THE UNIVERSITY OF CALGARY

Two-Lane Highway Capacity and Simulation

BY

Calvin Wong

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled, "Two-Lane Highway Capacity and Simulation," submitted by Calvin Wong in partial fulfillment of the requirements for the degree of Master of Science in Engineering.

John F. Morrall
Supervisor, Dr. John F. Morrall
Department of Civil Engineering

Michel Sargious
Dr. M.A. Sargious
Department of Civil Engineering

A Werner.
Mr. A. Werner
Alberta Transportation

Date July 14, 1987

Abstract

Current capacity methods of two-lane two-way rural roads in the 1985 Highway Capacity Manual (HCM) cannot adequately account for minor improvements made to highways. A Unified Flow Theory has been proposed to examine such changes. This capacity method is based on the demand for overtaking and the adequate supply of gaps for overtaking in the opposing stream. To develop this theory, a rural road simulation model is calibrated and validated. Overtaking demand functions were then generated and related to the HCM's level of service criteria.

The simulation model, called TRARR *(TRAffic on Rural Roads)*, was calibrated and validated to acceptable error limits. Relational demand functions developed between the Unified Model and the HCM's percent time spent following criteria showed the effects of volume, sight distance, directional split and addition of passing lanes on level of service. Areas of further research are suggested to expand the Unified Model to cover grades and trucks in the traffic stream.

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Chapter 1

Introduction

1.1 General

An overview of research into capacities of rural highways shows work in this area to be slow in development. In the area of two-lane highway research, the first work done was in the 30's. Major research at that time was limited to work by Prisk (1941) and Normann (1942). The lack of activity in the 50's and 60's was probably due to the concentration of research on freeway operations. Recently though; there has been a resurgence of interest in two-lane highways. This has been brought on mainly by constraints on capital funds available to highway agencies and a more realistic attitude to growth of transportation networks. Highway agencies are now looking at interim solutions to capacity problems rather than more costly solutions, such as highway twinning. Solutions considered include changes to roadway geometry and the installation of passing lanes. In order to determine the impact of operational improvements the highway engineer requires capacity methods which are sensitive to these changes. Chapter 8 of the 1985 Highway Capacity Manual (HCM) deals with rural highway capacity. Unfortunately, it does not adequately account for traffic flow improvements due to minor low cost operational improvement techniques such as auxiliary lanes.

To allow the effectiveness of low-cost operational improvements on two-lane roads to be analysed, a new theory on rural highway capacity has been proposed by Morrall and Werner (1985). This concept hypothesizes that a driver's perceived level of service is related to his ability to pass slower moving vehicles. The demand for passing is related to the traffic composition while the supply of passing opportunities is related to gaps in the opposing traffic stream and road geometry. This concept of highway operations has been called the Unified Model as it attempts to unify the supply and demand functions. Conceptually, this model is capable of analysing and is sensitive to small highway improvements, however model functions were developed using limited data and still require calibration. In particular, more data is required on overtaking rates at various traffic volumes and directional splits.

Simulation modelling is often used as a substitute to data collection in the development of capacity methods. It provides a much faster way of assembling required data plus the added advantage of examining the effects of particular conditions specified by the model user. The investigator must be aware of the limitations of the model he is working with and its accuracy over the range of applicability.

The problem addressed in this thesis deals with the further development of the Unified Model of Two-Lane Highway Capacity, more specifically the quantification of the demand functions in the Unified Model need to be researched. Once these functions are determined, relationships can be made between it and HCM Level of Service guidelines. To develop this capacity method an Australian rural simulation model called TRARR is calibrated and validated.

1.2 Background

This research represents a continuation of the work on rural road capacity initiated by Morrall and Werner. The Unified Traffic Flow Theory Model has been developed to the point where it can be applied in system evaluations. The Unified concept has been used by Alberta's highway agency (Alberta Transportation) to analyse 300 sections of two-lane road (Kilburn 1985a). Average annual daily traffic was used to determine passing demand and compare it with the net passing opportunities. A ranking of unsatisfied demand was then made giving a prioritization of the road sections for further analysis and possible upgrading. The model is now at the point where further refinement of its traffic descriptive functions are required to make the model valid for microscopic applications.

During the summer of 1985, the Department of Civil Engineering at the University of Calgary obtained a microscopic rural road traffic simulation model from the Australian Road Research Board in Melbourne, Australia. The model is deterministic and simulates most traffic operations in great detail. It has been used in a number of road improvement evaluations (Hoban 1983). This model, called TRARR, was developed in the early 1980's and is now in the calibration and validation stages. The detail of modelling performed by TRARR along with its relatively straight forward structure makes it suitable for use in further development of the Unified Model.

1.3 Study Objectives

The specific objectives of this research can be stated as follows:

1. calibration and validation of the TRARR model to Canadian conditions
2. development of the demand functions for the Unified Model
3. relating the Unified model to the Level of Service concept presented in the Highway Capacity Manual.

It is expected that the calibrated simulation model should be of immediate use to provincial highway agencies in analysing rural roads with particular application in the examination of low-cost improvement strategies and staging of improvements.

It is noted that this research has been sponsored by Alberta Transportation.

1.4 Organization of Thesis

The remaining chapters will discuss the areas of traffic simulation and highway capacity in the following manner.

Chapter 2 is a review of previous work in the field of rural road simulation models including studies of the individual components of highway operations. An overview of current rural highway capacity methods is given.

Chapter 3 discusses the simulation model to be used in development of the Unified Traffic Flow Theory. Overall structure and data requirements are examined. Deficiencies and model refinements are presented.

Chapter 4 details the methodology used in calibrating and validating the simulation model.

Chapter 5 develops the Unified Model supply and demand functions through simulation experiments. Quantification of the model functions are related to current level of service criteria.

Chapter 6 discusses the calibration of the simulation model and elaborates on the development of the Unified Traffic Flow Theory.

Chapter 7 concludes this work with a summary and discussion of future research in this area.

Chapter 2

Literature Review

2.1 General

This literature review is divided into two distinct sections. First, a summary of North American highway capacity procedures is discussed. Second, a review of road simulation models and associated operational parameters is given. Examination of literature in these two areas should give a good background for the task at hand, the development of a highway capacity method through computer simulation.

2.2 Highway Capacity

The major rural road capacity guide used by engineers for the past twenty years has been the 1965 Highway Capacity Manual (HCM). This document served as the basis for two-lane highway planning throughout North America. The familiar concept of level of service (L.O.S.) was defined by the 1965 HCM. However, the capacity procedures given in the manual contained deficiencies in their development. As the decades past, and the demand on rural highway facilities increased, these deficiencies became more apparent. Many authors published papers on the inaccuracies of the 1965 HCM noting errors in the prediction of traffic operating characteristics. Interim manuals were produced by highway agencies and recently, the Transportation Research Board released Special Report 209, Highway Capacity

Manual (simply referred to as the 1985 HCM). This new manual presented major changes with regards to two-lane highway capacity.

The major change in the new HCM comes in the defining criteria for level of service (L.O.S.) which is now specified by percent time delay. From Chapter 8 of the 1985 HCM:

> Percent time delay reflects both mobility and access functions, and is defined as the average percent of time that all vehicles are delayed while travelling in platoons due to the inability to pass. "Percent time delay" is difficult to measure directly in the field. The percent of vehicles travelling at headways less than 5 seconds can be used as a surrogate measure in field studies.

The use of this criterion is an improvement over the 1965 HCM. The volume/capacity ratios used in the previous manual were inappropriate for measuring levels of service as rural roads are not designed to operate at capacity. The speed criterion in the 1965 HCM was also inappropriate. Research has shown that speeds are relatively insensitive to volume changes (Yagar 1983) when compared to the speed/volume relationships presented in the 1965 HCM. Percent time delay corrects these faults. It is more sensitive to changes in volume, in fact is more of a measure of the driver's perceived level of service on a roadway.

A number of authors (Hoban 1986 and Morrall 1986a and Werner 1986) have noted the deficiencies in the new HCM analysis procedures. The HCM bases its procedures on equilibrium conditions and cannot account for the complete benefit of minor roadway improvements. For example, the benefits from a short passing lane section may still be visible four to eight kilometres downstream. The concept of percent time delay requires more research in relation to traffic flow conditions.

A new approach to two-lane highway capacity and L.O.S. is the Unified Traffic

Flow Theory Model proposed by Werner and Morrall (1985). This concept is more realistic in its analysis as it measures a driver's perceived mobility and subsequent level of service. Evaluation techniques used can examine the effects of low-cost minor improvements to rural two-lane roads, an example of such an improvement being the addition of passing lanes.

The Unified Model is based on a supply/demand concept. The supply of overtaking opportunities is related to the demand for overtaking. The effects of road geometry and addition of passing or climbing lanes on platoon building is shown in Figure 2.1.

The current development of the Unified Model does not allow for fully calibrated use. Relationships need to be derived between the supply/demand concept and the L.O.S. criteria used in the H.C.M. In particular, the demand for overtaking or the actual overtaking rates incurred for various operating conditions need to be quantified. The Unified Model is more fully discussed in Chapter 5.

2.3 Traffic Simulation

2.3.1 General

The area of rural road simulation modelling encompasses a large number of specific topics making a complete literature review impractical. In developing a model, one must investigate the various operational parameters that make the total picture. A review of research on speed and headway distributions, the overtaking maneuver, merging and vehicle performance modelling are all required. Each topic is in itself a major study area and to fully examine work in each area is beyond the scope of

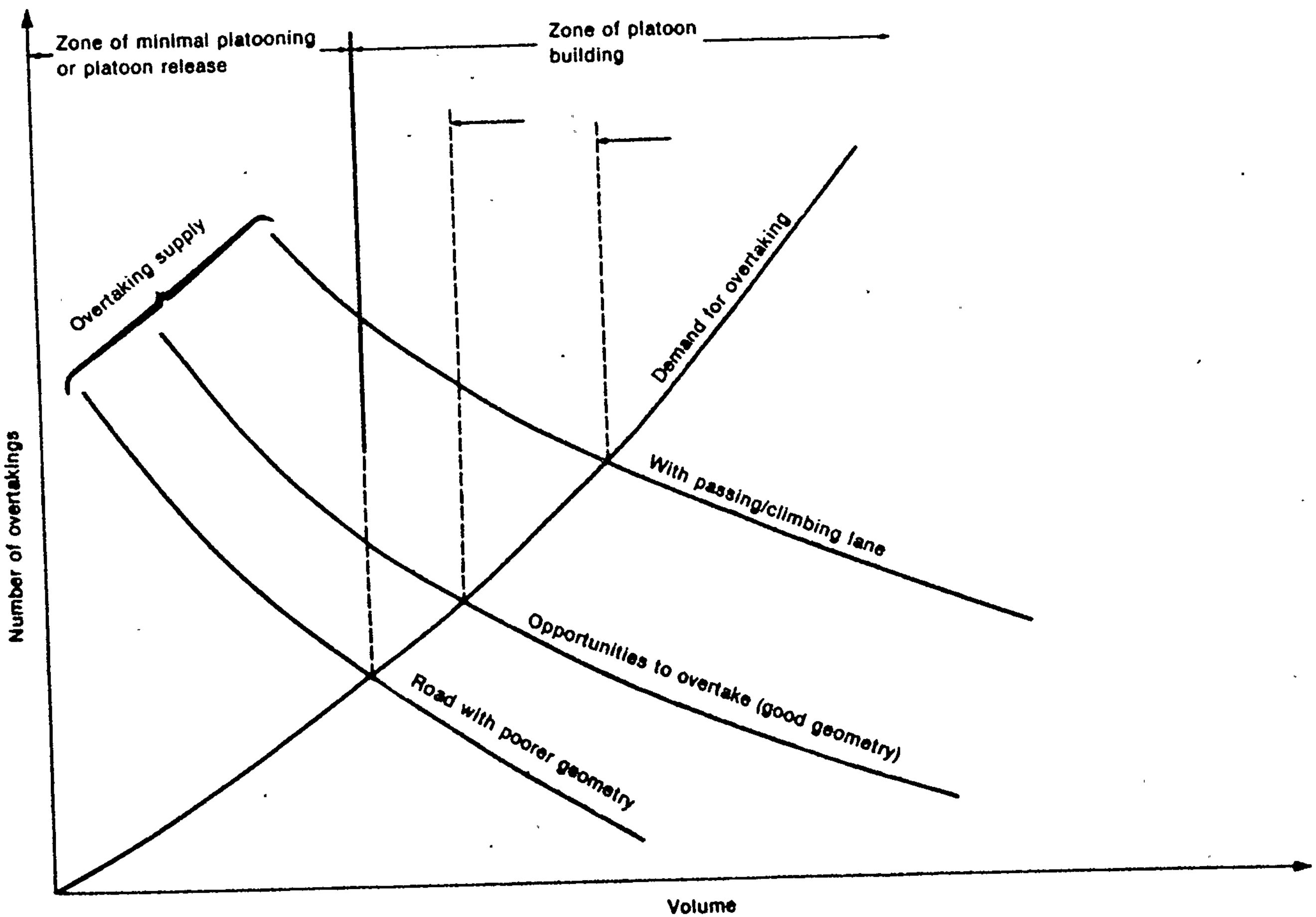


Figure 2.1: Concept of the Unified Model

(Werner and Morrall 1985)

this review. Rather, a more efficient approach is to review the literature for major and current work and trends in each category, to solicit and formulate opinions on the best methods of analysis.

