch headway, and concluded that there was a good agreement between the observed inter-bunch headway and an exponential function.

There is not an agreement on when a vehicle is in platoon. Time headway or spacing is used to distinguish platooning vehicles from those not in queue. Edle (1963) used different values of spacing as the criteria for identification of platooning vehicles when spacing is changed to time headway approximately 4-5 seconds is obtained. Montroll (1961) used a distance of 100 feet for clustering cars with speed of about 25 mph. Time headway corresponding to this is approximately 2.7 seconds. Daou (1964) gave a range from 2 to 4.5 seconds depending on speed and study site, and Miller (1961) value of 8 seconds and relative speeds in the interval of (-3 mph, 6 mph) as criteria for separating
platooning and non-platooning vehicles. Phal and Sands (1971) studied the interaction process of vehicles and for different lanes and flow rates found different time headways below which the interaction process will take place. These time headways range from 2.5 to 4.3 seconds.

E.3 KINEMATIC DISTURBANCE

Strong interaction between vehicles is manifested as kinematic disturbance that can propagate backward in upstream traffic and cause disruption of normal flow. Speed of this propagation is discussed in Lighthall & Whitham (1955), and Munjal and Pipes (1971) papers. Also numerical values of 16 ft/sec (Mika et al 1969), and 8-11 mph (Franklin 1967) is reported.

Myers (1973) investigated wave behavior and made comparison of theoretical and observed value speeds. He concluded that speed of propagation of waves carrying low speed value is basically constant, and the speed of the waves carrying a given space mean speed value is definitely different for acceleration and decelerating conditions. The deceleration waves propagates backwards on the average 23% faster than the corresponding acceleration waves. Also the linear path of wave on space-time plane departs from linearity in the vicinity of zero wave speed. Plots of
wave speed versus space mean speed showed the plateaus of nearly constant wave speed corresponding to low speed (less than approximately 30-40 fps). After the plateau there is a distinct quite linear increase of wave speed with increase of space mean speed for both accelerating and decelerating conditions.

The comparison of wave speeds showed that predicted wave speed from Greenburg, Underwood, and a proposed quadratic model are not in a good agreement with the observed wave speeds. Underwood's model gave comparable values. In general actual waves travel slower at absolute speeds and exhibit less difference in wave speed between individual waves that is predicted by the theory.

Although continuum approach predicts existence of shock waves on the highway, Treiterer and Myers investigation of kinematic waves concluded that, there was no shock wave present in the available data from the freeways, and it is doubtful that a fully discontinuous wave will ever be observed on the open roadway.

Investigation of freeway congestion in the vicinity of an observed bottleneck was conducted by Treiterer (1970 EES 278-3) and Clear (1970) using aerial photography. Eight platoons of vehicles all entering and leaving kinematic disturbance were analyzed. When a platoon was released
from maximum density value, a retardation was observed on volume, speed and kinetic energy plots when it was compared to "in going to maximum density" condition at respective densities. They also noticed that at density of approximately 30 vpm, where average spacing is 175 feet, the drivers adopt a headway of 1.8 to 2.2 seconds. The drivers' behavior is influenced by the time headway until a density of 95 vpm is reached. After this point the drivers' behavior becomes influenced by mainly the spacing. As density increases the platoon acts as a unit, and spacings between vehicles in platoon are approximately the same. Safety criteria were investigated and it was found that platoons (142, 143) always exhibited average separations which are much greater than the minimum required for the marginally safe condition \( S = 1.47U^*t \). All platoons before returning to stable flow conditions were subjected to a region where the chance of danger of collision was greater than the accepted value in a stable flow condition. On the other hand, when the platoon was near the state of maximum compression (speed of less than 6 mph) the separations were always greater than what is requested for absolute safety \( S = 1.47U^*t + U^*2/30f \); with a reaction time of 2 seconds, and a 5% probability of collision.
controlling the number of vehicles entering a freeway, through ramp metering, to alleviate congestion downstream from the ramp has been implemented in the past. May (1964) successfully experimented manual and automatic ramp control techniques. Treiterer (1972 EES 278.4) employed effectively fixed-rate ramp metering technique on I-71 in Columbus, Ohio. Miesse (1967) studied the effect of ramp closure and optimization of freeway traffic flow thru ramp closure, and recommended it is an effective mean of traffic control.

Drew et al (1969) utilized gap availability and gap acceptance to reduce the intervehicular influence that takes place during merging process; this is called a multi-level approach.

Theoretical investigation of peak period control of freeway traffic by Wattleworth (1965) using principles of linear programming concluded that ramp metering can be used as a control strategy. The objective was to maximize the volume leaving the system via the freeway mainline output and all of the exit ramps. Wattleworth and Berry (1967) used ramp metering to reduce the total travel time for all vehicles in the system (not individual ramps) and to maximize the system output. The output rate is maximized before the congestion develops because congestion at bottleneck decreases the flow rate below its capacity in the traffic.
Athol (1973) pointed out that freeway control strategies (ramp metering techniques) which were used to maintain a maximum flow rate based on hydrodynamic theory were not feasible due to the fact that the traffic flow breaks down near maximum flow rate. He suggested an analytical approach for multiple ramp control in which a finite probability of breakdown is associated with each level of operation. The objective was to maximize the expected value of uncongested peak period flow.

Kometani et al (1974) used a control strategy in which not only inflow traffic was controlled but also vehicles were requested to leave the expressway. Inflow control used linear programming technique only when there was not a traffic jam. Otherwise, the on ramps were closed in the order of the extent of influence. Their experience showed that the drivers followed the indicated control sign fairly well. 40% of the drivers said they will transfer to surface streets, and if the length of the congestion is known to be more than 5 km, more than 80% will transfer.

Bullen (1972) proposed a real time control algorithm, similar to Chicago Scheme, which uses real time information about flow and occupancy at the bottleneck and entrance and exit ramps to maintain an optimal flow on the freeway. The entrance ramp is controlled whenever there is indication
that the freeway is overloaded. The objective is to maximize throughput of the bottleneck not to sustain maximum flow on Q-K curve. This decision was made because Q-K curve is only obtained from an aggregated observed data that has many stochastic elements in dense traffic. Moreover there are doubts about continuity of Q-K curve. Lane occupancy is used as the indicator to distinguish congested state from uncongested state. The range of 26%-29% was reported to represent a transitional state. The risk of being in transitional state (flow breakdown) is considered in establishing the control volume.

E.5 OTHER APPROACHES

Procedures other than ramp metering are also used successfully to improve traffic operations in bottlenecks. The concept of introducing gaps into tunnel traffic to increase flow level has been discussed by Greenberg and Daou (1960); Edie and Foote (1960, 1961); Helly (1961); Foote et al (1962); Foote (1965); Edie and Bavares (1967); Foote and Crowley (1967); Duckstein (1967).

Gazis and Foote (1969) defined congestion a state corresponding to speed less the 20 mph and density over 55 vpm. Drivers do not accelerate efficiently once they slow down or stop and this causes congestion. Density is used as the
regulating parameter instead of speed, and the results confirmed the previous findings that peak flow can be increased and improved by limiting traffic input to the tunnel.

A procedure to increase efficiency of traffic operation was implemented on Lodge Freeway in Detroit by using closed circuit television (DeRose, 1963). The study site was a 3.2 mile section of the freeway with 9 on and 9 off ramps (3 and 4 lanes). There were 14 cameras approximately 1/4 mile apart covering the study site. The control system consisted of lane signals and variable speed signs on the freeway and ramp closure signs for control of on ramps. The lane signals used the red X, to tell the drivers to leave the lane, as soon as it is safe, and green arrow, to permit driving in that lane. The red X was used only when there was a maintenance or emergency situation on that lane. The lane control signals are installed at 11 locations (6 for northbound and 5 for southbound). Variable speed signs are used in conjunction with the lane signals. The reported results show an efficient handling of lane closure and significant reduction in travel time when the control system operates during an incident.

To evaluate the effectiveness of warning systems on the freeway an experimental warning system was installed in
Gulf Freeway (Dudek and Messer, 1973). Speed and kinetic energy ($aQU$) were used as the criteria for determining of stoppage waves. It would appear that when energy is below $1/2$ of maximum energy and speed is less than $1/3$ of free-flow a shock wave would be detected.

Drew and Keese (1965) suggested considering qualitative measures such as level of service, in freeway operation to overcome the weaknesses of procedures based on achieving higher traffic volume. They divide traffic energy into two components: kinetic energy ($aku^2$) and internal energy. Kinetic energy is the energy of motion of the traffic stream. Internal energy (acceleration noise) measure factors related to level of service such as interruption, comfort, and freedom to maneuver. They suggested that traffic energy should be used instead of traffic momentum (volume) for optimization purpose. Attempts should be made on maximizing kinetic energy and minimizing internal energy. May, Moskowitz, and Hess in discussion of this paper supported the idea of using Kinetic energy as a measure of traffic conditions. Treiterer (ess 278-3 1970) discussed some short comings of the energy concept and suggested that coeffient of variation of speed should be used as a measure of internal energy. His investigation indicated that principle of conservation of energy does not hold as long as kinetic energy is defined as $aKU^2$, no matter how internal energy is represented.
Two publications one from FHWA, Urban Freeway Surveillance and Control 1973 P.F. Everall and other from HRB 368, accomplishments in freeway operations, 1973, gave a good summary of previous works in freeway operations all over the world.

An algorithm for a real time speed control system was suggested by Kleinman and Wiener (1974) to increase comfort level and safety in dense traffic. The advisory signs are 0.1 miles apart along the highway and roadbed detectors transmitted speed and density data to an on-line computer to determine updated sign settings. The sign setting is restricted to 5 mph increments. The objective was to minimize the acceleration noise of all vehicles. A simulation example is given about how the system works.

Temporal development of congestion investigated by Leutzbach and Wiedeman (1977, 7th int’l) revealed that the shape of fundamental diagram depends on the location of the observation point on the study section. Their simulation study showed that the greater the distance of observation point from the bottleneck the better shape of the Q-K curves. Plot of volumes versus density, for a given point, when operation mode goes from free flow to congested flow and back to free flow showed the hysteresis phenomenon similar to that of Treiterer’s.
Heathington, Worrall and Hoff (1970) analyzed drivers performance for alternative visual information displays. The response of 732 drivers to different descriptors (signs) showed that in all congestion levels, the no information is the least preferred of all the descriptors. The drivers preferred some form of traffic information on the freeway regardless of level of congestion. In heavy congestion, they preferred information about the accident and the congestion more than any descriptor. The speed information is ranked second in dense traffic and first at the other levels. Travel time, delay and blank sign were less preferred.

Can drivers sense vehicle velocity correctly? A study by Snider (1967) concluded that in general the speed estimate or production was very close to actual speed, especially when there was feedback to drivers after each trial.

This background would help us to simulate the traffic movement through the bottleneck with less assumptions and with more realistic features.
Appendix F

MAIN COMPONENTS OF A MICROSCOPIC SIMULATION MODEL

Car following and acceleration routines are the most important part of a microscopic traffic simulation model. In chapter 3 a very detailed discussion of these essential parts was carried out. In this chapter some other important components will be discussed.

F.1 INTERARRIVAL TIME OF VEHICLES

Several distributions have been used to represent the vehicles interarrival time distribution. In the following sections the basic idea will be discussed very briefly in order to support the decision on what distribution should be used in this model.

How often should a vehicle be generated in a simulation model and what is the interarrival time distribution in a dense traffic? There is no consensus of opinion on this matter, mainly because the density which effects the distribution itself is a variable. Over the years, many sug-
gestions have been made, most of them about non-congested traffic situation. Poisson distribution for counting and exponential distribution for interarrival time is suggested by many people.

Another popular distribution is composite exponential suggested by Schuhl (1955). One function for free flow condition with random arrival, and the other one for congested flow for non-random arrival. In the following section the diversity of opinion will be reviewed in order to decide on the distribution to be used in this model.

May & Wagner (1960) analyzed headway characteristics on surface streets and expressways, and reported that the mode of distribution was less than the median and the median was less than the mean headway. Roughly 1/3 of vehicles were traveling at headways less than one-half the average headway, one-third at between one-half of the average and the average, and the rest at headways greater than average time headway.

Haight et al (1961) suggested the generalized poisson model, which is the sum of several poisson distributions, for counting distribution and the resulting Erlang distribution for headways. This model can be considered a composite model because it covers random arrivals when only one poisson distribution is used, and non-random arrivals when sum of several poisson distributions are used.
Kell (1962) modified Schuhl's model by using minimum headways and grouped vehicles into free flowing and restrained vehicles to study delay at an intersection. The model is composed of two shifted negative exponential functions.

Oliver & Thibault (1962) after reviewing the previous studies considered minimum spacing and platooning characteristics of traffic and derived a counting distribution for dense traffic using Schuhl's model.

May (1965) investigated the headways on median and shoulder lane of Eisenhower Expressway and concluded that the arrivals are not poisson, so negative exponential cannot be used to represent the headway distribution. A shift of one second improved the fit only a little. He suggested composite normal-poisson distribution. Normal distribution for vehicles in platoon (restrained vehicles), and poisson for vehicles out of platoon (free flowing). Mean time headway for vehicles in platoon was 1.5 seconds and proportion of vehicles having headway from 0 to 1.5 seconds was 50% of the vehicles in platoon.

Athol (1965) stated that for platoon of vehicles the distribution of headways does not closely resemble to negative exponential distribution as it is in random arrival case. Congestion effects the headway and proportion of vehicles in platoon. As volume increase the proportion of
vehicles in platoon increases, but the incidence of shorter headways decreases. However, the platoon headways may be considered independent of number of vehicles in platoon.

Buckley (1967) reviewed previous studies and proposed a negative binomial distribution for counting, and generalized counting distribution which was generalization of negative exponential, displaced negative exponential, and Erlang distribution.

Buckley (1968) proposed semi-poisson distribution for headways for single lane traffic. Semi-poisson is a mixed distribution model, and headways associated with it are a generalization of shifted delta-exponential, delta-exponential, shifted exponential, and exponential. Data from median lane of a 6-lane freeway was used for validation of the model. For high and high medium flows semi-poisson distribution with gamma distributed zone of emptiness (spacing between vehicles) was obtained. One problem with using this model is obtaining the parameters of the distribution.

Dawson & Chit'ni (1968) suggested a hyper-Erlang probability distribution model that considers minimum headway and two modes the vehicles might have; free flowing and constrained. This model is very general and includes the negative exponential, hyper-exponential, and Erlang distribution as a special case.
Weiss (1969) investigated the headway distribution by examining of the linear car following model of Pipes' and concluded that the model does not lead to reasonable results in determining a limiting headway distribution.

Tolle (1969, 1976); Treiterer (1969 EES-278-2) suggested lognormal distribution for time headways. A comparison was made between lognormal, exponential, shifted exponential, Pearson type III, and composite exponential model of Schuhl's. Data used was collected from I-71 and three other US routes. They concluded that lognormal gave better fit than any other distributions for a wide range of volume. Shifted lognormal distribution provided even better fit, but it had one more parameter to be estimated.

Cowan (1975) suggested a composite model with two components: one for freely traveling vehicles, and the other one for vehicles tracking their predecessor.

Bransten (1977) found that the mean headways for different lanes are not the same. For nearside lane it was 1.5 sec., but for offside lane it was 1.29 sec. It was also reported that the time spacing (headway between rear of vehicle to front of another vehicle) adopted by light vehicles (cars, vans) when following a heavy vehicle was 1.26 -1.16 seconds; and when a heavy vehicle was following a light vehicle it was 1.55-1.43 sec.
Wasielewski (1979) used modified version of semi-Poisson model in which cars were divided into two groups: leader and follower. Vehicles with headway about 3 or 4 seconds were considered leader and vehicles with headways less than 1 sec. were followers. This model was used for analysis of 42000 headways observed in urban expressway revealed that, at flow level of less than 1450 vph there is no significant probability of interaction behind a headway of 2.5 sec. For flow level of greater than 1450 this value was 3.5 sec.

Wasielewski (1981) investigated the effect of car size on headways in freely flowing freeway traffic. The study revealed that the medium size following car had lower headway than small and large size following car. Also the effect of small cars was more on the headways of the cars following them due to the fact that the length of the lead car is short. Therefore a potential capacity increase of 8% can be achieved by replacing the traffic mix with small cars.

For light traffic where there is no vehicle interaction, the arrival time distribution is a Poisson and the interarrival time constitutes an exponential function. As traffic volume increases the interaction between vehicles becomes more frequent, and at higher flow levels the interaction dominates the independence. At high density flow it is no
longer realistic to assume that the arrival is poisson and the headways have an exponential distribution. The fact that vehicles are advanced according to the car following rule indicates that there is strong dependency between spacing of vehicles. This is contrary to the assumption of independence between vehicles arrival used in poisson distribution. No matter what the dependency is, the headways will be a function driver-vehicle characteristics. Considering the behavior of drivers in car following situations one may expect that the headway distribution would have an exponential tail. Therefore the headways can be generated from any function that reasonably represents the headway distribution on dense traffic; such as lognormal, truncated normal, truncated exponential, or gamma.

For this model the time headways between successive vehicles is generated from a negative shifted exponential distribution for a given traffic volume. Cumulative distribution function for a negative exponential distribution is expressed as:

\[
F(t) = 1 - e^{-\frac{(t-\tau)}{H-\tau}}
\]

where, \(\tau\) is the amount of shift, and \(H\) is the average headway computed from traffic volume. To find the interarrival time, \(t\), a random number between 0 and 1 is generated from a uniform distribution and is set equal to \(F(t)\). Then this equation is solved for \(t\). The program is instructed
to generate another vehicle after a time interval of $t$ seconds. This process will continue until enough vehicles are generated, or other limiting criteria are satisfied.

When vehicles are allowed into the system their time and space headway will be determined by the car following algorithm. In dense traffic vehicle generation interarrival time is not as critical as it is in light traffic. This is due to the fact that traffic is moving below the desired speed and the spacing is controlled not by intrarrival time of the vehicles upon generation, but by the logic of the car following algorithm.

**F.2 GENERATION**

Vehicles are generated one at a time in the arrival routine. From a uniform distribution between zero and 1 a random number is generated and a corresponding perception brake reaction time is assigned to each driver. The perception brake reaction time is determined using cumulative frequency distribution plot of drivers reaction time under limited anticipation situation based on Johansson & Rumer data. Reaction time for surprised situations are obtained by multiplying these numbers by a factor of 1.35.

Adjustments are made for trucks such that no truck with a reaction time of less than 1 second is allowed to enter the
system. This adjustment is made to consider the physical limitation of trucks and to increase spacing when a truck is following another vehicle.

Vehicles desired speed are generated from a truncated normal distribution with a mean of 55 and a standard deviation of 5 mph. Adjustments are made to insure a safe entry into the system, if it is needed.

Driver-vehicle characteristics such as type of vehicle, length of vehicle, emergency deceleration, complying index, beginning speed and location, end speed and location, identification number, and entry time are assigned to every unit upon entry.

Traffic mix is a user's specified input variable. The percentage of trucks in freeway traffic depends on many factors such as time of day, location of the lane on the road freeway location, and physical or operational restrictions. Normally the percentage of trucks in traffic is less than 20%.

A complying index is used to identify the drivers who would drive according to the suggested speed limit from those who would not. The user specifies the percentage of drivers who would comply with the advisory message originated from the traffic control center.
The desired speed and the reaction time of vehicles are adjusted in order to have a safe entry and realistic characteristics. Maximum allowable speed that a vehicle can obtain while satisfying the non-collision constraint is computed from the car following rule. If the speed of the vehicle is greater than the maximum allowable speed the vehicle is not permitted to enter the system.

F.3 BRAKE REACTION TIME

Reaction time not only varies among drivers, but also for a given driver under different situations. The response of a driver jointly varies with stimulus and sensitivity of the driver. Johansson and Rumer (1971) conducted a study to measure the reaction time of drivers, to an auditory stimulus under expected and unexpected situations. In laboratory experiments where the subject knew he would react to a given signal (high degree of expectancy) the mean value of reaction time was between 0.45 and 0.60 seconds. When regular drivers were told to expect to brake in the next 10 km of their trip, the median brake reaction time for 321 drivers was 0.66 seconds and the range was between 0.3 to 2.0 seconds. In another experiment the reaction time of 5 drivers, each repeated 10 times, were measured under surprise conditions. The median of average reaction time was 0.73 seconds and the range was 0.6 to 0.9
seconds. When the drivers were expecting to react in the next 10 km the median for the 5 driver experiment was 0.54 sec. Correction factor for surprise situations was 1.35.

Hooper and McGee (1983) after reviewing AASHTO specification that uses 1.5 seconds for perception and 1 second for reaction time for computation of stopping sight distance, recommended that stopping sight distance standard does not adequately account for breaking inefficiency of trucks. A brake reaction time of 2 seconds for intersection sight distance where there is a stop sign on the minor street was recommended.

They divided the visual perception process into: latency, eye movement, fixation and focusing (detection), and recognition. The estimated total time to do all these processes varies from 2.3 seconds for the 50th percentile to 4.0 seconds for the 99th percentile of drivers. Attention should be given to the fact that, these numbers are obtained by adding the time required to perform each step. These overlapping steps of a continuous process cannot be simply added to determine the reaction time of a driver. The T&TEH (second edition 1982) allows only 1 second reaction time for drivers moving at higher speeds on an urban freeway.
A recent study by Olson, Cleveland et al (1984) used two
groups of drivers: the younger group, age of 40 or less;
and the older group, 60 years old or more. The drivers
reaction times were measured under 3 different conditions.
Surprise trails were when the drivers faced an obstacle on
the reverse slope of a hill with no prior knowledge. Alert
trails were when the same drivers were told that the obsta­
cle would be there. Brake trails were when a red lamp was
fastened to the front of a carhood and subjects were told
that they were to release the accelerator and tap the brake
as rapidly as possible whenever the lamp came on.

For surprise conditions the 5th and 95th percentile
range is 0.85 to 1.6 seconds for young drivers; and very
close to these values for older drivers. These experimen­
tal values may not be observed under normal driving condi­
tions. The range was doubled and the value of 2.4 sec. was
obtained which is very close to 2.5 seconds recommended by
AASHTO.

For young drivers the 5th and 95th percentile for alert­
ed conditions is 0.57 and 1.37; and for the brake condi­
tions the values are 0.38 and 0.86 seconds.