2.3.2 Speed Distributions

Traffic modelling techniques require information on both the desired speed of vehicles and the distribution of these speeds. Nationwide speed trends for the United States are given in the 1985 HCM. A major emphasis is given to the effect of the fifty-five mile per hour speed limit introduced in 1974 as a fuel saving measure. General findings showed that prior to the 55 mph limit, average speeds for cars and buses differed from trucks by a constant 11 to 12 km/h. After the speed limit implementation, differences in average running speed fell to 4 km/h. This could be due to the combination of effects. First, less variation in desired speeds around a lower speed limit and second, the emergence of a more powerful truck fleet able to sustain speeds similar to cars could explain the speed trends.

McLean (1983) gives typical speed distributions from four separate studies (Figure 2.2). Overall conclusions drawn from comparisons suggest that normal distributions represent vehicle speeds very well. Morrall and Werner (1985) studied speed distributions along the Trans-Canada Highway. Differences in passenger car and truck speeds were found to be negligible. This data collected in 1980, depicted more uniformity in speed as compared to the 1965 HCM, confirming the findings of the 1985 HCM.

McLean (1981) studied the effects of road curvature on vehicle speed and found strong relationships between the two. McLean proposed that speed on a curve was

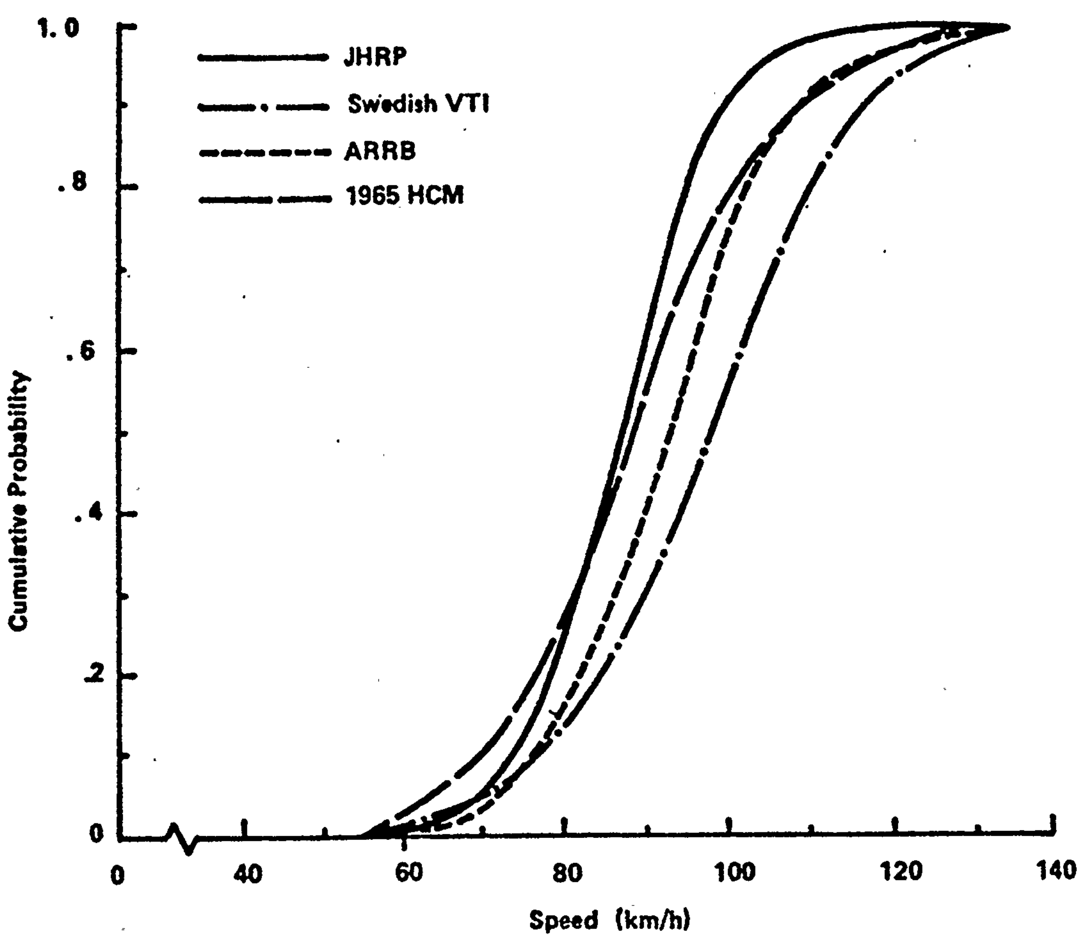


Figure 2.2: Various Speed Distributions

McLean (1982 b)

related to both the horizontal radius of the curve and the design speed or "*speed environment*" of the roadway section. Van Aerde and Yagar (1983a) developed a multiple linear regression model to determine speeds on two-lane roads as a function of environmental and geometric factors. They did not find any statistically significant effect due to curvature but hypothesized that that this was due to lack of data or colinearity with other factors such as speed limit and sight distance. The authors found the following factors, in decending order of importance, had an effect on speed; land use adjacent to the road, speed limit, grade, access and lane width. In an accompanying paper, Van Aerde and Yagar (1983b) found speeds to be insensitive to volumes over normal ranges.

2.3.3 Headway Distributions

Mathematical descriptions of headways are desired in the simulation modelling process. One has the option of either using an appropriate headway distribution model or to read in actual headway data. Input of actual or synthesised data has the disadvantage of being labour intensive as well as being computationally inefficient, but it does allow investigation of specific traffic situations. However, a mathematical headway model allows greater flexibility and its ease of use makes it applicable to traffic modelling purposes.

Tolle (1971) examined and tested a variety of headway distributions. He found the log-normal distribution gave the best fit over general conditions while the composite exponential and Pearson Type III distributions also gave good results.

The basic headway model is the negative exponential (N.E.) distribution. Using this formulation, the probability of a headway, h, equal to or greater than a time t, can be stated as:

$$P(h > t) = e^{\frac{-t}{T}}$$

where T is the mean of the interval distribution. The model is primitive as it predicts too many short headways (Gerlough and Huber 1975). A modification to the N.E. distribution was developed by shifting the N.E. function to prohibit small headways. Gerlough and Huber(1975) found that the shifted N.E. distribution fitted data for low flows but did not accurately model higher flow rates.

Schuhl (1955) developed a composite exponential headway model which described two distinct traffic populations, free flowing vehicles and those constrained in platoons. The model is a composite of shifted and unshifted exponential distributions.

2.3.4 Overtaking

The overtaking maneuver is the most complex event on a rural two-lane highway. It is the main characteristic that sets apart two-lane highways from freeways. Overtaking is the key to maintaining a high operating standard and level of service. Because overtaking requires travel in the opposing lane of traffic, judgement must be made by the driver. Judgements and skill levels of drivers are heterogeneous throughout the population, therefore are difficult to model. For any simulation model used in capacity and level of service determinations, the overtaking maneuver must be accurately reproduced. The intricacies of overtaking can include many steps requiring a general breakdown of the maneuver.

The components of a typical overtaking can include the following elements: catchup, following, decision to overtake, acceleration, passing, merging, move to a new free or following speed. The overtaking maneuver itself has been defined by various authors (Prisk 1941, Ahman 1972, Troutbeck 1981) as starting when the overtaking vehicle first crosses the centerline and ending when the vehicle is clear of the opposing lane. The basic parameters are shown in a time-space diagram presented in Figure 2.3. Troutbeck (1981) studied overtaking rates on Australian two-lane highways. Regression equations were developed which relate mean speed and variance of the overtaking vehicle to the length and speed of the overtaken vehicle. Troutbeck then used the equations to calculate overtaking sight distance and the effect of no-passing zones.

2.3.5 Gap Acceptance Theory

One method used in modelling of overtaking is gap acceptance theory. The overtaking decision is basically a choice of whether or not to accept an offered gap in the opposing traffic stream to pass a slower vehicle.

Crawford (1963) performed controlled experiments to obtain threshold values of required distance for overtaking as related to relative speed of the overtaken and oncoming cars. A linear relationship was found for what the author termed the threshold interval. Threshold conditions are defined as the set of conditions where half the time a driver will overtake and the other half of the time, the driver will reject overtaking. This is an example of an inconsistent driver model. Each driver is assumed to have a variable response to a given set of stimuli. Ashworth and Bottom (1977) found that this type of model better represented driver behavior in their studies of right turn movements. A consistent driver model has been used in studies by Miller and Pretty (1968) in which driver reactions to a given set of

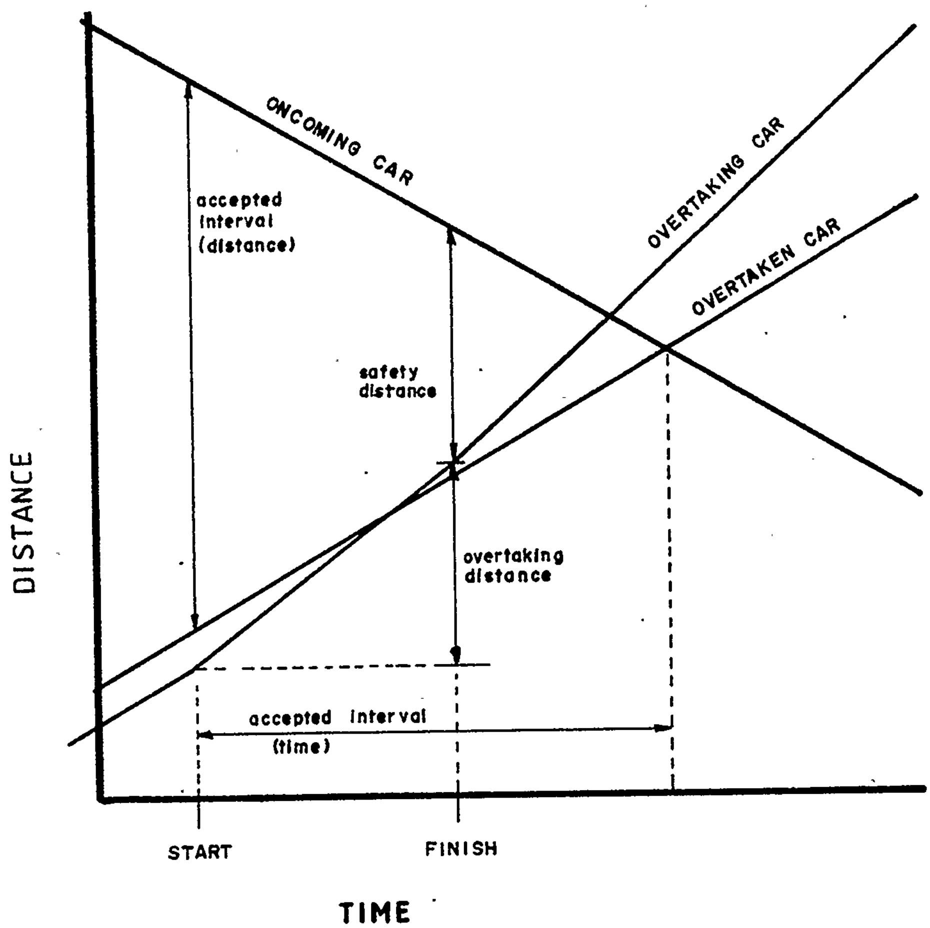


Figure 2.3 : T - S Diagram of Overtaking

stimuli do not vary.

2.3.6 Merging

Simulation of the merging event is important in the modelling of passing lane operations. The merge area is viewed by some engineers as a problem in terms of operations and safety (Hoban and Morrall 1985). A review of the literature has shown very little work in passing lane merging. This is to be expected as the concept of passing lanes is relatively new and their acceptance by all highway engineers is somewhat tentative.

Morrall and Blight (1984) monitored experimental passing lanes in Banff National Park to study their operation and develop design guidelines. Their findings suggested a merge taper length of 200 metres at the drop of a passing lane. This length was required to allow the merging of platoons rather than single vehicles. Harwood et al (1985) supported the idea of longer merge taper lengths in studying accident rates at a number of passing lane sites in the U.S. and found no safety problems in the merge area. Their appears to be qualitative analysis of merging at a passing lane termination but little detailed modelling of the situation.

There are a number of studies dealing with merging from a minor road to a major road. Drew (1967) used probit analysis to develop linear relationships between gap acceptance and gap size for freeway mergings. Merge models were developed to account for relative and absolute speeds of mainstream and merging traffic, varying points of entry, outside lane volumes and platoon merging.

2.3.7 Vehicle Performance

Reviews by Hoban and McLean (1982) and Botha et al (1980) revealed that there are well established theories of modelling vehicle performance. However, in simulations, the methods employed in calculating vehicle performance and acceleration vary widely. Models evaluated by Botha et al used force equations, Society of Automotive Engineers (SAE) practices, performance curves and empirical data in their analysis.

One of the most detailed studies into vehicle performance was written by St. John and Kobett (1978). Vehicle movement was described by comprehensive equations which accounted for gear and axle ratios, vehicle weight, brake horsepower and air and rolling resistance. Werner and Morrall (1976) developed passenger car equivalents for trucks, buses and recreational vehicles for rural highways. The 1985 HCM includes detailed procedures for determining truck performance on grades.

2.4 Simulation Models

2.4.1 Early Developments

Rural road simulation models were slow to develop due mainly to the complexity of modelling the traffic characteristics. Rural highway operations are very complex and modelling them accurately requires a great amount of computing power which has only recently been available to the general transportation profession. As a consequence, early simulation models were somewhat simplistic in their approach and made a number of assumptions to ease computational requirements.

One of the earliest two-lane simulation models was proposed by Shumate and

Dirksen (1964). They began development of a language to simulate rural road traffic flow. The model accounted for arrival times, vehicle lengths, speeds, accelerations and driver "valor". This original model presented some of the concepts in two-lane simulation and tried to relate operational characteristics to overall highway performance.

Warnshuis (1967) developed a very simple model. It allowed modelling of a long straight road as a circular track. Simple overtaking, acceleration and headway submodels were used. The model's simplified approach did not allow for any practical applications. The model was never calibrated. Boal (1974) formulated another circular road model which was further developed by Luk (1976). The major objective of this model was to study the overtaking maneuver and to identify the critical parameters that govern. They considered only sight distance as a geometric factor in overtaking.

2.4.2 Detailed Simulation Models

Cassel and Janoff (1968) developed one of the first detailed simulation models at the Franklin Institute Research Laboratories (FIRL). This model (SIMMOD) was developed to evaluate traffic flow and safety benefits arising from improvements to overtaking and passing maneuvers. Roadway geometry is specified by no-passing zones and sight distance restrictions for each direction of travel. "Slow-down" factors are used to account for speed reduction due to curves and grades. Speed and headway distributions are predetermined by the user.

This traffic model can be broken down into four main sections. The main subroutine calculates output statistics, updates positions and speeds and prints results. The maneuver subroutine determines the action of each vehicle at each time increment. Vehicles may be given any one of four possible maneuver states. The overtaking logic is based on probability distribution curves developed at FIRL. Overtaking probability was found to be dependent on oncoming gap and lead car speed. Given these two independent variables, the probability of passing is determined by linear interpolation of the developed curves. The speed subroutine calculates each vehicle speed as a function of present and next maneuver state. Finally, the accident subroutine checks if corrective action should be taken to safely complete the pass or if the pass should be aborted.

The SIMMOD model requires three sets of input data. Road data consists of road length, no passing zones, sight distance restrictions and time of simulation. Vehicle data required includes desired speeds, maximum speeds, arrival headways and maneuver state. Acceleration/deceleration rates were obtained from the Traffic Engineering Handbook (1976). Speed distributions were determined from observational studies. Headways, in the form of a modified Poisson distribution, were taken from the 1965 Highway Capacity Manual. Passing probability data consists of four curves developed from observational studies. Curves give probability of passing as a function of lead car speed and oncoming gap size. The probabilities will change with changes in the roadway design. A validation and reliability check of the model was made by comparing simulation results to observations made by Normann (1942).

A modification of the FIRL model was carried out by Heimbach et al (1973) at North Carolina State University. Subroutines were added to simulate vehicle performance on grades and better speed and headway generation methods were employed. Field data showed a Schuhl headway distribution model to best fit observed data.

Further work by Wu and Heimbach (1981) gave rise to the Simulation of Vehicular Traffic (SOVT) model. This model allows for the user to specify any percentage distribution of five vehicle types. Individual vehicle type acceleration/deceleration and speed distributions are defined by the user. Speed restriction zones can also be specified to denote sharp horizontal curves, narrow roadway width, school zones or sight distance restrictions. Vertical grades and no-passing zones are also specified. A major modification was the addition to the simulation of effects of minor cross roads. Up to eight minor stop-controlled intersections may be placed on the simulated roadway. The user can specify the vehicle volume and composition of the turning movements. The model also allows the addition of passing bays and climbing lanes on the road. The authors did not perform a validation of the model with field data, but rather ran the simulation for a hypothetical roadway section and checked the reasonableness of the results.

St. John and Kobett (1978) developed a very comprehensive rural highway simulation model at the Midwest Research Institute (the Midwest Model). The primary purpose of their work was to study traffic flow on grades and determine passenger car equivalency factors for various vehicle types. Detailed equations of vehicle motion on grades were developed accounting for vehicle speed, acceleration, power, mass, rolling resistance and air resistance. Vehicles enter the simulation via Schuhl headways plus a warm-up zone. The overtaking logic is based on the gap acceptance probability relationships developed at FIRL. A driver "workload" factor has been incorporated to reduce desired speed with the increase in the number of opposing vehicles met. Botha et al (1980) have modified the Midwest model to allow simulation of passing lanes.

Stock and May (1976) developed a Monte Carlo rural road simulation model. This model was used to analyse two-lane evaluation techniques in the 1965 HCM. The SIMTOL (Simulation of a Two-Lane Road) model considers only two vehicle types, cars and trucks. Recreational vehicles cannot be modelled. This model makes a number of limiting assumptions. Due to lack of calibration data, trucks are assumed not to pass. The model is limited to roadway sections of high design standards as speed reductions on horizontal curves are not modelled. This makes the model unsuitable for analysis of highway improvements as the effects of poor road geometrics cannot be shown.

A major limitation lies in the simulation methodology. Only one direction of travel is explicitly simulated. In the other direction, vehicles arrive at random headways but at a constant speed. Sight distance restrictions are not explicitly specified but are reflected in the specification of passing and no-passing zones. Overtaking is based on gap acceptance probabilities developed at FIRL and Swedish studies.

The Swedish National Road and Traffic Research Institute (VTI) has been developing a rural road simulation model since 1969. It has been used to revise design standards for climbing lanes and analyse improvements on existing roads. The VTI model is capable of handling cars and three classes of trucks. Vehicles enter the simulation according to specified platoons or shifted exponential headways. Overtaking is again modelled using gap acceptance probabilities. Vehicle free speeds are modified for road width, horizontal curvature, speed limits and grades. The VTI model has been used in many roadway improvement studies and has been validated with much data, however the model has a drawback in its user-friendliness. The model is written in SIMULA-67. While SIMULA is an efficient language in modelling applications, it is difficult to use and its limited availability on mainframe computers makes the portability of the VTI model constrained. The available English documentation of the model is also limited.

Palaniswamy et al (1984) modified the VTI model for use in evaluation of the Indian highway system. Unlike Western countries, homogeneous traffic flow conditions are seldom prevalent. In India, vehicle characteristics, such as size, weight, width and speed vary greatly. The majority of Indian roads are single lane containing a traffic mix ranging from bullock carts to heavy trucks. The authors adjusted the VTI model to allow study of the impact of road width on various traffic overtaking scenarios. The modified model was calibrated and validated with very good results.

Kaesehagen et al (1978) developed a model in Australia as part of a Brazilian highway economics study. The model, called SOFOT, is macroscopic using generalizations of traffic relationships. The model contains simplifying assumptions in the overtaking submodel which may make its use in evaluating auxiliary lanes inappropriate (Botha et al 1978). SOFOT was intended to determine costs in conjunction with traffic volume and composition for different design alternatives. The model is simple but is backed by a large database for calibration of the submodels.

Another macroscopic model was developed by Sananez and May (1983). This model, RURAL1, is basically a computerized version of the capacity methods to be presented in the 1985 Highway Capacity Manual. The model has the advantages of being inexpensive to run, modular formulation and a relatively small data requirement compared to microscopic models. The model's disadvantages are its limited accuracy and lack of interaction between subsections. This model cannot be used to simulate passing lane conditions or intersections and can only be used in situations where the demand on the facility is less than capacity.

A major modelling effort was initiated by the World Bank in 1969 to study the effects of operating costs on low-volume roads. The resulting work culminated in the release of the Highway Design and Maintenance Standards Model (HDM). Since its original use in 1977, it has undergone further development and is currently released as version three (HDM-III). HDM-III incorporates speed relationships based on work conducted in Kenya (Hide et al 1975), the Caribbean (Morosiuk and Abynayaka 1982), India and Brazil. The speed prediction models in these studies were evaluated by Bennett (1985). He found relationships developed from the Brazil data base to be most suitable for developed countries. Comparison of the various models showed that the Brazil relationships best accounted for the effects of curvature, gradients and surface roughness on vehicles speeds.

2.5 Summary

The review of simulation modelling techniques has shown that there are a variety of approaches to reproducing traffic flow on two-lane highways. Macroscopic models available are inexpensive to run and have small data requirements when compared to most microscopic models. The macro-models reviewed, though, were found to be inappropriate for the objectives of this project. Detailed traffic analysis cannot be generated. Some of the micro-models reviewed may be appropriate for this purpose with some additional development but in their current state are either too simplistic, cannot model all pertinent aspects of traffic flow or are not user-friendly. The Midwest model developed by St. John and Kobett could be used in this project as it models traffic flow in enough detail but its algorithm logic does not make it easy to examine or modify.

The TRARR model discussed in the next chapter was found to be the most suitable for the purpose of Unified Model development.

Chapter 3

The TRARR Model

3.1 Introduction

One of the most flexible rural traffic simulations is the TRARR model. The TRARR model was originally developed by Dr. Geoff Robinson at the Australian Road Research Board between 1978 and 1980. Since that time, the model has been updated and improved by Hoban et al (1985) with new features and options being added. The most recent release of TRARR, version 3.1, was made available at the end of 1986. TRARR stands for *TRAffic on Rural Roads.*

TRARR is a detailed microscopic simulation allowing for very specific modelling of rural highway operations. The program is written in FORTRAN 77, allowing it to be run on most mainframe and personal computers. In North America, TRARR is being experimented with by two Canadian highway agencies and a number of research institutes. The potential use in this model justifies current work in its calibration.

The main reason for the great interest in TRARR is its flexibility in modelling rural traffic operations. TRARR accounts for most parameters of interest in evaluating highways and highway improvements. As an introduction, TRARR allows traffic composition to be made of up to eighteen vehicle types. Highway barrier line markings, horizontal curvature, grade and sight distance are all accounted for in roadway geometrics. Output options allow the user to specify points on the road to collect information. Passing lanes can also be modelled. Similarily, passing bays, very short auxiliary lanes used by slow trucks, are also included. Output options allow for varying methods of display; animation, time-space diagrams, fuel consumption information or extended speed, overtaking and platoon size data.

3.2 Data Requirements

The flexibility of TRARR is reflected in its data requirements. Input is needed to cover the modelling situations of interest. TRARR requires major data input in four areas, driver/vehicle characteristics, road geometrics, traffic composition and observational requirements. Each area is discussed separately. Sample input data files are given in Appendix A.

3.2.1 Driver/Vehicle Characteristics

Driver/vehicle characteristics are contained in the input data file VEHS. Sixty characteristics are specified for eighteen vehicle types. Vehicle performance factors include acceleration/deceleration, power, weight and vehicle length. Driver behavioural characteristics are modelled explicitly by obeyance of barrier line markings, safety factors, aggression numbers and true/false flags on overtaking restrictions.

The eighteen vehicle types shows the flexiblity of the model. In areas such as the National Parks, road simulations would require modelling of not only a varied vehicle composition, but also a greater variation in driver behaviour. The size of the VEHS file accomodates this. However, the large number of variables greatly increases the difficulty in calibrating the model. Assemblage of all the vehicle characteristics constitutes a major effort.

3.2.2 Road Geometry

The TRARR model requires very detailed road geometry information. For a specified unit length of road, commonly 100 metres, the following information must be given for each direction of travel:

- barrier lines
- presence of passing lanes
- sight distance
- grade
- road speed indices

The road speed index is used to modify vehicle speeds due to changes from the ideal unconstrained situations. The road speed index can be used to account for roadway constraints such as horizontal curves, narrow pavements, speed limits and other speed limiting factors. The indices are related to speed multipliers which can reduce or increase a vehicle's desired speed. The work is based on research done by McLean (1981) on what he has labelled *speed environments*. He has shown that speed on curves depends on the overall perceived desired speed over a highway section. Current use of road speed indices is limited to horizontal curves but could also be extended to narrower pavement width, speed limits and single lane to passing lane transitions.

Both the road speed index for curves and the highway grades can be obtained from as-built plans. Grade is given as a percentage for one direction and assumed to be the negative value in the other direction.