For older drivers the values are a little longer; but
the difference is not much. Actually for surprise condi­
tions the 95th and 5th percentile values for older and
younger drivers are very close.
They also mentioned a study by Triggs and Harris about reaction time of drivers to the road stimuli that indicated that under car following condition the 85th percentile perception response time for unsuspecting drivers is 1.3 seconds. This value is almost equal to the 85th percentile value under surprise conditions.

INTRAS used a constant value of 0.3 sec. for reaction time which should be less than the time scanning interval.

The reaction time used in this model are obtained from a cumulative distribution plot constructed based on Johansson and Rumer's data. The values vary from 0.4 to 1.5 sec for expected condition, in increments of 0.1 seconds. There are 12 different reaction times and one of them will be assigned to each driver. The probability of assigning each one of the 12 values is not the same. The percentage of drivers having a reaction time of less or equal to the specified value are given in Table 1.

These reaction times are used in this model with a limiting maximum value of 1.5 sec. Under normal traffic conditions the reaction times for surprised situations are used. However, at high density traffic conditions the values for alerted conditions is used. It is reasonable to assume that when drivers are in high density and low speed traffic their reaction time will be shorter than free flow.
Table 1

<table>
<thead>
<tr>
<th>% of drivers</th>
<th>alerted situation</th>
<th>surprised situation</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.50</td>
<td>2.03</td>
</tr>
<tr>
<td>98</td>
<td>1.40</td>
<td>1.89</td>
</tr>
<tr>
<td>96</td>
<td>1.30</td>
<td>1.76</td>
</tr>
<tr>
<td>94</td>
<td>1.20</td>
<td>1.62</td>
</tr>
<tr>
<td>90</td>
<td>1.10</td>
<td>1.49</td>
</tr>
<tr>
<td>88</td>
<td>1.00</td>
<td>1.35</td>
</tr>
<tr>
<td>81</td>
<td>0.90</td>
<td>1.22</td>
</tr>
<tr>
<td>72</td>
<td>0.80</td>
<td>1.08</td>
</tr>
<tr>
<td>64</td>
<td>0.70</td>
<td>0.95</td>
</tr>
<tr>
<td>48</td>
<td>0.60</td>
<td>0.81</td>
</tr>
<tr>
<td>20</td>
<td>0.50</td>
<td>0.68</td>
</tr>
<tr>
<td>4</td>
<td>0.40</td>
<td>0.54</td>
</tr>
</tbody>
</table>

conditions. The use of varying reaction time is more realistic than using a constant value for all conditions. When a driver is in a platoon or in unexpected traffic congestion on the freeway he would be driving with more attention to the situation then when there are a few cars on the freeway.

The values used in CARSIM are very close to young drivers' reaction time in Olsen, Cleveland's et al study.

To assign a reaction time to a driver a random number between 0 and 1 is generated from uniform distribution, and
placed in the proper category on the table. The reaction
time for this category is assigned to this particular driv­
er. There are 12 different categories.

4.4 SPEED DISTRIBUTIONS

Various distributions are used to represent the speed
distribution on the roads for different traffic conditions.
One important factor influencing the shape of the distribu­
tion function is the density of traffic. In high density
traffic the variation in speed is not as much as free flow
traffic condition, thus more uniformity is expected. Sev­
eral papers on this subject will be surveyed to show the
effect of using one distribution versus the other, and to
decide on what distribution should be used.

Duncan (1973) assumed that vehicle’s speed at a point
could be represented by a normal distance curve. Ashworth
(1976) proposed to use Erlang distribution instead of nor­
mal distribution because it may give a slightly better rep­
resentation of observed speeds. Breiman et al (1977)
investigated statistical properties of freeway traffic
using aerial photographic data of Long Island Expressway.
They found that successive speeds are heavily correlated,
even at moderate volumes and the correlation increases as
one goes from outer lane to inner lane. Serial correlation
coefficients up to lag six were computed. Lag-one and lag-two serial correlation coefficient were positive in almost all cases. The correlation for heavy flow on the inner lanes did not fall rapidly with the lag. The standard deviation of the speeds ranged from about 5 to 15% of the mean speed with decreasing tendency at higher volume. The standard deviation seemed to increase at the heaviest flow. This can be attributed to breakdown of flow at very heavy traffic. They hypothesized the speeds were normally distributed, and suggested to use modified Chi Squared Test to check this normality because the speeds are highly correlated. Modified Chi Squared procedure is to form the usual Chi Squared but compute its distribution under the null hypothesis that the sequence form a normally distributed process with an autocorrelated function. At all volumes the time sequence of speeds is normally distributed. The space speeds are normally distributed too. The standard deviation of space speed was not affected appreciably by the flow level.

Breiman and Lawrence (1977) developed a stochastic model which provides a good fit for both the speed and headway distributions. The speed distribution given by the model is not quite normal but so nearly normal that would pass the goodness-of-fit test. They used Long Island Expressway data and noticed that the standard deviation at the highest
flow level increased on all three lanes as it was predicted by their model.

Their investigation of the correlation between speed and headway showed that there is no significant correlation at all but at the highest flow level. A positive correlation was observed at the higher flow level.

Phal (1971) made statistical analysis on measured data and found that speed is normally distributed and supported this by simulation results.

Bransten (1979) showed that under capacity condition proportion of vehicles with speed of $U$ is $1/(1+a*U^{(-b)})$; where $a$ and $b$ were positive constants. This model indicated that when speed increases the proportion of drivers in the platoon will increase too.

Richard et al (1965), Arizona Model, used normal distribution to generate desired speed of vehicles which was categorized into 10 equal increments.

Kobett (1968), MIR Freeway Models, used truncated normal distribution for desired speed. The values more than three standard deviations from the mean were not accepted as desired speeds. St. John et al (1970), MIR simulation model for mountainous terrain, used a normal distribution for desired speeds.
Buher et al (1968), Texas Transport Institution Simulation Model, generated pseudorandom numbers from normal distribution with a mean of 0.85 times maximum freeway speed, and use them as the desired speeds of vehicles.

In Sinha's model (1970), the desired speeds were generated as normally distributed random variables with the mean equal to a fraction of maximum freeway speed.

INTRAS uses lane-specific mean desired speed which is a fraction of overall mean desired speed. Individual vehicles desired speed are generated from lane specific mean desired speed using a cumulative frequency distribution which describes the variation about the mean lane value. The ratio of vehicle desired speed to lane specific mean desired speed identifies a drive type.

In CARSIM the desired speeds of vehicles are generated from a truncated normal distribution. From normal distribution with a mean of 55 mph and a standard deviation of 5 mph a pseudorandom number is generated. If this value is not on either tail of the distribution curve it is used as the desired speed of vehicle. The values which are different more than one standard deviation on either side of the mean are truncated.
Appendix G

DESCRIPTION OF COMPONENTS OF CARSIM

This simulation program is composed of a preamble, a main routine, 5 event routines, and 12 subprograms.

G.1 PREAMBLE

Preamble marks the beginning of a program, and contains the static description of each modeling element. There is no executable statement; only declaration statements are used in the preamble. All modeling elements such as events, sets, processes or resources must be declared in the preamble. Variables used in different parts of a program are named here. Temporary and permanent entities and their attributes are declared. Set name and set owner and set member are specified. Global integer or real variables are declared. Mode and dimensions of an array are specified. Background conditions for the variables can be set and performance measures to be collected can be specified here. Following this preamble is the main routine, the events, and the subprograms. Descriptions of each one of them will be given in the following sections.
G.2 MAIN

The first statement in the main program causes the execution of a SIMSCRIPT program to start. Specifying man-machine-environment characteristic, reading of input data, allocating of computer memory, scheduling of initial or future events are all performed in this part of the model.

G.3 ARRIVAL

Vehicles are generated one at a time and the arrival time of each vehicle is recorded. A reaction time from reaction subprogram, and a desired speed from truncated normal distribution with a mean of 80.66 ft., and a standard deviation of 7.33 fps is assigned to this vehicle. In order to provide a safe entry for this vehicle into the system an adjustment is made on the entry speed. The following attributes are assigned to each vehicle upon entry: arrival time, brake reaction time, desired speed, type of vehicle, length of vehicle, emergency deceleration of vehicle, complying index of the driver, position of a vehicle at the beginning, position of a vehicle at the end of a time interval, speed of a vehicle at the end of a time interval, speed of a vehicle at the end of scanning time interval. After assigning the attributes the vehicle is filed as a system entity.
Another vehicle is scheduled to arrive in a time interval which is generated randomly from a shifted exponential distribution. It is assumed that the interarrival time of the vehicles has a negative exponential distribution, shifted one second in the positive direction. This process of rescheduling itself will continue until enough vehicles are in the system, or the time of entry to the system is past.

G.4 UPDATE

This part of the program updates the speed and position of all vehicles at the end of every time scanning interval, usually 1 second. The time scanning interval (update period) is 1 second in this program. The first vehicle in the system is considered the lead car and the second vehicle as the following car. After updating speed and position of the second car it is considered the lead car and the third car is considered the the following car. This procedure is repeated until speed and position of all vehicles are updated.

For a given position of following vehicle the speed limit will be checked for that area. Depending on whether the driver will obey the advisory sign or not, an acceleration or deceleration will be determined for this vehicle.
There will be two types of drivers: those who comply with the advisory sign and those who do not. The desired speed of the driver from the first group is equal to posted advisory speed limit. For the second group the speed limit is equal to their desired speeds or the speed limit.

To update speed and position of a vehicle the following program is called to determine proper acceleration and deceleration value. Statistics related to the status or trajectory of vehicle may be collected by activating the related subprogram. A binary indices will be assigned to trajectory index and status index; one indicating data collection and zero showing no data collection.

Once speed and position of all the vehicles in the system are updated another update event will be scheduled to happen in update time (1 second).

G.5 SUBPROGRAM REACTION

A reaction time is determined randomly from a set of 12 numbers ranging from 0.4 to 1.5 seconds. The reaction time is for alerted (limited anticipation) conditions; for surprise situations it is multiplied by a factor of 1.35. Less than 4% of the drivers have a reaction time of 0.4 seconds and only 2% have a reaction time of 1.5 seconds, for the alerted conditions.
A random number between 0 and 1 is generated from a uniform distribution. The interval containing this number corresponds to a specific reaction time. There are 12 not necessarily equal intervals covering the range of 0 to 1. Each interval corresponds to a reaction time which differs by a value of 0.1 seconds from the reaction time of immediately adjacent intervals. A maximum reaction time of 1.5 seconds is used for all conditions.

G.6 SUBPROGRAM SHIFTED EXPONENTIAL DISTRIBUTION

Generates headways from negative exponential distribution for a given volume. Cumulative distribution function for the negative exponential distribution is:

\[ C.D.F = F(t) = 1 - e^{-\frac{(t-Tau)}{(T-Tau)}} \]

\( F(t) \) varies between 0 and 1 so it can be represented by a uniform distribution between 0 and 1. Since \( U \) is a random number between 0 and 1, every C.D.F. value will be equal to a value of \( U \). Therefore, one can solve this equation and find \( t \)

\[ C.D.F = U = 1 - e^{-\frac{(t-Tau)}{(T-Tau)}} \]

\[ 1-U = e^{-\frac{(t-Tau)}{(T-Tau)}} \]

\[ \ln(1-u) = -(t-Tau)/t-Tau \]

Solve this for \( t \);

\[ t = Tau - (T-Tau) \ln(1-U) \]
where \( \tau \) is the amount of the shift; and \( T \), the average headway, is equal to the reciprocal of traffic volume. The \( t \) value can also be computed from:

\[
\frac{t}{T} = \frac{\tau}{\tau - T} \ln(1 - U)
\]

because \( U \) and \( 1 - U \) both are random variates.