Barrier line markings and sight distance are best determined by actual inspection of the road. The method employed in this study used videologs of the highway sections prepared by Alberta Transportation. This method of data collection has proven to be a very efficient means of rapidly assembling and checking this data file. Field surveys were also performed to ensure the accuracy of this input file.

3.2.3 Traffic Composition

The input data file TRAF contains run time parameters and traffic composition information. Time of simulation, warm-up period and warm-up zones are specified in this file. The overall traffic composition can be made up of any proportion of eight different traffic streams. These streams might be cars, passenger trucks, recreational vehicles or buses. Further, each stream is defined by probability distributions of the eighteen vehicle types noted in the vehicle characteristics input file. For each traffic stream, two-directional volume, directional split, mean desired speed and standard deviation of desired speed are specified. The arrival probability distribution is also selected in this file. The user can select normal, lognormal or other distributions specified by histograms in the model.

3.2.4 Observational Requirements

Another example of TRARR's flexiblity is demonstrated in its presentation of simulation results. The model allows the user to specify what type of data should be collected and at what points along the simulated roadway segment to collect it. The user may request frequency distributions of speeds, overtakings, platoon size, travel times, overtaking rates or very detailed output of position and time and/or fuel consumption, speed and acceleration at every time unit for every vehicle.

By allowing the user to specify at what points on the road to collect information, the simulation can very efficiently be used to examine the effect of roadway modifications. Currently, model output data is collected at 1000 metre intervals along a simulated road with interval spacing being reduced to 100 metres at the start of a passing lane. More detailed output can be collected at the start and end of a passing lane.

3.3 Computational Requirements

TRARR is a very portable simulation model. It has been run successfully on a number of mainframe and microcomputers. At the University of Calgary, TRARR simulations were carried out on the CDC Cyber 175 mainframe computer and the CDC Cyber 205 SuperComputer. Initial attempts to vectorize the TRARR source code on the Cyber 205 were met with some success. Performance of the program increased substantially when compiled using two levels of optimization and the vector preprocessor, VAST (Pacific-Sierra Research Corporation). Timings from various computers are shown in Table 3.1.

The comparisons of computational time requirements were made using a standard TRARR run. Test conditions of the benchmark data set were a 11 kilometre road section accomodating a total traffic flow of 537 vehicles per hour with 10

Table 3.1: System Performance Evaluation

System		Time (sec)
IBM XT		3420
IBM AT Compatible		1320
CDC 175		51.1
CDC 205	OPT = 0	18.2
	OPT = 1	13.2
	VAST	13.2

percent of the traffic stream being heavy trucks. The total simulated time was approximately 4000 seconds.

The Cyber facilitates the use of a Cyber Instruction Analyser (CIA) to give an indication of where the program is spending its computational time. This information can be used in two areas. It gives a ranking of subroutine importance and also indicates the best place to start making the program more efficient. CIA output shown in Table 3.2, indicates three subroutines to use most of the computational time, MANVR, POSIT and OBS. The MANVR subroutine simulates driver actions. It is described in a following section. The POSIT subroutine updates vehicle movement while OBS handles the output of data generated by TRARR.

3.4 Program Structure

The TRARR model is written in FORTRAN and consists of a main program which calls various subroutines. A simplified flow chart is given in Figure 3.1. The subroutines can be classified into six groups: initialization, vehicle simulation

Table 3.2: TRARR Subroutine Time Usage

Subroutine	Percentage Time
SETRR	0.2
SETGT	0.0
ENDS	0.4
ENDGT	3.4
GEN	1.1
GNVPL	0.4
PARM	0.0
MANVR	19.4
IOV	1.7
POSIT	27.7
REORD	5.6
LVOPP	3.4
RAND	0.0
RND12	0.4
CCM	0.0
SVTLO	1.1
SELECT	0.1
SETOB	0.0
OBS	30.1
FINAL	0.0

and updating, accounting procedures, observation routines, and final output. Of these groupings, the vehicle simulation and updating obviously contain the greatest detail.

3.4.1 Traffic Generation

TRARR allows two methods of traffic generation. The first method reads explicit speeds and arrival times for each vehicle. The user is offered the flexibility of replicating any traffic condition he wishes. This method though, is labour intensive and requires collection of input data. TRARR is usually run using its own internal traffic generation routines. The user specifies flow and stream probabilities for each of the eighteen vehicle types. TRARR then generates platoons from this data. Platoon size is given by the Borel-Tanner distribution:

$$P(I) = \frac{(Fe^{F*I})^{I-1}e^{-F}}{I}$$

where : F = percent of vehicles following in platoons

I = platoon size

P(I) = probability of platoon size I

The slowest vehicle in each platoon is moved to the front. Headways within platoons are set at a mean value. They will be modified once the platoon begins travel on the simulated segment. Headways between platoons are randomly generated. By inputting the level of platooning at the start of a road, the user can generate the required traffic situations of interest.

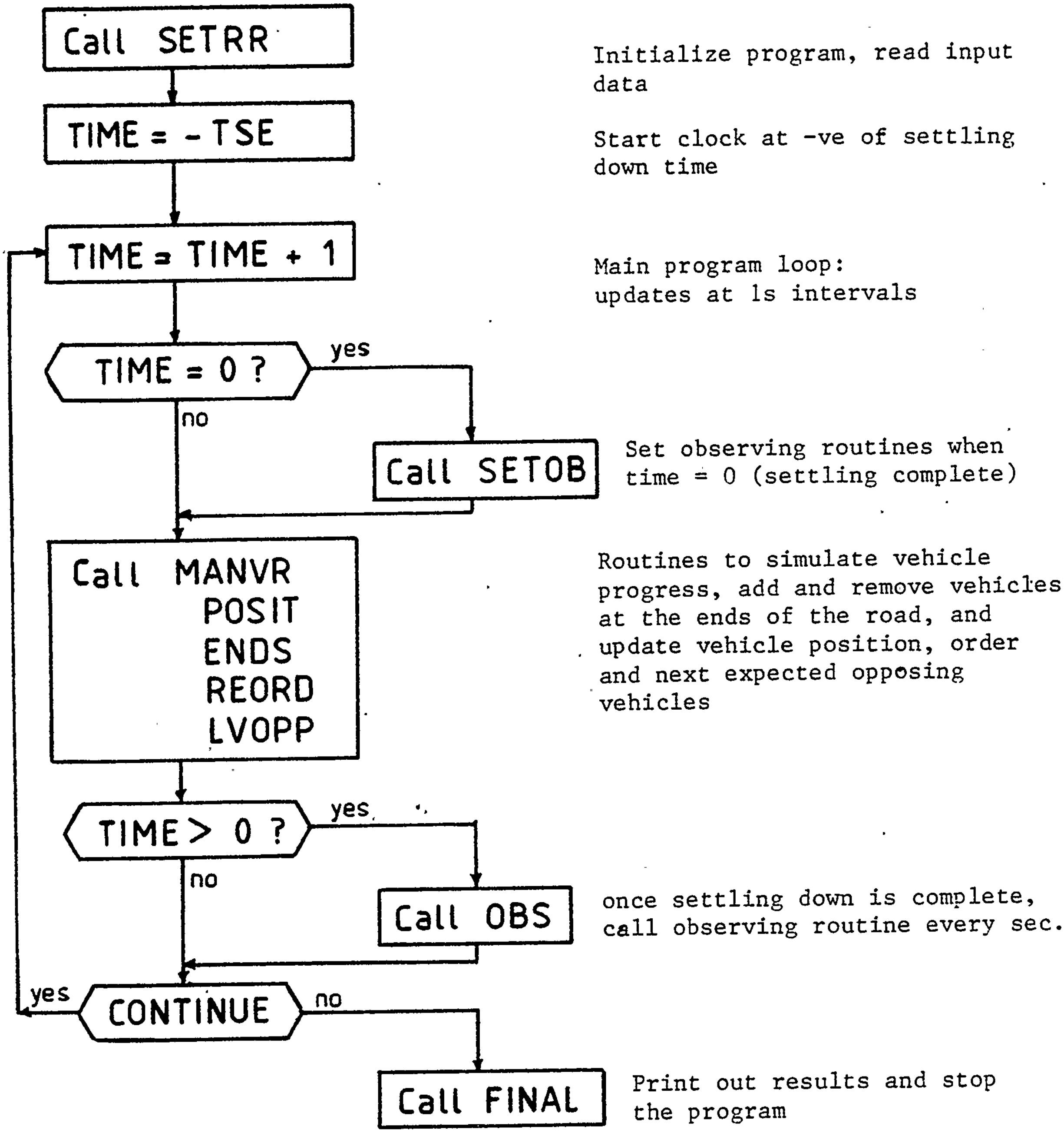


Figure 3.1: Simplified TRARR FLOWCHART

Hoban et al (1985)

3.4.2 The Maneuver Subroutine - MANVR

The bulk of the simulation complexity lies in the maneuver subroutine. Here, the program examines the state of each vehicle and determines its actions. To ease following of the program logic, the subroutine is written as a number of blocks, each block describing a specific state the vehicle is in. These states or maneuvers, represent a vehicle's current following, overtaking and merging situation. As each vehicle is examined, it is given a maneuver number. Each time unit, this maneuver number is re-examined to determine if the vehicle should change states. The blocks of the maneuver subroutine cover the following areas:

- vehicle in free state
- following in basic lane
- following in overtaking lane
- overtaking
- merging after overtaking
- merging at end of an auxiliary lane

3.5 Deficiencies of the Model

TRARR is like all other simulation models in that it only approximates the real situation. The accuracy of the approximations made within TRARR depends on a number of factors. Often limited data collection forces general assumptions to be made. Modelling of some circumstances can be so complex that simplifications must be used due to limited knowledge. The point is that simulations do not have to exactly replicate the real condition but be accurate enough to give reasonable results. It is usually sufficient to predict traffic conditions at a level of accuracy equal to other estimated parameters in the total transportation planning process.

TRARR has a number of simplifying assumptions that may require modification or calibration for better results. These model deficiencies will require examination to determine if they are the cause of descrepancies between observed and simulated traffic flow. Some of the deficiencies of TRARR have been noted by Hoban et al (1985).

1. Modelling of vehicle performance is not that precise. Gear changes and rolling resistances are not modelled. Maximum vehicle performance is often used unneccessarily.
2. A consistent driver model is used. Actions of one driver type are constant.
3. Drivers are assumed to have a perfect knowledge of the characteristics of nearby vehicles. Drivers are also given a zero reaction time. Both these factors will overestimate driver performance.
4. Driver behaviour is independent of traffic intensity
5. Obeyance of overtaking restrictions for the start of a pass also are enforced for the completion of the pass.
6. The road is modelled in discrete units with no smooth transitions between sections.
7. Lane change time is not modelled.
8. Deceleration on downgrades are not modelled. This is particularly important for the examination of the effect of trucks in the traffic stream.
9. To determine car following, only the lead vehicle's speed is examined, not acceleration.

3.6 Modifications to the Model

A number of minor modifications and additions were made to TRARR to customize it for use in this project. These changes are more cosmetic rather than changes to the program logic.

The output of the model was revised to give the distribution of gap sizes in the traffic stream at a specified point. This allowed for a check of the model's ability to reproduce functions inherent to the Unified Traffic Flow Theory.

To assist in analysing TRARR output, a short BASIC program, RPLOT was written. RPLOT reads the standard TRARR output file and produces the results in graphical form (Figure 3.2). These plots allow for easier analysis of simulation events over the entire road length as well as an indication of the interaction between the opposing traffic streams.

Finally, a FORTRAN program was written to allow plotting of time-space diagrams produced from TRARR extended output files. The program is site specific to the University of Calgary's computer graphics system and has limited portability to other computer sites. An example of a T-S diagram generated by TRARR is given in Figure 3.3.

3.7 Summary

The TRARR model has shown to be an excellent choice for this simulation study. Its flexibility allows for accurate modelling of the roadway situations in question. In a research environment, the time and resources are available to properly examine and calibrate the model. TRARR's modelling is at a high level but it is still one of

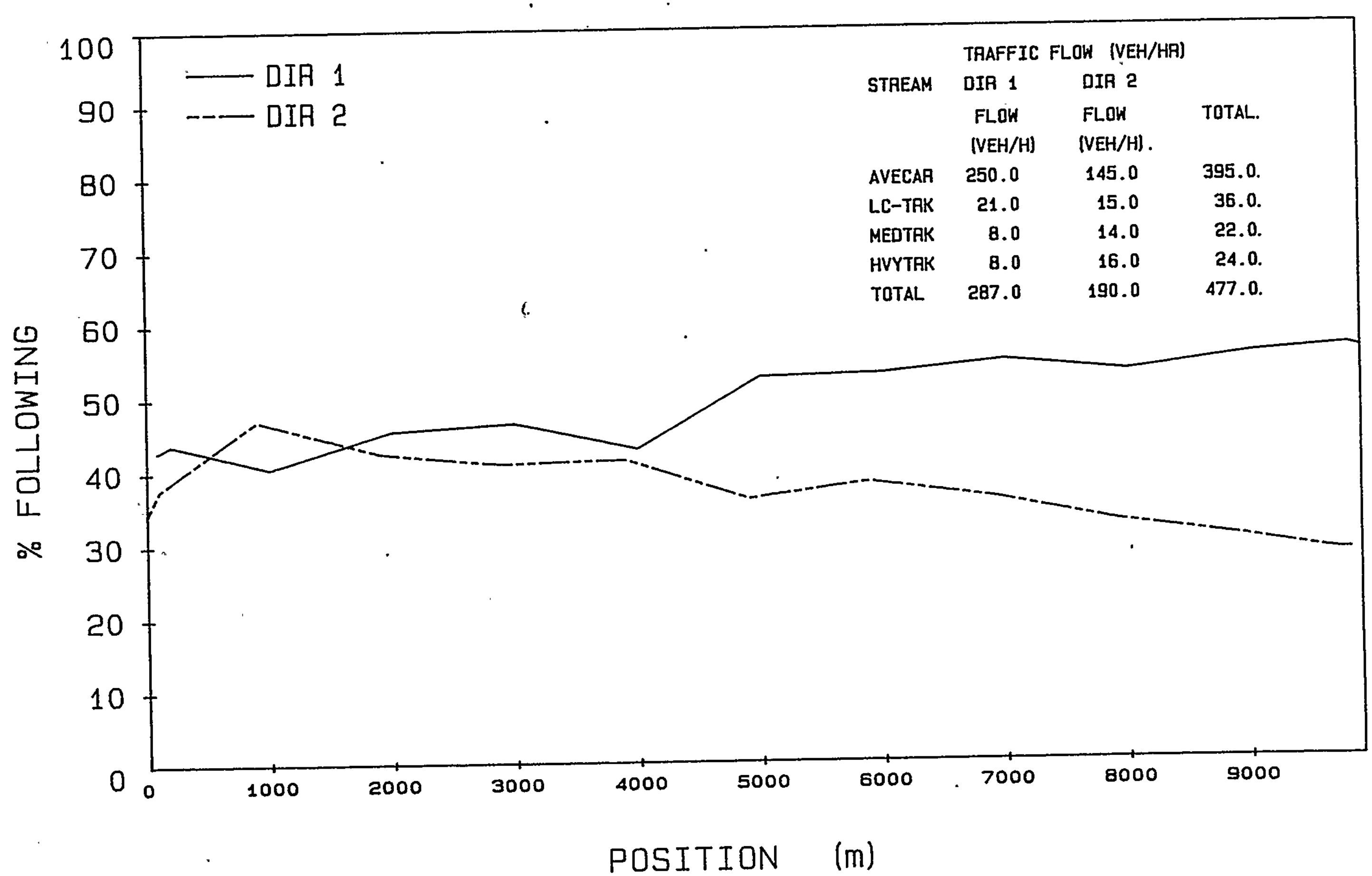


Figure 3.2 : Typical RPLOT Output

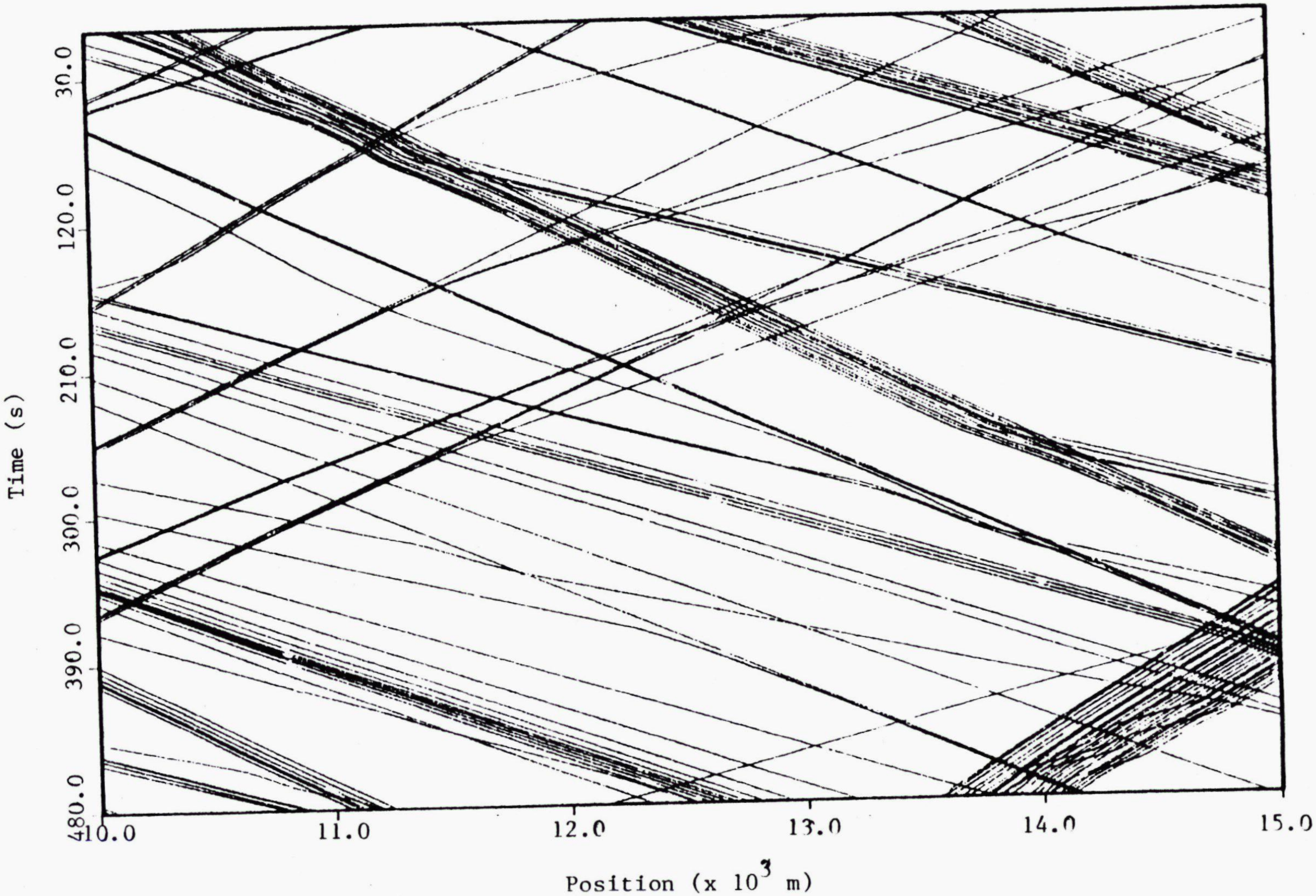


Figure 3.3 : TRARR Generated Time-Space Diagram

the most user friendly simulations available. The polished modular form of its logic makes examination and modification relatively straightforward. Once the model is calibrated to local conditions, highway agencies should be better able to apply the TRARR model to its full capabilities.

Chapter 4

Calibration and Validation

4.1 Introduction

In order for any model to be of value, the user must be confident of the results it produces or know the range of error associated with the modelling effort. The calibration of the TRARR model must be rigorous in that it will be used to generate other descriptive traffic functions for the Unified Model. The framework that will be followed in evaluating TRARR was suggested by Pilgrim (1975). He proposed a four level outline that could be used in the evaluation of mathematical simulation models. The stages of the framework are:

- examination of model structure
- estimation of model parameters
- verification or validation of model accuracy
- prediction of the range of applicability

Taylor (1979) suggests using a *dual-sampling* technique to more fully calibrate and validate a model. Two totally independent data sets are used, one in the model calibration and the second to evaluate model performance. This increases the model reliability in that there is less chance for compensating errors in the calibration process to nulify each other.

4.2 Methodology

TRARR calibration will be taken as a four step process. Each step, however, is not independently performed. Modifications made in one area may have ramifications in other portions of the model. Calibration then becomes an optimization of many variables. The complexity of TRARR and the large number of variables and estimated parameters makes a complete calibration impossible. Realistically, one should be able to examine the major components of the model and replicate actual data within specified limits. The calibration of TRARR can be thought of as a fine tuning process.

The first step in calibration was performed in Chapter 3, examination of the TRARR model structure. Both the overall modelling methodology and the individual submodels were inspected. Next, a number of descriptive driver behavioral variables were analysed for their sensitivity. The following step will be to compare the model output to the first data set and isolate areas of concern, then reestimate model parameters or modify the program logic to achieve a satisfactory fit between modelled and actual data. Finally, as a further check, the model should be able to reproduce operating conditions given a second independent data set.

4.3 Sensitivity Analysis

The microscopic modelling of the overtaking maneuver in TRARR relies on a number of safety factors used to determine a vehicle's decision to overtake. Modelling of the overtaking maneuver and overtaking rates is critical to accurate two-lane simulation. To determine what effect these safety factors have on the modelling procedures, a sensitivity analysis was conducted. The two safety factors most critical to TRARR are the VSFSN factor for sight restricted overtakings with no auxiliary lane present and the VSFVN factor for an opposing vehicle visible with no auxiliary lane present.

Determination of sight distance required for overtaking is very complicated, reliant on many variables. Troutbeck (1981) suggests that safety margins are dependent on speed of the overtaken vehicle as well as its dimensions. The VSFSN factor must be a general indicator of driver behaviour. Sensitivity test of the VSFSN on mean speed and percent following for varying available sight distances are given in Appendix B. These graphs will be of use when calibrating the TRARR model.

Overtaking rates where an opposing vehicle is visible are more critical at higher volumes. Safety margins for passing maneuvers under such conditions are also noted by Troutbeck. The VSFVN safety factor sensitivity analysis showed the selection of an appropriate value to be crucial at higher volumes. A 10 percent change in VSFVN may produce a 36 percent change in overtaking rate. Appendix B also contains the results of the sensitivity analysis for VSFVN in graphical form.

4.4 Calibration

4.4.1 Test Site

Data used in the calibration of TRARR was collected from a primary highway in Southern Alberta. Highway 2 between Nanton and Claresholm (see Figure 4.1) was instrumented with a traffic recording system as described in Appendix C.

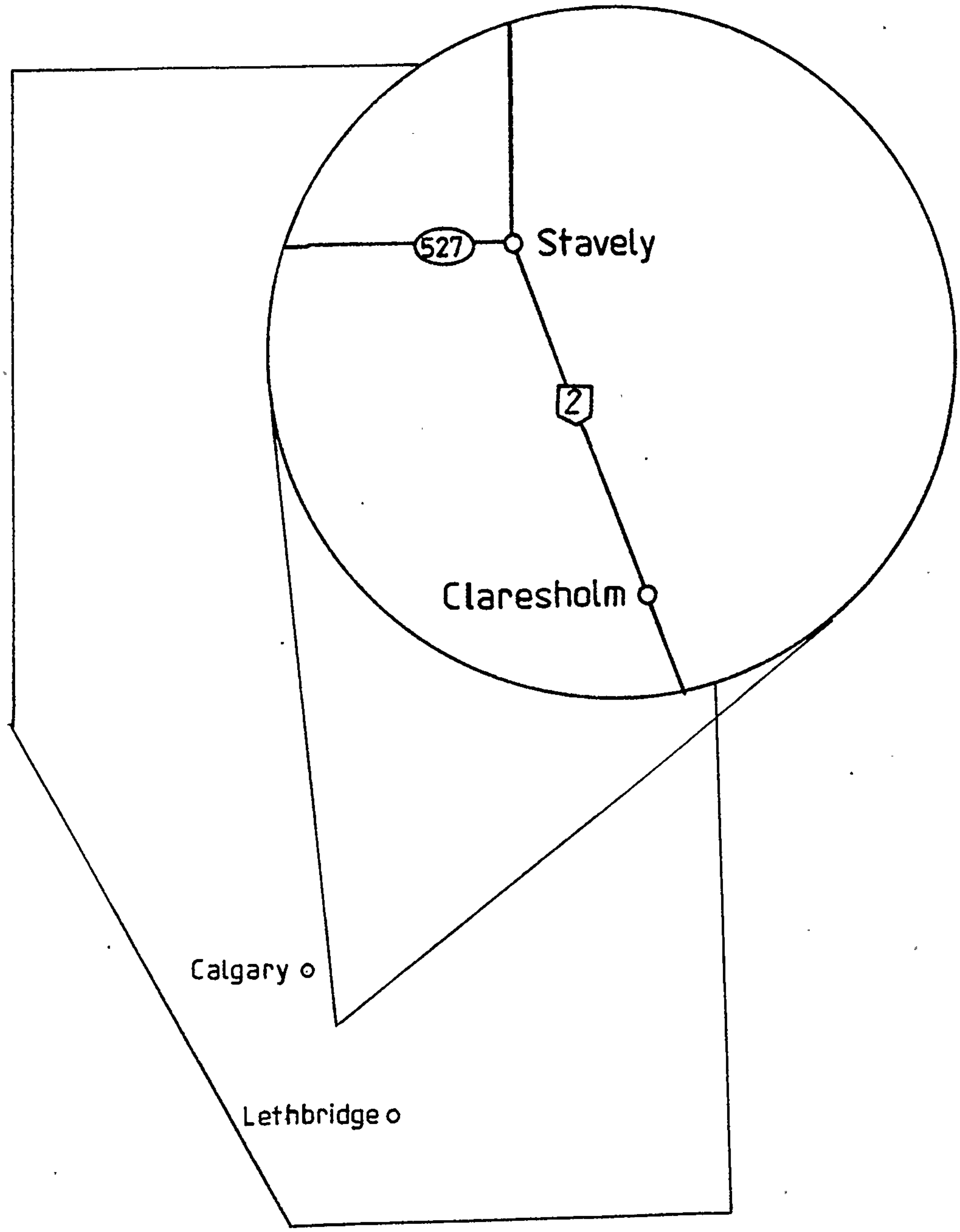


Figure 4.1 : Location of Calibration Test Site

This highway is part of the major north/south corridor through the province. It contains a high percentage of trucks travelling intercity between Calgary and Lethbridge while also providing a link connecting the surrounding communities. The roadway contains very few turnoffs with the majority of turning volumes being low. The exception to this may be at Stavely where a major intersection could disrupt the normal flow patterns of the roadway. However, turnouts and merge lanes are provided for all turn movements so the impediment caused by turning vehicles should be minimal.