### G.7 SUBPROGRAM ADJUSTMENT

It adjusts the entry speed of all vehicles except the very first one so they can enter the system without any accident, and with enough spacing from the lead car. If the adjusted speed is less than 30 fps the vehicle is denied the entry.

The reaction time for trucks is adjusted to be at least one second.

### G.8 SUBPROGRAM POINTER

It sets up the pointers for listing of the status of the vehicles, and for collecting of data. The user specifies the time interval for the data collection, and the status listing. This subprogram checks if the time has elapsed enough for such activities.
G.9 SPEED LIMIT

This routine determines the speed limit for the vehicle when the location of a vehicle is known. When the system is off, the speed limit is either the desired speed of the vehicles or 55 mph for entire section of the roadway. However, when the system is on, the advisory signs are posted, then the speed limit is the posted advisory sign. Each advisory sign would influence the speed of the vehicles starting 500 feet upstream from the sign. Each sign can have a different advisory speed limit. They are in decreasing order until the bottleneck is passed and then in increasing order.

G.10 SUBPROGRAM FIND

It checks if the vehicle will pass a data collection point by the end of the current update time. Along the road there are several data collection statistics which are activated when a vehicle passes one of them. From this program a binary code is obtained which in turn activates the data collection routine. A vehicle would be able to activate only one data collection point in every update interval.
G.11   COAST DECELERATION

Determines the coast deceleration of a vehicle. Coast deceleration here is when a driver does not apply the brake but the vehicle will slow down because of rolling resistance and friction between moving parts.

G.12   CAR FOLLOWING

This subprogram determines the speed and the location of a vehicle at the end of every update time. It is the vehicle advancing mechanism of the model. An essential and important part in microscopic simulation models. Proper acceleration or deceleration is determined by calling the acceleration routine. A full description of this routine is given in the car following section of this study.

The car following routine also computed the number of stops and stop delays for the vehicles in the system.

G.13   ACCELERATION

One of the most important tasks in microscopic simulation of traffic flow is finding the proper acceleration or deceleration for the vehicles. This routine does this job in a very realistic way. A full description of acceleration routine used in this model has been given before in the car following section.
Basically, it computes several, acceleration or deceleration values and picks up the most appropriate one. All safety and operation constraints must be checked.

For every vehicle A1, A2, A3, A4, A5 and AC (comfortable deceleration) is determined in every scanning time interval. An acceleration or deceleration value is selected according to the logic described in the car following section.

**G.14 COMFORTABLE DECELERATION**

It finds the comfortable or the normal deceleration rate for a passenger car. 75% of these values are used for trucks since the rates of normal acceleration to maximum acceleration for the passenger car is approximately 3/4.
APPENDIX H

COPY OF ONE OF THE PROGRAMS USED

IN TRAFFIC WAVE STUDIES
```
// JOB
// REGION=1024K, TIME=(2,25)
// JOBPARM LINES=19000, DISKIO=15000, V=S
// PROC LIB DD DSN=TS4.SIMSCRIT.CNTL, DISP=SHR
// SIM EXEC GMCSIM, PARM.CMP=NOEXPDAT
// PRE SYSIN DD *
```

**PREAMBLE**

NORMALLY MODE IS REAL

TEMPORARY ENTITIES

Every vehicle has an `ARR.TIME`, `A TYPE`, `A LENGTH`, `A CMPLY.IND`, `AN ID`, `A BRT`, `A DS.SP`, `A END.SP`, `A BEG.SP`, `A END.PO`, `A BEG.PO`, `A ACCEL`, `A END.DCL`, `AN IND.STOP`, `A TM.MOVE`, `A TM.STOP`, `A STOP.DELAY`, `A STR.TM`, `A STR.DLY`, `A SLOW.IND`, `A TM.SLOW`, `A TM.RECOV`, `AN IND.RECOV`, `AN IND.MOVE`, `MAY BELONG TO THE HOLD and MAY BELONG TO THE SEG`

PERMANENT ENTITIES

Every station has a location, a `S.L`

The system owns a `HOLD` and a `SEG`

DEFINE `HOLD` and `SEG` AS A SET RANKED BY HIGH END.PO

EVENT NOTICES INCLUDE

`ARRIVAL, INPUT.LIST, UPDATE, RESET` and `SIM.RESULTS`

DEFINE `ARR.TIME`, `BRT`, `DS.SP`, `END.SP`, `BEG.SP`, `END.PO`, `BEG.PO`, `ACCEL`, `TRUCK.MIX`, `PER.CMPLY`, `ENT.AXL`, `TH.UPDATE`, `DELTA.TH`, `FSP`, `FPS`, `FDS.SP`, `FAXL`, `FBR`, `FARR`, `SFD.LMT`, `DELAY`, `STR.DLY`, `STR.TM`, `P.DELAY`, `DC.SP`, `DC1.SP`, `DC2.SP`, `DC3.SP`, `DC4.SP`, `DC5.SP`, `DC.HDW`, `DIST.HW`, `DC.TIME`, `INC.TRM`, `INC.STS`, `UP.AXL`, `P.DENS`, `P.VOLM`, `P.VEL`, `STOP.DELAY`, `DURA.SLOW`, `SYSTEM.TM`, `LAST`, `STOP.DELAY`, `THRUPUT` AS VARIABLES


DEFINE `OPERATION` AS INTEGER VARIABLE “1 WHEN SYSTEM IS ON(SPEED LIMITS ARE POSTED), 0 IF NOT

DEFINE DENSITY AS 1-DIM ARRAY

DEFINE SUB.PROGRAM TO MEAN ROUTINE

DEFINE SECONDS TO MEAN UNITS

TALLY MN: `DURA.SLOW` AS THE MEAN, `SN.DURA.SLOW` AS THE SUM, `N.DURA.SLOW` AS THE NUMBER OF DURA.SLOW

TALLY MN: `STOP.DELAY` AS THE MEAN, `SN.STOP.DELAY` AS THE SUM, `N.STOP.DELAY` AS THE NUMBER OF ASTOP.DELAY

TALLY MN: `DELAY` AS THE MEAN, `SN.DELAY` AS THE SUM, `N.DELAY` AS THE NUMBER OF DELAY

TALLY MN: `TM.SYSTEM` AS THE MEAN, `SN.TM.SYSTEM` AS THE SUM, `N.TM.SYSTEM` AS THE NUMBER OF TM.SYSTEM

TALLY MN: `VEL` AS THE MEAN, `SN.VEL` AS THE SUM, `N.VEL` AS THE NUMBER OF P.VEL

TALLY MN: `DENS` AS THE MEAN, `SN.DENS` AS THE SUM, `N.DENS` AS THE NUMBER OF P.DENS

ELSE
  IF U < -0.48 AND U > 0.20
    LET BR.TM = 0.6
    RETURN
  ELSE
    IF U < -0.20 AND U > 0.04
      LET BR.TM = 0.5
      RETURN
    ELSE
      LET BR.TM = 0.4
      RETURN
END "SUB.PROGRAM REACTION"

"" GENERATES HEADWAYS FROM NEG.EXP.DIST FOR A GIVEN VOLUME "" ON THE LANE.
DEFINE H.U, HBAR AND TAU AS VARIABLES
DEFINE QUE AS INTEGER VARIABLE
LET HBAR = 3600.0/QUE
LET U = RANDOM.F(S)
LET TAU = 1.0
LET H = TAU - (HBAR - TAU)*LOG.E.F(U)
RETURN WITH H
END "SHIFTED.EXP.DIST"

SUB.PROGRAM ADJUST(LSPEED, LBRT) YIELDING LAJ.SP, LAJ.BRT
"" TO ADJUST THE ENTRY SPEED AND BREAK REACTION TIME OF VEHICLES.
DEFINE LVOLUME AS AN INTEGER VARIABLE
DEFINE L.TM, L.SP, L.LEN, L.POS, L.ACCEL, POS.Lead, SP.Lead, M.D.T AS VARIABLES
LET LVOLUME = N.SEG
IF LVOLUME = 0
  LET LAJ.SP = LSPEED
  LET LAJ.BRT = LBRT
ELSE
  LET L.TM = TIME.V - TM.UPDATE
  LET L.SP = BEG.SP(L.SEG)
  LET L.LEN = LENGTH(L.SEG)
  LET L.POS = BEG.PO(L.SEG)
  LET L.ACCEL = ACCEL(L.SEG)
  "" FIND POSITION OF LEADING VEH NOW (AFTER L.TM)
  LET POS.LEAD = L.POS + L.TM*L.SP + L.ACCEL*L.TM**2/2
  LET SP.LEAD = L.TM*L.ACCEL + L.SP
  "" FIND THE SPEED WHEN NON-COLLISION CONSTRAINT ARE SATISFIED TO FIND
  "" IT CONSIDERS THE WORST CASE WHICH IS WHEN THE LEADER IS A PASSENGER
  "" CAR AND THE FOLLOWER IS A TRUCK
  "" XL > LL + 10 + VF*LBRT + VF*2/(2*M.D.T) - VL**2/(2*M.D.C)
  "" SOLVE THIS EQUATION AND FIND THE INTERVAL WHICH IS ACCEPTABLE
  "" VF = LBRT*M.D.T + SQRT.F(LBRT**2*M.D.T*M.D.T + 2*M.D.T*(XL-LL-10)) + VL**2
  LET MAX.SPEED = LBRT*M.D.T + SQRT.F(LBRT**2*M.D.T**2 + 2*M.D.T*(POS.Lead - L.LEN - 10) + SP.Lead**2)
  IF MAX.SPEED > LSPEED
    LET LAJ.SP = LSPEED
  ELSE
    LET LAJ.SP = LSPEED
    END "SUB.PROGRAM ADJUST"
PRINT 1 LINE WITH CODE,TIME.V,P.VEL,P.DENS,P.VOLM,THRUPUT,QUE, 
SEEDLINE THUS 
** **** *** ** **** *** ** **** *** ** **** *** ** **** *** ** 
ALWAYS 
ALWAYS 
"FOR R=1 TO SIM.LENGTH*10 BY 1 ,DO 
" LET DENSITY(R)=NO(R)/0.1 
" LOOP 
IF TIME.V<SIM.TIME 
SCHEDULE AN UPDATE IN 1.00 SECONDS 
ELSE 
PRINT 1 LINE WITH TIME.V THUS 
AT TIME=****.**** THE SIMULATION IS STOPPED FROM UPDATE ROUTINE 
STOP 
ALWAYS 
RETURN 
END ""EVENT UPDATE 

SUB.PROGRAM REACTION YIELDING BR.TM 
""THIS IS TO GENERATE A RANDOM NO FROM U(0,1) AND ASSIGN 
""THE REACTION TIME OF DRIVER ACCORDINGLY 

LET U=RANDOM.F(1) 
IF U>0.81 
GO TO 2 
ELSE 
GO TO 3 
"2" IF U<1.0 AND U>0.98 
LET BR.TM=1.5 
RETURN 
ELSE 
IF U<0.98 AND U>0.96 
LET BR.TM =1.4 
RETURN 
ELSE 
IF U<0.96 AND U>0.94 
LET BR.TM=1.3 
RETURN 
ELSE 
IF U<0.94 AND U>0.90 
LET BR.TM=1.2 
RETURN 
ELSE 
IF U<0.90 AND U>0.88 
LET BR.TM =1.1 
RETURN 
ELSE 
LET BR.TM=1.0 
RETURN 
"3" IF U<0.81 AND U>0.72 
LET BR.TM =0.9 
RETURN 
ELSE 
IF U<0.72 AND U> 0.64 
LET BR.TM=0.80 
RETURN 
ELSE 
IF U<0.64 AND U>0.48 
LET BR.TM=0.7 
RETURN
"ALWAYS
IF (END.SP(VEHICLE)>80.57 AND SLOW.IND(VEHICLE)=1 AND ((TIME.V>
TM.MOVE(VEHICLE)) AND IND.MOVE(VEHICLE)=2) OR (ID(VEHICLE)=NO.RECOV+1))

IF IND.RECOV(VEHICLE)=0 " 0 MEANS NOT RECOVERED YET
LET TM.RECOV(VEHICLE)-TIME.V
LET DURA.SLOW-TM.RECOV(VEHICLE)-TM.SLOW(VEHICLE)
LET INDEX=3
LET TM.LAST-TM.RECOV(VEHICLE)
WRITE QUE,SEEDLINE,DECEL.RATE,INDEX,ID(VEHICLE),TM.RECOV(VEHICLE),BEG.P0(VEHICLE),BEG.SP(VEHICLE),
END.SP(VEHICLE) AS BINARY USING 1
WRITE AS / USING 1
PRINT 1 LINE WITH CODE,QUE,SEEDLINE,DECEL. RATE,INDEX,
ID(VEHICLE),TM.RECOV(VEHICLE),BEG.PO(VEHICLE),BEG.SP(VEHICLE) THUS
** **** ** **** **** ***** **** ***
LET IND.RECOV(VEHICLE)!!!!
LET NO.RECOV-NO.RECOV+1
ALWAYS
ALWAYS
ALWAYS
LET LTYPE=FTYPE
LET LSPEED=LSP
LET LPOSITION=LPOS
LET LLEN=LEN
IF LPOS>SIM.LENGTH*5280
LET NO.EXIT-NO.EXIT+1
LET IN.VOLUME-IN.VOLUME-1
LET TM.SYSTEM-TIME.V-ARR.TIME(VEHICLE)
LET DELAY=SIN.LENGTH*5280/80.67 - TM.SYSTEM
IF ID(VEHICLE)=235
PRINT 1 LINE WITH TIME.V,GRID(VEHICLE),END.SP(VEHICLE) THUS
AT THE-***.** VEHICLE NO.-*** LEFT WITH SPEED OF-***.**
ALWAYS
DESTROY THIS VEHICLE
GO TO 30 " TO LOOP
ELSE
"DEFINE NO(SUB.NO) AS A ONE DIMENSIONAL ARRAY
"LOCAL TO UPDATER ROUTINE
IF SUB.NO=1
LET NO(SUB.NO)=NO(SUB.NO) +1
ALWAYS

LET SUM.VEL=SUM.VEL + END.SP(VEHICLE)

"CALL FIND(FPOS,LPOS) YIELDING COLL.IN
"20" FILE THIS VEHICLE IN HOLD
"30" LOOP
LET P=N.HOLD
FOR M=1 TO P,DO
REMOVE THE FIRST VEHICLE FROM HOLD
FILE THIS VEHICLE IN SEG
LOOP
IF TIME.V>180
IF N.SEG>1
" DENSITY AND VELOCITY COMPUTATION FOR PLATOON
LET P.DENS-(N.SEG-1)*5280/(END.PO(F.SEG) - END.PO(L.SEG))
LET P.VEL-(SUM.VEG-END.SP(F.SEG))/(N.SEG-1)
LET Thruput=P.DENS*P.VEL/1.467
LET Thruput=P.DENS*P.VEL*2
LET BEG.SP(VEHICLE)=FS
LET END.SP(VEHICLE)=LSPEED
LET END.PO(VEHICLE)=LPOSITION
LET ACCEL(VEHICLE)=LAXL.MAX
LET UP.AXLF=LAXL.MAX ""ACCEL. OF FOLLOWER DURING THIS TIME INTERVAL
""IF K=80 AND K<85 AND TIME.V>180
""PRINT 1 LINE WITH BEG.PO(VEHICLE), BEG.SP(VEHICLE), END.PO(VEHICLE),
""END.SP(VEHICLE), ID(VEHICLE), ACCEL(VEHICLE), BRT(VEHICLE), TIME.V THUS
""ALWAYS
IF TIME.V>180
IF (ABS.(END.SP(VEHICLE)-80.67)>0.1 AND ACCEL(VEHICLE)<-0.0)
AND (TIME.V-ARR.TIME(VEHICLE))>15.0
IF SLOW.IND(VEHICLE)=0 ""O MEANS IT HAS NOT SLOWED DOWN
LET TM.SLOW(VEHICLE)=TIME.V
LET NO.SLOW=NO.SLOW+1
LET SLOW.IND(VEHICLE)=1 ""1 MEANS IT HAS SLOWED DOWN
WRITE QUE, SEEDLINE, DECCEL.RATE, SLOW.IND(VEHICLE), ID(VEHICLE),
TM.SLOW(VEHICLE), BEG.PO(VEHICLE), BEG.SP(VEHICLE),
END.SP(VEHICLE) AS BINARY USING 1
WRITE AS / USING 1
PRINT 1 LINE WITH CODE, QUE, SEEDLINE, DECCEL.RATE, SLOW.IND(VEHICLE),
ID(VEHICLE), TM.SLOW(VEHICLE), BEG.PO(VEHICLE), BEG.SP(VEHICLE) THUS
** **** ** **** **** *** *** *** *** *** *** *** *** *** ***
ALWAYS
ALWAYS
IF END.SP(VEHICLE)>-0.01
IF IND.STOP(VEHICLE)=1 AND ID(VEHICLE)=NO.MOVE+1
""1 MEANS THE VEHICLE WAS STOPPED
LET TM.MOVE(VEHICLE)=TIME.V
LET IND.MOVE=IND.MOVE+1
LET INDEX=2
WRITE QUE, SEEDLINE, DECCEL.RATE, INDEX, ID(VEHICLE),
TM.MOVE(VEHICLE), BEG.PO(VEHICLE), BEG.SP(VEHICLE),
END.SP(VEHICLE) AS BINARY USING 1
WRITE AS / USING 1
PRINT 1 LINE WITH CODE, QUE, SEEDLINE, DECCEL.RATE, INDEX,
ID(VEHICLE), TM.MOVE(VEHICLE), BEG.PO(VEHICLE), BEG.SP(VEHICLE) THUS
** **** ** **** **** *** *** *** *** *** *** *** *** *** ***
ALWAYS
LET IND.STOP(VEHICLE)=0 ""MOVING
ELSE
IF IND.STOP(VEHICLE)=0
LET TM.STOP(VEHICLE)=TIME.V+1.0
LET NO.STOP=NO.STOP+1
WRITE QUE, SEEDLINE, DECCEL.RATE, IND.STOP(VEHICLE), ID(VEHICLE),
TM.STOP(VEHICLE), END.PO(VEHICLE), BEG.SP(VEHICLE),
END.SP(VEHICLE) AS BINARY USING 1
WRITE AS / USING 1
PRINT 1 LINE WITH CODE, QUE, SEEDLINE, DECCEL.RATE, IND.STOP(VEHICLE),
ID(VEHICLE), TM.STOP(VEHICLE), END.PO(VEHICLE), BEG.SP(VEHICLE) THUS
** **** ** **** **** *** *** *** *** *** *** *** ***
LET IND.STOP(VEHICLE)=1 ""WAS STOPPED
ALWAYS
ALWAYS
""IF TIME.V>580 AND TIME.V<650 AND ID(VEHICLE)>115
""PRINT 1 LINE WITH END.SP(VEHICLE),
""END.SP(VEHICLE), SLOW.IND(VEHICLE), NO.RECOV, ID(VEHICLE), K,
TIME.V THUS
** **** ** **** **** *** *** *** *** *** *** *** ***
**always**

**always**

**always**

**always**

**else**

let operation = 0

**always**

let delta.tm = 1.00

**define** no as a 1-dim, integer array

**define** k, n, coll.in, p, m, r, llen, sub.no, sts.in, trj.in,

**index** as integer variables

**define** sum.vel as variables

**reserve** no(*) as 50

**let** th.update-time.v

"*call* pointer yielding trj.in, sts.in

**let** n.n.seg "no. of vehicles in the segment

**for** k = 1 to n, do

**remove** the first vehicle from seg

"?? let ds.sp(vehicle) = 80.67

"*copy* attributes of this vehicles

**let** fid = id(VEHICLE)

**let** fsp = end.sp(VEHICLE) "speed of follower at begining of update time

**let** fps = end.po(vehicle)

**let** fds.sp = ds.sp(VEHICLE)

**let** faxl = accel(VEHICLE) "follower accel. during previous time interval

**let** ftype = type(VEHICLE)

**let** flen = length(VEHICLE)

**let** fcmpl = comply.ind(VEHICLE)

**let** fbrt = brt(VEHICLE)

"*when* traffic is congested, drivers are more alerted

**if** p.dens > 60 and end.sp(VEHICLE) < 30

**let** fbrt = brt(VEHICLE) / 1.35

**always**

**if** fcmpl = 1 "driver obey the advisory sign

**call** speed.limit(fpos) yielding sp.limit

"*for* a given veh at a given location it finds the speed limit

"*at* data collection points the speed changes will take place

**let** fds.sp = sp.limit

"go to 9

**else**

**let** fds.sp = spd.lmt

"9* call car.following(fsp, fpos, fds.sp, fbrt, ftype, llen)

yielding lspeed, lposition, laxl.max, sub.no

"19* **if** sts.in = 1

"list the status of the vehicles

**call** sts.list(k, fid, fds.sp, fbrt, ftype, fsp, lspeed, faxl, laxl.max, fpos, lposition)

**always**

**if** trj.in = 1

**call** data2 (fsp, lspeed, fpos, lposition, laxl.max, ftype, delta.tm, k, fid,

lsf, lpos, llen, trj.in)

**always**

**let** beg.po(VEHICLE) = fpos
IF U= TRUCK.MIX
LET TYPE(VEHICLE)=2
LET LENGTH(VEHICLE)=TRK.LEN
LET EMR.DCL(VEHICLE)=M.D.T
ELSE
LET TYPE(VEHICLE)=1
LET LENGTH(VEHICLE)=CAR.LEN
LET EMR.DCL(VEHICLE)=M.D.C
ALWAYS

"TRUCKS WILL HAVE AT LEAST 1 SEC. REACTION TIME"

IF TYPE(VEHICLE)=2 AND BRT(VEHICLE)<1.0
LET BRT(VEHICLE)=1.0
ALWAYS
LET U=RANDOM.F(3)
IF U<=PER.CMPLY
LET CMPLY.IND(VEHICLE)=1
ELSE
LET CMPLY.IND(VEHICLE)=0
ALWAYS
LET BEG.PO(VEHICLE)=0.0
LET TERM=TH.UPDATE+1.0-ARR.TIME(VEHICLE)
LET END.SP(VEHICLE)=AJ.SPEED*TERM +ENT.AXL*TERM**2/2
LET END.P0(VEHICLE)=AJ.SPEED*TERM +ENT.AXL*TERM**2/2
LET IN.VOLUME=IN.VOLUME + 1 "NO OF VEHICLES IN THE SYSTEM"
LET NUMBER=NUMBER + 1
LET ID(VEHICLE)= NUMBER
FILE THIS VEHICLE IN SEG

"CALL SH.EXP.DISQUE YIELDING EACH"
IF NUMBER<235 "MAX 231 CARS ARE AFFECTED SO WE GENERATED ENOUGH"
SCHEDULE AN ARRIVAL IN EACH SECONDS
ELSE
PRINT 1 LINE WITH TIME.V,NUMBER THUS
"TIME IS ****.*,**** NO. OF VEHICLES GENERATED ****"
ALWAYS
RETURN

END "EVENT ARRIVAL"

EVENT UPDATE

LET CODE=1 "1 MEANS THE SYSTEM IS USED."