The total test site length of 36 km was divided into two distinct sections (see Figure 4.2), each section providing data for a particular facet of the model. The sections can be broken down as follows:

Section 1 : Claresholm to Stavely

This is a very flat highway section providing good sight distance and passing opportunities in both directions. It is ideal for a first examination of TRARR as the varied traffic composition combined with excellent road geometry should induce a large number of passing situations. Instrumentation was provided at the ends of this test section to give both input and output statistics of traffic flow.

Section 2 : Passing Lanes at Stavely

The second portion of the test site contains a set of passing lanes constructed in a head to head fashion. Traffic counting equipment was again installed at the beginning and end of each passing lane. Calibration of this aspect of the model is important in the examination of roadway improvements using the Unified Model.

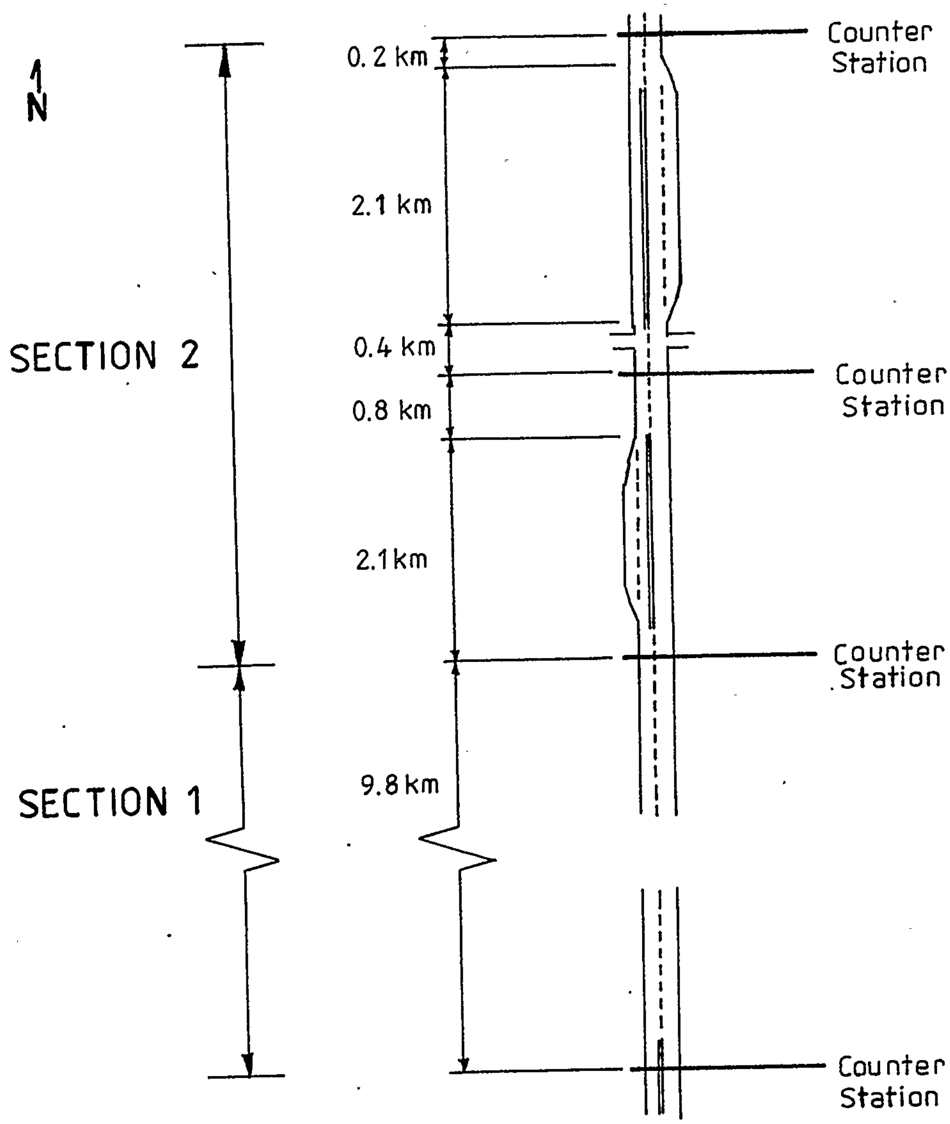


Figure 4.2 : Calibration Test Sections

Table 4.1: Vehicle Classification for TRAF File

Stream	Length	Vehicle Types
cars	$l < 500$ cm	small, medium cars
large cars	$500 < l < 750$ cm	large cars, passenger trucks
trucks	$750 < l < 1300$ cm	single, dual axle trucks
large trucks	3500 cm $< l$	semitrailer/tractor

Table 4.2: Free Speeds for TRAF Vehicle Classifications

Stream	Sample Size	Mean Speed	σ	Skewness
cars	223	106.2	7.2	0.44
large cars	100	103.8	7.0	0.12
trucks	19	95.3	8.2	0.15
large Strucks	30	96.5	6.2	0.04

4.4.2 Calibration Procedure

For modelling purposes, the eighteen vehicle types available in TRARR were divided into four traffic streams specified by length. This was neccessitated by the capabilities of the traffic counting equipment used in this study. The capabilities of the traffic counting equipment are given in Appendix C. The vehicle classification system is presented in Table 4.1. The free speed distributions of each stream were determined by a spot speed study. The results are presented in Table 4.2.

Experience with TRARR by the author has shown that correct values for desired speeds are extremely important to the output of the model. Each road has its unique speed environment due to its road geometry and driver familiarity with the highway. It is local factors that make desired speeds vary from location to location. Using estimated values for desired speeds is often inadequate for proper modelling.

Initial TRARR runs showed a substantial descrepancy between modelled and actual data. The model tended to underestimate the overall passing rate along the entire roadway thereby producing an artificially high platooning rate. This was also found in a similar study using TRARR by Morrall (1986b). The poor correlation between the points may be due to a number of reasons.

Examination of TRARR output showed the modelled speed into a test section to be significantly lower than the desired speeds input into the model. This may indicate a number of sources of model inaccuracies. First, the traffic generation technique, particularly platoon generation, may be inadequate. This may be related to program logic, random number generation or road speed index factors. Second, vehicle performance could be underestimated and thus does not allow individual vehicles to attain their desired speeds. Third, TRARR will reorder platoons and lead vehicle speeds if the input percent following value cannot be matched with internal vehicle characteristics. Each of these areas were examined more closely, the end results suggesting vehicle performance was underestimated in the VEHS file. Power and acceleration factors and vehicle characteristics were reviewed again and adjusted[1].

TRARR was then run with the modified VEHS file. Results of subsequent runs gave much better fits to observed data (See Table 4.3)[2]. However, the percentage

[1]Revised vehicle performance factors were obtained from Canadian Consumer Autosource Magazine, research reports of vehicle speeds (CalTrans) and information contained in the latest release of TRARR.

[2]Remaining tables are presented in Appendix D.

Table 4.3: TRARR Calibration (volume = 191)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	97.1	99.6	2.5	2.6
σ (Km/h)	11.6	8.7	-2.9	-25.0
No. of Platoons	38	40	2.0	5.3
% Following	35	39	4.0	11.4
No. of Gaps > 30s.	46	45	-1.0	-2.2
% of Hour with Gaps > 30s.	59	68	9.0	15.3

of vehicles in platoons were still above recorded levels for the same volume. Lack of multiple overtakings in modelling or observational errors may account for this but modelled speed distributions correlated with the calibration data, therefore the overestimation of platooning was attributed to the use of paved shoulders for overtakings.

4.4.3 Shoulder Overtakings

One of the most difficult aspects of two-lane highway modelling is to accurately represent the effects of wide paved shoulders. In Alberta, most of the major two-lane highways are built with a 3 metre paved shoulder. A primary purpose of this extra road width is to increase safety by providing a refuge for disabled vehicles. Wide paved shoulders also increase operating speeds by allowing for increased lateral vehicle separation while also contributing to a greater number of overtakings. It is common practice in Western Canada for slower vehicles to pull over onto the shoulder to allow following vehicles to pass. In effect, the shoulder is acting as a surrogate passing lane. Previous limitations to overtaking, such as sight distance, oncoming vehicles and presence of barrier lines have a reduced impact on the decision to overtake. However, the presence of a wide paved shoulder does not create uniform increase in overtaking. The complex interrelationships make modelling of shoulder overtakings strenuous with any results being highly suspect.

It is basically the larger number of variables connected with shoulder overtakings that makes microscopic examination difficult. First, there are a limited number of slow vehicles willing to move onto the shoulder. Shoulder use is also a function of structural quality and traffic volume. At higher volumes, a driver is much more reluctant to move onto the shoulder as he knows he may experience difficulty in finding a gap in which to return to the main through lane. There is also an inconsistent use of shoulders on grades. A percentage of drivers will not move onto the shoulder when travelling on a upgrade for fear of an obstruction on the shoulder beyond the available sight distance. Correspondingly, a large variation exists in the behavior of the overtaking vehicle. The decisions made are different from those of a standard overtaking. The overtaking driver does not put as much emphasis on barrier lines and oncoming vehicles but may have uncertainty as to the merging of the vehicle from the shoulder.

4.4.4 Calibration for Shoulder Overtakings

A decision was made against modifying the existing logic of TRARR to microscopically model shoulder overtakings. The modular design of the TRARR programming would make the addition of such a subset to the overtaking logic relatively easy but the data collection effort required made the exercise impractical and the highly stochastic nature of such a model would make results inconsistent. A more macroscopic approach was made. Studies by Morrall (1984) on the Trans-Canada Highway in Banff National Park indicated shoulder passings may account for approximately 25 percent of the total overtakings. This level of shoulder overtaking was recorded at a two-way volume of 1000 vehicles per hour and a directional split of 75/25 with almost all of the overtaking occurring in the heavy flow direction. This number may be high compared with overtakings on other rural roads as the survey sight in the National Parks system contains a unique vehicle composition along with driver behavioral characteristics nontypical of most rural highways. A study by Fambro et al (1981) on the operational and safety effects of driving on paved shoulders in Texas showed that only 5 percent of all vehicles used the shoulders at any one location. Modifications to TRARR should therefore be made to increase overtakings somewhere in this range.

The existence of a useable paved shoulder in effect removes some driver perceived hinderences to the overtaking maneuver. Modification of driver behavioral factors in the VEHS file can allow for the emulation of the effect of paved shoulders. In essence, the following factors were adjusted:

1. VOSFN - Overtaking Speed Factor (no auxiliary lane)
    - Increasing this factor will allow for more overtakings by decreasing the time required to complete the maneuver.
2. VHSFN - Happy Speed Factor (no auxiliary lane)
    - The VHSFN is multiplied by the desired speed to give the tolerable following speed. Decreasing VHSFN will make more vehicles switch to the *waiting to overtake* state rather than *happy to follow* state.
3. VSFSN - Safety Factor for Sight Restricted Overtakings

- Decrease of the VSFSN allows for shorter sight distances to be accepted for overtakings. This is often the case in shoulder driving. Frequently, passes are initiated with marginal sight distance, the overtaking driver knowing that the paved shoulder provides adequate room for evasive action by cars in the outer lane if required. There is also the practice of some platoon leaders moving over to the shoulder after an overtaking has commenced.

4. VSFVN - Safety Factor for Opposing Vehicle Visible

   - This factor was also decreased in an attempt to increase overtakings. The reasoning behind this is similar to that for VSFSN.