IF TIME.V>180 AND TIME.V<600
LET OPERATION=1
IF TIME.V>540
LET S.L(1)=80.67
ELSE
IF TIME.V>540
LET S.L(2)=80.67
ELSE
IF TIME.V>540
LET S.L(3)=80.67
ELSE
IF TIME.V>480
LET S.L(4)=80.67
ELSE
IF TIME.V>360
LET S.L(5)=80.67
LOCATION(4),S.L(5),LOCATION(5),TRUCK.MIX,QUE THUS

TRUCK LENGTH
CAR LENGTH
% COMPLYING
ENTRY ACCEL.
MAX DECEL OF TRUCK
MAX DECEL OF CAR
TIME INCREMENT FOR STATUS LISTING
INCREMENT OF TIME FOR TRAJECTORY
SIMULATION TIME
WARMUP TIME
TRJ.COL.TM
STS.COL.TM
SEED LINE NO.
SEED 1
SEED 2
SEED 3
SEED 4
SEED 5
SEED 6
SEED 7
SIMULATION LENGTH
SPEED LIMIT
NUMBER OF STATIONS
SPEED LIMIT LOCATION
TRUCK MIX IN TRAFFIC
VOLUME OF ARRIVAL (GENERATION)
"" <<< SCHEDULE EVENT AND START SIMULATION
PRINT 1 LINE WITH TIME.V THUS INPUT LIST TIME= ""*****.**
RETURN
END ""INPUT.LIST

EVENT ARRIVAL

DEFINE BR,U,SPEED,AJ.SPEED,AJ.BRT,ECH AS VARIABLES
CREATE A VEHICLE
CALL REACTION YIELDING BR
LET ARR.TIME (VEHICLE )=TIME.V "1" LET SPEED=NORMAL.F(80.67,7.33,4)
  IF SPEED>95.33 OR SPEED <73.33
    GO TO 1
  ELSE
    LET DS.SP(VEHICLE)=80.67 "?? SPEED
""THEN THIS ASSIGNED SPEED IS ADJUSTED: THE SUB.PROGRAM FOR ADJUSTMENT
"" IS CALLED TO DETERMINE THE ENTRY SPEED TO THE SYSTEM
LET ACCEL(VEHICLE)=ENT.AXL ""AT THE ENTRANCE EVERY VEHICLE HAS
""ACCELERATION OF 1 FT/SEC/SEC ,THIS IS
""JUST A STARTING VALUE
CALL ADJUST(SPEED,BR) YIELDING AJ.SPEED,AJ.BRT
LET BEG.SP(VEHICLE)=AJ.SPEED
LET BRT(VEHICLE)=AJ.BRT
IF BRT(VEHICLE)<0.40 OR BEG.SP(VEHICLE)<-30.0
  LET NO.REJ=NO.REJ + 1
  DESTROY THIS VEHICLE
  GO TO 2
ELSE
  LET U=RANDOM.F(2)
```
<<<<< TRAFFIC CHARACTERISTIC >>>>>>

READ TRUCK.MIX "TEAFFIC MIX WHICH SHOW % OF TRUCK
SKIP 1 LINE
READ QUE " VOLUME USED IN NEG EXP FN E.G. 1200
SKIP 1 LINE

SCHEDULE AN INPUT.LIST NOW
SCHEDULE AN ARRIVAL NOW
SCHEDULE AN UPDATE IN 1.0 SECONDS
SCHEDULE A RESET IN WARMUP.TIME SECONDS
SCHEDULE A SIM.RESULTS AT SIM.TIME
START SIMULATION

END " MAIN

EVENT RESET

PRINT 1 LINE WITH TIME.V THUS
RESET TIME=****.****
RETURN

END" RESET

EVENT SIM.RESULTS

PRINT 1 LINE WITH TIME.V, NO.EXIT,NO.REJ THUS
RESULT: TIME=****.**** NO OF CARS EXITED=**** REJECTED=****
PRINT 8 LINES WITH MN.DURA.SLOW,SM.DURA.SLOW,N.DURA.SLOW,MN.STOP.DELAY,
SM.STOP.DELAY,N.STOP.DELAY,MN.DELAY,SM.DELAY,N.DELAY,MN.TM.SYSTEM,
SM.TM.SYSTEM,N.TM.SYSTEM,MN.VEL,SM.VEL,N.VEL,MN.DENS,SM.DENS,
N.DENS,MN.VOLM,SM.VOLM,N.VOLM,MN.THRUPUT,SM.THRUPUT,
N.THRUPUT THUS
DURATION OF SLOWDOWN MEAN=****.** SUM=************ NO=****
DURATION OF STOP MEAN=****.** SUM=************ NO=****
DELAY MEAN=****.** SUM=************ NO=****
TIME IN SYSTEM MEAN=****.** SUM=************ NO=****
OVERALL SPEED MEAN=****.** SUM=************ NO=****
OVERALL DENSITY MEAN=****.** SUM=************ NO=****
OVERALL VOLUME THROUGHPUT MEAN=****.** SUM=************ NO=****

STOP
RETURN

END" SIM.RESULTS

EVENT INPUT.LIST

PRINT 30 LINES WITH TRK.LEN,CAR.LEN,PER.CNPLY,ENT.AXL,M.D.T.M.D,C,
INC.STS,INC.TRJ,SIM.TIME,WARMUP.TIME,TRJ.COL.TM,STS.COL.TM,
SEEDLINE,SEED.V(1),SEED.V(2),SEED.V(3),SEED.V(4),SEED.V(5),
SEED.V(6),SEED.V(7),SIM.LENGTH,SPD.LMT,N.STATION,S.L(1),
LOCATION(1),S.L(2),LOCATION(2),S.L(3),LOCATION(3),S.L(4),
```
TA LLY Mn.THRUPUT AS THE MEAN, SM.THRUPUT AS THE SUM,N.THRUPUT AS THE NUMBER OF THRUPUT
END ""PREAMBLE

MAIN

"" READS VEHICLES AND DRIVERS CHARACTERISTIC,ROADWAY PARAMETERS,
"" TRAFFIC CHARACTERISTICS, INPUT DATA, AND ALLOWS COMPUTER MEMOR TO
"" ARRAYS, SCHEDULES ARRIVAL, RESET, SIM. RESULT, AND STARTS SIMULATION
"" <<<<<<< VEHICLE AND DRIVER CHARACTERISTICS >>>>>>>

READ TRK.LEN ""TRUCK LENGTH
START NEW CARD
READ CAR.LEN ""PASSENGER CAR LENGTH
START NEW LINE
READ PER.CMPLY ""% OF DRIVERS COMPLY WITH THE ADVISORY SPEED LIMIT
SKIP 1 LINE
READ ENT.AXL ""ENTRY ACCELERATION
SKIP 1 LINE
READ M.D.T ""MAX DECELERATION OF TRUCK
SKIP 1 LINE
READ M.D.C ""MAXIMUM DECELERATION OF CAR
SKIP 1 LINE
READ INC.STS ""INCREMENT OF TIME STATUS IS LISTED
SKIP 1 LINE
READ INC.TRJ ""INCREMENT OF TIME TRAJECTORY DATA IS COLLECTED
SKIP 1 LINE

"" <<<<<<< SIMULATION PARAMETERS >>>>>>>

DEFINE I AS INTEGER VARIABLE
READ SIM.TIME ""SIMULATION TIME
SKIP 1 LINE
READ WARM.UP.TIME ""WARMUP TIME
SKIP 1 LINE
READ TRJ.COL.TM ""TRAJECTORY DATA COLLECTION TIME
SKIP 1 LINE
READ STS.COL.TM ""STATUS OF VEHICLE COLLECTION TIME
SKIP 1 LINES
READ SEEDLINE
SKIP 1 LINE
FOR I=1 TO 7 DO
  READ SEED.V(I)
  START NEW LINE
END LOOP

"" <<<<<<< ROADWAY CHARACTERISTICS >>>>>>>

READ SIM.LENGTH "" SIMULATION LENGTH
SKIP 1 LINE
READ SPD.LMT ""THIS SPEED LIMIT IS USED WHEN SIGNS ARE NOT ON
SKIP 1 LINE
"" ALLOWS MEMORY TO ARRAYS
RESERVE DENSITY(*) AS 50

READ N.STATION
CREATE EVERY STATION
FOR EVERY STATION
  READ S.L(STATION), LOCATION(STATION)
LET LAJ.SP = MAX.SPEED
ALWAYS

""SPEED IS ADJUSTED AND VEHICLE WILL ENTER THE SYSTEM. IF THE
""ADJUSTED SPEED IS LESS THAN 30 FT/SEC (20 MPH) THE ENTRY IS DENIED

""FOR SURPRISE CONDITION THE REACTION TIMES ARE MULTIPLIED
""BY A FACTOR OF 1.35

LET LAJ.BRT = LBRT * 1.35
ALWAYS
RETURN
END ""ADJUST

SUB.PROGRAM POINTER YIELDING TRJL, STSL

"" THIS SETS UP PRINTER FOR VEHICLE STATUSE
"" AND TRAJECTORY DATA COLLECTION

DEFINE TRJL, STSL AS INTEGER VARIABLES
IF TIME.V = INC.STS*NO.STS OR TIME.V = 1.0
   LET NO.STS = NO.STS + 1
   LET STSL = 1
ELSE
   LET STSL = 0
   ALWAYS
   IF TIME.V = INC.TRJ*NO.TRJ OR TIME.V = 1.0
      LET NO.TRJ = NO.TRJ + 1
      LET TRJL = 1
   ELSE
      LET TRJL = 0
   ALWAYS
RETURN
END ""POINTER

SUB.PROGRAM SPEED.LIMIT(FP) YIELDING S.LIMIT

DEFINE FP, S.LIMIT AS VARIABLES
DEFINE I, J AS INTEGER VARIABLES
IF OPERATION = 1 ""SYSTEM IS ON , SPEED LIMITS ARE POSTED
   IF FP < LOCATION(I) - 500 ""NOT PASSED REGION OF STATION 1
      LET S.LIMIT = SPD.LMT
      RETURN
   ELSE
      FOR I BACK FROM N.STATION TO 1 BY 1, DO
         IF FP > LOCATION(I) - 500
            LET S.LIMIT = S.L(I)
            RETURN
         ELSE
            LOOP
      ELSE
         ""OPERATION = OFF