These factors were previously examined for their sensitivity. Consequently, new factors could be chosen to give an increased overtaking rate. The overtaking rates were compared to the available site distances, and the VSFSN factor was modified to allow more overtakings. At the same time, overtakings were compared to opposing volume to determine the overall effect of the VSFVN factor. Modifications were made to both factors until a satisfactory fit to observed data was obtained. Adjustments were made to the two factors by proportioning their values accordingly from the sensitivity analysis graphs to obtain matching platooning rates. The VSFVN factor showed to be the most sensitive at the test volumes.

A total of eight situations were tested for calibration. The tests represented a wide range of volumes, directional splits and traffic composition. A summary of the errors in the TRARR runs are presented in Table 4.4. More detailed data on the individual calibration runs is found in Appendix D.

4.4.5 Calibration for Passing Lanes

The calibrated TRARR model was then run for Test Section 2. This section contains a set of passing lanes, one in each direction. Unfortunately, a major turnoff

Table 4.4: Calibration Errors - No Passing Lanes

Traffic Characteristic	Δ	%
Speed (Km/h)	3.5	3.8
No. of Platoons	4.1	18.0
% Following	4.1	14.4
No. of Gaps > 30s.	2.9	7.7
% of Hour with Gaps > 30s.	4.0	6.5

Table 4.5: Calibration Errors for Passing Lanes

Traffic Characteristic	Δ	%
Speed (Km/h)	2.7	2.5
No. of Platoons	5.8	14.2
Ave. Platoon	0.2	7.6
% Following	4.9	15.7
No. of Gaps > 30s.	3.0	9.9
% of Hour with Gaps > 30s.	2.1	3.9

is located between the two passing lanes. The turning volumes were significant enough to cause compatibility problems between data collection points at the three survey sites located midway and at the ends of the test site.

Few data could be found that were continuous throughout the roadway in respect to vehicle mix. This neccessitated that each passing lane be modelled separately rather than as a pair. The results of the TRARR modelling is presented in Table 4.5 . Model results fall in an error range of 2.5 to 15.7 percent.

4.5 Summary of Calibration

As seen in Table 4.4 and Table 4.5 , TRARR is capable of reproducing observed data within an acceptable error range for both highways with and without passing lanes. The best analysis of TRARR's accuracy is determined by examining the individual calibration run results, but the summary tables do give a good overall indication of TRARR's performance. The errors produced by the model are within the limits of the erorr in the input data, the traffic classification and road geometry measurements. Traffic statistics taken on the highway show speed distributions to be slightly skewed towards the higher speeds. This effect is not accounted for in the modelling. It is noted that the roadway used in the calibration provides little variation in terms of road geometry, not fully testing the model. More calibration is also required at the upper volume ranges. Unfortunately, data at higher volumes is very difficult to collect. For the volume ranges the model was tested for, results proved to be acceptable.

Note, however, even though TRARR's modelling of passing lanes is very good, the model still requires further research in this area. The passing lanes chosen for calibration data collection allowed for only a limited test of TRARR's modelling logic. The roadway sections leading into and out of the passing lanes already contain sufficient passing opportunities to subdue platoon generation. Thus, the passing lanes' operational characteristics were not fully examined. Correspondingly, volume on the highway never approached capacity at any time during the monitoring program. Finally, the turnoff located between the passing lanes caused disruptions to the traffic flow along the main road which could not be modelled.

Unfortunately project limitations did not allow for data collection at another passing lane site.

4.6 Validation

Validation of TRARR was performed by examining the model's capability of reproducing traffic operations data collected at a second site. This should provide further insights into model inaccuracies. It is expected that the model should only require minor adjustments to replicate the second data set. In turn, the final calibrated and validated model can again be tested on the first data set.

4.6.1 Site Description

For the purpose of model validation, a test site was chosen in Banff National Park (see Figure 4.3). Highway 93 (Icefields Parkway), from the Trans-Canada Highway to the David Thompson Junction was coded for TRARR modelling purposes. Two 50 kilometre sections were chosen for analysis (Figure 4.4).

Icefields Parkway is primarily a summer recreational highway, experiencing moderately high traffic volumes during weekends. The traffic composition contains a high percentage of slow moving recreational vehicles, thereby increasing the demand for passing. The roadway has limited passing opportunity due to poor horizontal and vertical geometry. This increases both the frequency and size of platoons. From a modelling point of view, the Icefields Parkway provides an excellent validation site as the high degree of variability in both road geometrics and traffic composition should test most aspects of TRARR's program logic.

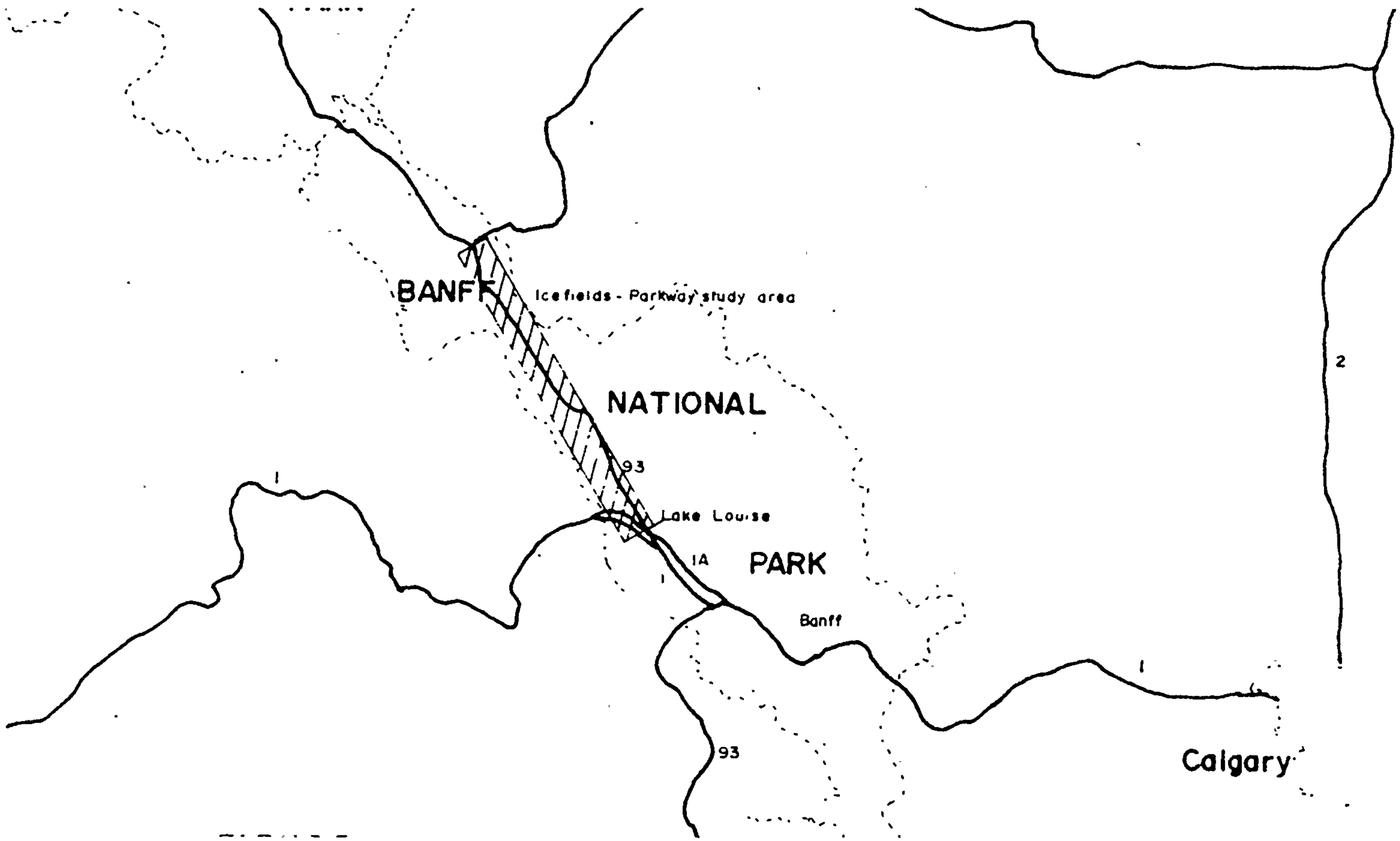


Figure 4.3: Validation Data Collection Site

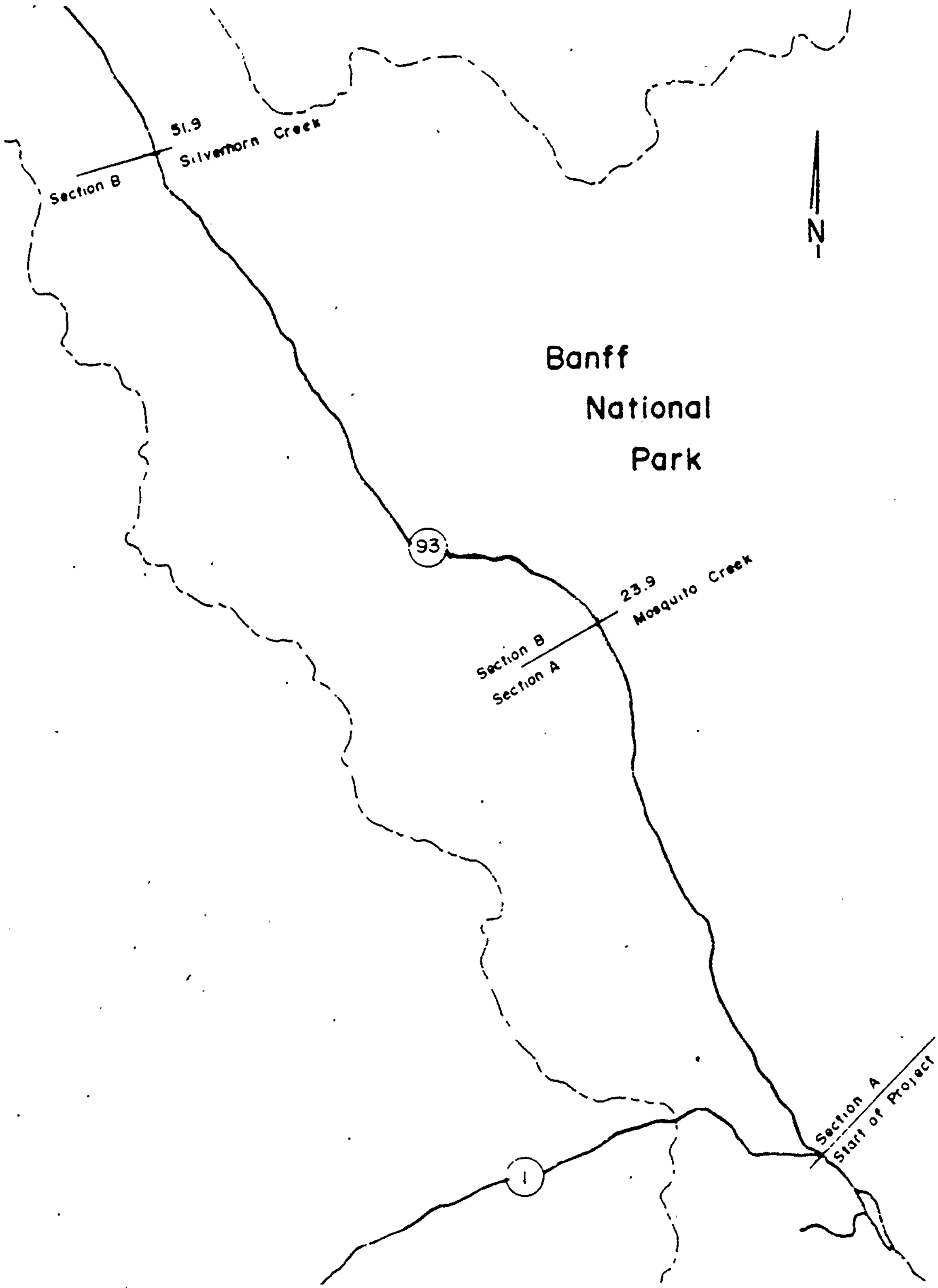


Figure 4.4: Icefields Parkway Test Sections

Table 4.6: Validation Error Range

Traffic Characteristic	Δ	%
Speed (Km/h)	4.7	5.0
% Following	7.8	21.5
No. of Overtakings	7.8	34.3

4.7 Data Analysis

For the purpose of validation, field studies were conducted at two points along the Icefields Parkway, one in each test section. Manual records were kept of vehicle speeds, types and arrival times. Overtakings were also recorded in two categories; overtakings by and overtakings of the various vehicle types. These data will provide the validation check.