   LET S.LIMIT = SPD.LMT
   ALWAYS
RETURN
END ""SPEED.LIMIT
SUB.PROGRAM FIND(XBEG,XEND) YIELDING CLLC

DEFINE XBEG,XEND AS VARIABLES
DEFINE CLLC,I AS INTEGER VARIABLES

"TO FIND OUT DATA COLLECT POINT ACTIVATED BY A VEH DURING AN UPDATE
IF XEND<LOCATION(I)
RETURN WITH 0
ELSE
FOR I BACK FROM N.STATION TO 1 BY 1 ,DO
  IF XEND>LOCATION(I)
    "VEH HAS PASSED THE DATA COLLECTION POINT OR IS AT THERE
    IF XBEG<LOCATION(I)
      LET CLLC=1
    ELSE
      LET CLLC=0
    ALWAYS
  RETURN
ELSE
LOOP
END "FIND

SUB.PROGRAM COAST.DECEL(FSP,FTYPE) YIELDING COAST.DECEL

DEFINE COAST.DECEL AS VARIABLE
IF FSP>95.333
  LET COAST.DECEL=-3
  GO TO 1
ELSE
  IF FSP>80.667 AND FSP<95.333
    LET COAST.DECEL=-2.8
    GO TO 1
  ELSE
    IF FSP>66.000 AND FSP<80.667
      LET COAST.DECEL=-2.3
      GO TO 1
    ELSE
      IF FSP>51.333 AND FSP<66.000
        LET COAST.DECEL=-1.9
        GO TO 1
      ELSE
        IF FSP>36.667 AND FSP<51.333
          LET COAST.DECEL=-1.6
          GO TO 1
        ELSE
          LET COAST.DECEL=-1.3
"1" IF TYPE(VEHICLE)=2 " TRUCK
  LET COAST.DECEL=0.75*COAST.DECEL
ALWAYS
RETURN
END "COAST.DECEL

SUB.PROGRAM CAR.FOLLOWING(VF, XF, DS.VF, BRTF, XF, DLT.T, TYP.F, TYP.L, VL, XL, AXLF, LL) YIELDING NW.SP, NW.POS, NW.AXL, SUB.NO
DEFINE VL, XL, DS.VF, BRTF, DLT.T, VF, XF, AXLF, NW.SP, NW.POS, NW.AXL, A AS VARIABLES
DEFINE KF, TYP.F, TYP.L, LL, SUB.NO, R AS INTEGER VARIABLES

"" IF THIS IS THE FIRST CAR THERE WILL NOT BE A CAR FOLLOWING
LET DECEL.RATE = -8

"1" IF KF=1
" THIS IS TO DECELERATE IN MAXIMUM DECELERATION
" IF TIME.V>180.00 AND TIME.V<360.00
LET NW.AXL = DECEL.RATE
GO TO 51
ELSE
-- IF TIME.V>25.0 AND TIME.V<30.0
-- LET NW.AXL = -6.0
-- GO TO 51
-- ELSE
-- IF TIME.V>30.0 AND TIME.V<33.0
-- LET NW.AXL = 0.0
-- GO TO 51
-- ELSE
-- IF TIME.V>33.0 AND TIME.V<39.0
-- LET NW.AXL = 6.0
-- GO TO 51
-- ELSE
" THIS PART WAS TO INTRODUCE DISTURBANCE
-- IF VF = DS.VF
LET NW.AXL = 0.0
ELSE
CALL ACCELERATION(KF, VL, XL, VF, XF, TYP.F, TYP.L, LL, DLT.T, DS.VF, BRTF, SUB.NO) YIELDING AXL, A
IF VF < DS.VF
LET NW.AXL = A
ELSE
IF CMPLY.IND = 1
CALL COAST.DECEL(VF, TYP.F) YIELDING CST.DECEL
LET NW.AXL = CST.DECEL
ELSE
LET NW.AXL = MIN.F(AXL, 0.0)
" " NON COMPLYING DRIVER AT CONSTANT SPEED OR DECELERATION
ALWAYS
ALWAYS
ALWAYS
GO TO 51
ELSE
CALL ACCELERATION(KF, VL, XL, VF, XF, TYP.F, TYP.L, LL, DLT.T, DS.VF, BRTF, SUB.NO) YIELDING AXL, A

IF VF <= VL + 3 AND (VL < 0.5 AND XL - XF < LL + 10)
LET NW.AXL = VF
GO TO 51
ELSE
LET NW.AXL = AXL
"51" LET VELOCITY = VF + NW.AXL * DLT.T
IF VELOCITY > 0.0 " CAR FOLLOWING GIVES A + SPEED; LET NW.SP =
LET NW.SP = VELOCITY " BE EQUAL TO THIS COMPUTED SPEED
ELSE
LET NW.AXL = -VF / DLT.T
LET NW.SP=0.00

"61" LET NW.POS=XF+VF*DLT.T+NW.AXL*DLT.T**2/2
RETURN

END "CAR FOLLOWING

SUB.PROGRAM ACCELERATION(K,UL,YL,UF,YF,TY.FOL,TY.LD,LL,DL.TM,D.SP,
BRF,SUB.NO) YIELDING AXL,A

"THIS WOULD FIND ACCELERATION AND DECELERATION OF FOLLOWER FOR
"VARIOUS CASES

"CASE 1- FOLLOWING VEHICLE IS MOVING. ACCELERATION OF THIS VEHICLE
"DEPENDS ON ITS SPEED. FOR A GIVEN SPEED INTERVAL ACCELERATION WILL
"BE ASSIGNED FROM THE TABLE. CALL THIS ACCELERATION A1.
"

"CASE 2- COMPARE THE SPEED OF VEHICLE WITH ITS DESIRED SPEED OR THE
"SPEED LIMIT; AND COMPUTE ACCELERATION OR DECELERATION NECESSARY TO
"REACH THE SPEED LIMIT. CALL THIS A2.
"

"CASE 3- FOLLOWING VEHICLE IS STOPPED, FIND ACCELERATION WHEN
"VEHICLE STARTS FROM A STILL POSITION. CALL THIS A3
"

"CASE 4- COMPUTE ACCELERATION FROM CAR FOLLOWING MODEL NOT LIMITED
"BY NON-COLLISION CONSTRAINT. CALL THIS A4.
"

"CASE 5- COMPUTE ACCELERATION FROM CAR FOLLOWING LIMITED BY
"NON-COLLISION CONSTRAINT. CALL THIS A5
"

"THESE VALUES SHOULD BE COMPARED WITH THE LIMITING VALUES
"

"MX.F IS MAX. ALLOWABLE DECELERATION OF FOLLOWER
"MX.L IS MAX. ALLOWABLE DECELERATION OF LEADER

DEFINE UL,UF,YL,YF,DL.TM,D.SP,BRF AS VARIABLES
DEFINE TY.FOL,TY.LD,SUB.NO,LL,K AS INTEGER VARIABLES
IF TY.FOL=1
  LET MX.F=16
ELSE
  LET MX.F=16
ALWAYS
IF TY.LD=1
  LET MX.L=M.D.C
ELSE
  LET MX.L=M.D.T
ALWAYS
IF P.DENS LT 60
  LET MX.F=16 " MAX DECELERATION IS THE SAME IN ALL DENSITIES
ALWAYS
IF K=1
  GO TO 11
ELSE
  " NOTE THIS IF STATEMENT IS ADDED ON 12/10/85
IF UF = 0.0 AND UL = 0.0
LET MIN.AXL = 0.0
GO TO 3
ELSE
   " TO FIND A5 , LET US CONSIDER THE WORST CASE IN CAR FOLLOWING
   " IT WOULD BE WHEN THE LEADING VEHICLE IS DECELERATING AT THE
   " MAX RATE AND THE FOLLOWER IS TRYING TO KEEP A SAFE DISTANCE
   " FROM THE LEADER. FOR THIS CASE THE CAR FOLLOWING RULE IS ::
   " X L - ( X F + V F * D T + 1 / 2 * A S * D T ** 2 ) - L L - 1 0 >
   " MAX(((VF+AS*DT)*BRT+(VF+AS*DT)**2)/(2*MX.F)-VL**2/2*MX.L) OR 0)
   " NOTE THAT RIGHT HAND SIDE IN SOME CASES MIGHT BE NEGATIVE (E.G
   " VF>VL). IN SUCH A SITUATION THERE ARE TWO CHECK POINTS WHICH
   " WOULD PREVENT USING OF A5. WHEN RIGHT HAND SIDE IS NEGATIVE
   " IT MEANS THERE WILL BE MORE SPACE AVAILABLE FOR THE FOLLOWING CAR
   " TO DECELERATE. HOWEVER , ONE SHOULD NOTICE THAT LEFT HAND SIDE OF
   " THE EQUATION MUST BE POSITIVE ALL THE TIMES. OTHERWISE THE FOLLOWER
   " WOULD RUN AWAY FROM THE LEADER. TO OVERCOME THIS SITUATION THE
   " ACCELERATION RATE FROM CAR FOLLOWING , A4, IS COMPUTED. A4 IS
   " COMPUTED BASED ON THE FACT THAT THE SPACING BETWEEN VEHICLE
   " MUST BE GREATER THAN ZERO. ALSO IN THE CAR FOLLOWING MODEL
   " WHENEVER THE RIGHT HAND SIDE OF THE ABOVE EQUATION IS NEGATIVE
   " THE LEFT HAND SIDE IS SET EQUAL TO ZERO AND AS IS COMPUTED.
   " THEREFORE WITH THESE DOUBLE CHECKS ONE CAN FIND A5 FROM THE
   " EQUATION WITH RIGHT HAND SIDE OF A POSITIVE NO.
   " X L - ( X F + V F * D T + 1 / 2 * A S * D T ** 2 ) - L L - 1 0 >
   " (VF+AS*DT)*BRT+(VF+AS*DT)**2/(2*MX.F)-VL**2/(2*MX.L)
   " TO FIND SOLUTIONS TO THIS EQUATION MULTIPLY AND COLLECT THE TERMS
   ON ONE SIDE
   " A5**2*DT**2+A5*(B)+C
   " A5=-B+SQRT.F(B**2-4*DT**2*C))/(2*DT)
   " NOTE THAT + VALUE FROM THE RADICAL IS USED BECAUSE IT GIVES LOWER
   " ACCELERATION OR DECELERATION
   " TO FIND A5 THE FOLLOWING ASSUMPTIONS WERE MADE TO SIMPLIFY THE COMPUTATION
   " B-BEE
   " C-CEE
   IF UF<K3
   LET BEE=2*UF*DL.TM+2*MX.F*DL.TM*BRF+HX.F*DL.TM**2
   LET CEE=2*MX.F*(YL-YF-UF*DL.TM-LL-3-UF*BRF+UL**2/(2*MX.L))+UF**2
   LET RAD=BEE**2-4*CEE*DL.TM**2
   IF RAD GE 0.0
   LET A5=(BEE+SQRT.F(BEE**2-4*CEE*DL.TM**2))/(2*DL.TM**2)
   " AFTER FINDING A5 CHECK IF THE RIGHT HAND SIDE OF THE CONSTRAINT
   " EQUATION IS GREATER THAN OR EQ TO (UF+AS*DL.TM)*BRF. IF IT IS NOT
   " THEN SET LEFT HAND SIDE EQUAL TO THIS AND SOLVE IT AGAIN FOR A5.
   LET R.H.S=(UF+AS*DL.TM)*BRF+(UF+AS*DL.TM)**2/(2*MX.F)-UL**2/(2*MX.L)
   IF R.H.S<(UF+AS*DL.TM)*BRF
      " COMPUTE A5 FROM THE FOLLOWING EQUATION
      'X L - ( X F + V F * D T + 0 . 5 * A S * D T ** 2 ) - L L - 3 -(UF+AS*DL.TM)*BRF
      LET A5=(YL-YF-UF*DL.TM-LL-3-UF*BRF )/(0.5*DL.TM**2+DL.TM*BRF)
      ALWAYS
   ELSE
      LET A5=(YL-YF-UF*DL.TM-LL-3-UF*BRF )/(0.5*DL.TM**2+DL.TM*BRF)
      ALWAYS
   ELSE
      LET BEE=2*UF*DL.TM+2*MX.F*DL.TM*BRF+HX.F*DL.TM**2
      LET CEE=-2*MX.F*(YL-YF-UF*DL.TM-LL-10-UF*BRF+UL**2/(2*MX.L))+UF**2
LET RAD = BEE**2 - 4 * CEE * DL.TM**2
IF RAD GE 0.0
LET A5 = (-BEE + SQRT.F(BEE**2 - 4 * CEE * DL.TM**2)) / (2 * DL.TM**2)

" AFTER FINDING A5 CHECK IF THE RIGHT HAND SIDE OF THE CONSTRAINT
" EQUATION IS GREATER THAN OR EQ TO (UF + A5 * DL.TM) * BRF. IF IT IS NOT
" THEN SET LEFT HAND SIDE EQUAL TO THIS AND SOLVE IT AGAIN FOR A5.

LET R.H.S = (UF + A5 * DL.TM) * BRF + (UF + A5 * DL.TM) ** 2 / (2 * MX.F) - UL**2 / (2 * MX.L)
IF R.H.S < (UF + A5 * DL.TM) * BRF
" COMPUTE A5 FROM THE FOLLOWING EQUATION
" XL - (XF+VF*DL.TM+0.5*A5*DL.TM**2)-LL-10*(UF+A5*DL.TM) * BRF
LET A5 = (YL-YF*UF*DL.TM-LL-10-UF*BRF) / (0.5*DL.TM**2+DL.TM*BRF)
ALWAYS
ELSE
LET A5 = (YL-YF-UF*DL.TM-LL-10-UF*BRF) / (0.5*DL.TM**2+DL.TM*BRF)
ALWAYS

" TO FIND A4 USE THE FOLLOWING EQUATION
" XL-(XF+VF*DL.TM+.5*A4*DL.TM**2)+LL+10
LET A4 = 2 * (YL-YF-UF*DL.TM-LL-10) / (DL.TM**2)

" THESE ARE ASSIGNED BECAUSE WE FIND MIN.F

" IF UF = 0.0 AND UL > 0.0
IF STR.DLY(VEHICLE) = 0.0
LET STR.TM(VEHICLE) = TRUNC.F(TIME.V)
ALWAYS
LET STR.DLY(VEHICLE) = TIME.V - STR.TM(VEHICLE) + DL.TM
IF STR.DLY(VEHICLE) >= MIN.F(3.0, 1.0 + BRT(VEHICLE) + 0.325)
IF A5 = 0.0
LET A3 = 0.0
ELSE
LET STR.DLY(VEHICLE) = 0.0
IF TY.FOL = 1
LET A3 = 2.0 "ASSUMED VALUE
ELSE
LET A3 = 1.0 "ASSUMED VALUE
ALWAYS
ELSE
LET A3 = 0.0
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS
ALWAYS

IF UF < D.SP
LET A2 = (D.SP - UF) / DL.TM "A2 IS +

IF TY.FOL = 1
IF UF > 0.0 AND UF < 22.0
LET A1 = 8.80
GO TO 2
ELSE
IF UF > 22 AND UF < 44.0
LET A1 = 5.50
GO TO 2
ELSE
IF UF >= 44.0 AND UF < 58.667
LET A1 = 5.17
GO TO 2
ELSE
    IFUF>58.667 AND UF<73.333
        LET A1=6.17
        GO TO 2
    ELSE
        IFUF>73.333 AND UF<88.0
            LET A1=3.08
            GO TO 2
        ELSE
            LET A1=2.09
            GO TO 2
    ELSE
        "VEH IS TRUCK"
    IFUF>0.0 AND UF<22.0
        LET A1=2.20
        GO TO 2
    ELSE
        IFUF>-22.0 AND UF<44.0
            LET A1=1.10
            GO TO 2
        ELSE
            IFUF>-44.0 AND UF<58.667
                LET A1=0.88
                GO TO 2
            ELSE
                LET A1=0.44
                GO TO 2
        ELSE
            "UF>D.SP"
            LET A2=(D.SP-UF)/DL.TM

COMPARING THE VALUE OF A4 AND A5 FOR DECELERATION INDICATES THAT
"CARFOLLOWING WITH NON COLLISION CONSTRAINT WOULD GIVE FASTER
"DECELERATION THAT JUST CAR FOLLOWING AS LONG AS THE RIGHT HAND SIDE
"OF NON COLLISION CONSTRAINT EQUATION IS POSITIVE . IN OTHER WORD,
"ABSOLUTE VALUE OF A5 IS GREATER THAN ABSOLUTEM VALUE OF A4
"(A5<A4 SINCE BOTH ARE NEGATIVE)
"ON THE OTHER HAND WHEN THE RIGHT HAND SIDE OF THE EQUATION IS
"NEGATIVE CAR FOLLOWING WITH NONCOLLISION CONSTRAINT GIVES SLOWER
"DECELERATION THAN JUST CAR FOLLOWING RULE. IN THIS CASE ABSOLUTE
"VALUE OF A5 IS LESS THAN ABSOLUTE VALUE OF A4(A5.A4 SINCE BOTH
"ARE NEGATIVE)
"NOTE THAT IN BOTH CASES WE WILL TAKE THE SMALLEST DECELERATION
"VALUE,THE FASTEST DECELERATIOPN.

ONE SHOUD CONSIDER THE REAL TRAFFIC BEHAVIOR AND MAKE SOME
"MODIFICATION ON THE ALGEBRAIC EQUATIONS TO REPRESENT THE ACTUAL
"SITUATION MORE ACCURATLY.DECELERATION OF VEHICLES ON THE
"HIGHWAY IS NOT BASED ON RIGID RULES BUT HUMAN DECISION. FOR
"EXAMPLE IF A2 IS LESS THAN A COMFORTABLE DECELERATION
"THERE IS A GOOD CHANCE THAT THE DRIVER WOULD NOT USE A2 TO DECELERATE
"BUT WOULD DECELERATE AT A COMFORTABLE RATE . THIS IS TAKEN INTO
"ACCOUNT IN MODEL BUILDING. CONSIDER THE FOLLOWING SITUATION

A2-------------------AC------------A5 USE AC
AC-------------------A2-----------A5 USE A2
AC-------------------A5------------A2 USE MIN(A5,A4)
A5-------------------AC------------A2 USE MIN(A5,A4)
A5-------------------A2-----------AC USE MIN(A5,A4)
A2-------------------A5------------AC USE MIN(A5,A4)

CALL COMF.DECEL(UF,TY.FOL)YIELDING AC
IF A2<AC AND AC<MIN.F(A4,A5)
   LET MIN.AXL=AC
   GO TO 3
ELSE
   IF AC<A2 AND A2<MIN.F(A4,A5)
      LET MIN.AXL=A2
      GO TO 3
   ELSE
      LET MIN.AXL=MAX.F(A5,-MX.F)
   END IF
GO TO 3

"2" LET A=MIN.F(A1,A2,A3)
IF A3>=0.0
   LET MIN.AXL=MIN.F(A1,A2,A3,A4,A5)
ELSE
   LET MIN.AXL=MAX.F(A5,-MX.F)
ALWAYS
"3" LET AXL=MIN.AXL
RETURN
END "ACCEL

SUB.PROGRAM COMF.DECEL(SP.F,TY.F) YIELDING AC

"THIS FINDS COMFORTABLE (OR NORMAL) DECELERATION RATE. INFORMATION
"FROM TABLE 2.7 OF THEI IS USED. THESE ARE NORMAL DECELERATION FOR
"FOR PASSENGER CAR; 75% OF THESE ARE USED FOR TRUCKS (THE RATIO OF
"NORMAL TO MAX FOR PASSENGER CARS IS APPROXIMATELY 75%)
IF SP.F<-22.0
   LET AC=-7.333
ELSE
   IF SP.F>22.0 AND SP.F<-44.0
      LET AC=-6.746
   ELSE
      LET AC=-4.840
   END IF
ALWAYS
IF TY.F=2
   LET AC=0.75*AC
ALWAYS
END "COMF.DECEL

SUB.PROGRAM DATA2(FSP,LSPEED,FPOS,LPOSITION,LAXL.MAX,FTYPE,DELTA.TM,K,
                   FID,LSP,LPOS,LLEN,TRJ.IN)

PRINT 1 LINE WITH TIME.V THUS
   DATA2  TIME=*****.***
RETURN
END

SUB.PROGRAM STS.LIST(K,FID,FTS.SP,FBRT,FTYPE,FSP,LSPEED,FAXL,LAXL.MAX,
                       FPOS,LPOSITION)
PRINT 1 LINE WITH TIME.W THUS
STATUS LIST TIME=****.****
RETURN
END

/*
//GO.SYSIN DD *
50
16
1.00
1.0
16
16
500.0
500.0
1200
0
600
600
16
1165804615
369470775
131478063
1804851546
780159578
21388690
1064018740
3
80.67
5
51.33
51.33
51.33
51.33
51.33
0.0
1200
/*
//SAS EXEC PROC=SAS
//SASDATA DD DSN=*.SIM.GO.SIMUO1,DISP=(OLD,DELETE)
//FLOTPARM DD DUMMY
//SYSIN DD *
GOPTIONS DEVICE=VERSATEC;
DATA SIMOUT;
INFILE SASDATA;
INPUT (QUE SEED RATE TYPE ID TIME POSITION SPEED ESPD) (S*184. 4*RB4.);
DATA ALL;
SET SIMOUT;
PROC SORT;
BY TYPE ID;
PROC PRINT;
PROC CPLOT;
PLOT TIME*ID=TYPE;
PLOT POSITION*ID=TYPE;
PLOT POSITION*TIME=TYPE;
SYMBOL1 C=BLACK L=1 I=JOIN V=STAR;
SYMBOL2 C=BLACK L=1 I=JOIN V=PLUS;
SYMBOL3 C=BLACK L=1 I=JOIN V=DIAMOND;
SYMBOL4 C=BLACK L=1 I=JOIN V=SQUARE;
SYMBOL5 C=BLACK L=1 I=JOIN ;
SYMBOL6 C=BLACK L=1 I=JOIN ;
SYMBOL7 C=RED L=2 I=JOIN ;

SYMBOL8 C-RED L=2 JOIN;
SYMBOL9 C-RED L=2 JOIN;
SYMBOL10 C-RED L=2 JOIN;
SYMBOL11 C-RED L=2 JOIN;
SYMBOL12 C-RED L=2 JOIN;
SYMBOL13 C-CYAN L=20 JOIN;
SYMBOL14 C-MAGENTA L=7 JOIN;
SYMBOL15 C-BLACK L=8 JOIN;
/*
COMMAND? 000
ARE YOU STILL THERE?


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