Initial runs of the model showed a generally good agreement with the validation data (Table 4.6). At moderate volumes though, overtakings were being underestimated[3]. Analysis indicated this to be due to a defficiency in overtaking of recreational vehicles. Field observations showed that a large number of RV drivers on this highway travel on the shoulders allowing for others to overtake. TRARR does not allow separate safety factors when overtaking a particular class of vehicle, therefore overall adjustments were made to the VSFSN and VSFVN factors in an attempt to induce more passing. The VSFSN factor was adjusted the most as the major constraint to overtaking on the Icefields Parkway is sight distance. Volumes are generally low so changes in the VSFVN factor would have little effect

[3]Tables of individual validation runs are presented in Appendix E

Table 4.7: Calibration Error Values

Traffic Characteristic	Δ	%
Speed (Km/h)	4.4	4.7
No. of Platoons	5.3	18.5
% Following	2.6	10.2
No. of Gaps > 30s.	1.9	4.8
% of Hour with Gaps > 30s.	4.5	6.9

on overtakings. A 5 percent reduction was made to VSFVN factors for the more aggressive cars. The results of the adjusted model are presented along side the original validation runs in Appendix E.

The results show only slight improvement in some of the validation runs. In fact, some parameters show a greater error. Analysis of the results shows no apparent pattern in the error values. This may be in part due to the high variability of traffic on the roadway. The observed data contains an inherent deviation from mean values which is magnified by the nonuniformity present in the roadway geometrics and composition.

To determine which set of parameters to use, the model with adjustments made through validation, was run again for the Stavely calibration site. Table 4.7 gives a summary of results. When compared with the original calibration run, the adjusted model predicts traffic conditions with greater accuracy, up to 4 percent less error. The adjustments made to the calibrated model were therefore, adopted.

4.8 Summary

Calibration and validation of a complex model, such as TRARR, is a continuous process. The calibration effort presented here represents a first attempt of adopting this simulation for Canadian use. As more traffic data becomes available, subsequent calibrations and validations can be made. However, users of TRARR must question as to what accuracy of modelling they require. The primary idea of simulation is an optimization of resources. Refinement of models beyond a certain error range gives diminishing returns. The model should be able to reproduce traffic conditions to the same level of accuracy as the input data. Both the model results and input data should be in the error range required at the planning level.

TRARR has shown to be capable of reproducing Canadian highway operations. Adjustment of internal parameters accounted for differences between any Australian and Canadian driving characteristics. The accuracy of the model is well within the errors of the input data. TRARR can thus be used to examine Unified Model demand functions with some confidence.

It should again be stressed that this is only a first attempt at a systematic calibration for North America. Data from different highways would be desirable to further validate the model. In particular, more data on passing lanes is required. Passing lane operation is still a relatively new field of investigation. As more information becomes available, it should be used to update the TRARR model.

Chapter 5

The Unified Model

5.1 General

The Unified Traffic Flow Theory Model was proposed by Morrall and Werner (1985) as a new method for analysing level of service on rural two-lane roads. At the time, the existing Highway Capacity Manual (1965) was the primary tool used in examining traffic operations. The manual used speed as one of the defining parameters of Level of Service (L.O.S.). It has been shown that speed is relatively insensitive to volume. The 1965 HCM also ignored the effect of directional split. As previously noted in Chapter 2, the 1985 HCM made a substantial departure in its analysis method. Percent time delay is used as the major indicator of two-lane highway L.O.S. While this is a substantial improvement, the 1985 HCM does not provide procedures to access improvements made to passing opportunities. This is a major shortcoming as there are a number of low-cost improvements that can be made to a roadway which will substantially improve the L.O.S. The Unified Model provides a mechanism by which the effect of such improvements, for example passing lanes, can be quantifiably measured. The Unified Model is aptly named as it links the demand for passing with the supply of passing opportunities available for a given set of operating conditions.

Morrall and Werner base their model on the following premise:

> "It is hypothesized that drivers perceive level of service as one's ability or inability to pass slower moving vehicles and this ability to overtake

> is dependent upon the overall demand for passing by the main stream and the supply of sufficient gaps for passing in the opposing stream, provided sufficient passing sight distance is available or permitted by road geometry."

A graphical representation of the Unified Model functions is given in Figure 5.1. A clarification of the Unified Model's interactions can be better seen by examining the two model components, the supply function and the corresponding demand function.

5.2 The Supply Function

The availability of passing opportunities on two-lane highways is dependent on two major criteria, opposing traffic volume and roadway geometrics. The literature suggests that passing maneuvers may be successfully completed if a gap of 25 to 30 seconds appears in the opposing traffic stream. This gap is not a predicted time of collision but a gap in the opposing lane measured at a stationary point. Traffic volume can thus be directly related to the average number of acceptable gaps available for overtaking. In using a simulation model such as TRARR, to examine the supply of available gaps, the model's traffic generation technique must be properly calibrated and matched to give the proper distribution of vehicle headways. Inadequacies in traffic generation may lead to over or underestimation of the supply of overtaking gaps. Overestimation of initial platoon size formation would lead to overestimation of the percentage of time with adequate gaps for overtaking. Conversely, an abundance of single cars in a free state would result in an underestimation of available gaps. The overtaking mechanism of a simulation model must

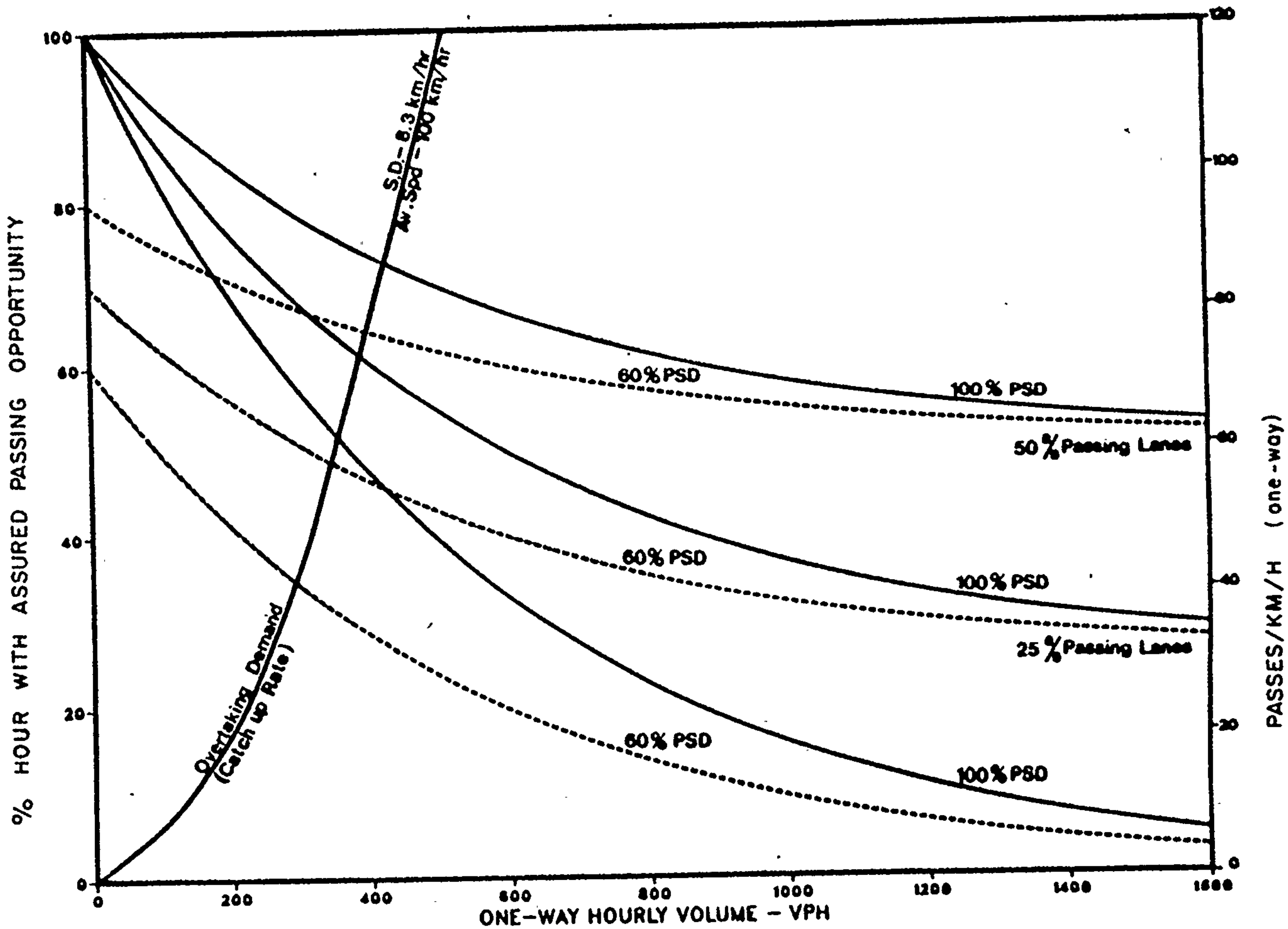


Figure 5.1 : The Unified Model - Supply/Demand Functions

Werner and Morrall (1985)

be accurate to give traffic dispersion along a highway segment comparable to actual findings.

The second constraint on passing opportunities relies on roadway geometrics. Accompanying an available gap in the opposing traffic stream, an overtaking driver usually requires suitable passing sight distance (PSD) and legality or permission to overtake. The required PSD to overtake is a complex function as described by Troutbeck (1981).

The two criteria of the supply function are combined into one parameter which is called "Assured Passing Opportunity" (APO). This accounts for both the road geometry and traffic volume.

$$APO = APSD \times GAP$$

where: APO = percent assured passing opportunity

APSD = percent of road with adequate PSD

GAP = percent of time with adequate gaps for overtaking

The selection of values for adequate PSD and overtaking gaps is debatable. The 1985 HCM suggests using a value of 450 m (1500 ft) as an appropriate value to allow most overtaking to occur.

The basic method for improving L.O.S. on a two-lane highway is to increase the supply of passing opportunities presented to motorists. This can be accomplished in a number of ways. Improving the road geometry, generally through realignment, should increase the passing sight distance, PSD being one component of assured passing opportunity. Second, more difficult and expensive, is to increase the number of opposing gaps suitable for overtaking.

Construction of passing lanes is one good method of increasing APO. Passing lanes guarantee overtakings regardless of opposing volume or available sight distance. The improvement to traffic flow from a passing lane does not only occur over the passing lane length. Beneficial effects can be measured well downstream of a passing lane. The topic of passing lane effectiveness is more fully discussed in Chapter 6.

5.3 The Demand Function

One measure of effectiveness of a roadway is its ability to satisfy the existing demand for overtaking. If this demand is not met, vehicles are forced to follow slower vehicles at less than desired speeds resulting in platoon formation. A theoretical relationship for the overtaking demand was developed by Wardrop (1952).

$$P = \frac{1}{\sqrt{\pi}} \frac{\sigma Q^2}{\overline{U}^2}$$

where: P = demand for passing (passes/km/h)

Q = one directional traffic flow (vph)

$\overline{U}$ = mean desired speed (km/h)

σ = standard deviation of desired speeds (km/h)

This model calculates the passing demand required to maintain a free flow. If this demand cannot be met, platooning occurs. The difference between actual overtakings and overtaking demand gives the unsatisfied passing demand (UPD). This UPD is related to the platooning level on the roadway.

The actual overtaking rate is a complex function of the two opposing streams. The only practical way of determining these rates is through simulation. It affords the flexibility of examining many scenarios such as varying traffic volumes, sight distance and addition of passing lanes. Alternatively, one could perform a massive data collection program over a large number of highways to obtain similar data but such a scheme would be impractical in terms of time and cost.

5.4 Generation of Demand Function by Simulation

Actual overtaking rates for varying traffic conditions were determined through simulation with the TRARR model. A total of 275 runs were performed to quantify overtaking and platooning rates for different volumes, directional splits and roadway geometrics. One-way volumes ranged from 200 to 1000 vehicles per hour with a directional split starting at 50/50 and ending at 90/10. Passing opportunities based on sight distance were varied between 100% and 20% or conversely, no passing zones ranged from 0% to 80% of the roadway. A test road section of length 10 kilometres was used. Only one traffic stream was input, that of cars, no heavy trucks were allowed. Only level terrain was modelled. Finally, road conditions containing 0%, 10% and 20% passing lanes were analysed.

No passing zones were inserted in 500 metre blocks on a symetrical criteria. These 500 metre no-passing zones were placed on the road in such a manner to evenly decrease sight distance along the road. Double barrier lines were first placed in the center of the section and then at the first and third quarter points. At the higher percentages of no-passing, 1000 metre blocks were used. Determination of the proper distribution of sight distance is a major problem in this stage of model development. Heimbach et al (1973) and Stock and May (1976) describe methods of determining sight distance distributions along a rural road. At this point in the Unified Model development it was decided to distribute the no passing zones in a simple fashion.

The results of the simulation study are presented in Figures 5.2 to 5.6. The percent time spent following is plotted against the traffic volume for individual road geometries[1]. Note that the family of curves generated for varying sight distance exhibit only moderate change. The absence of slow moving vehicles, particularly heavy trucks, could explain much of the lack of sensitivity. Sensistivity becomes very low at higher volumes. However, the results correspond to Level of Service criteria given in Table 8.1 of the 1985 HCM for a 50/50 directional split and 100 percent cars (Figure 5.7). Changes in v/c ratio due to percent no-passing zones match with changes in percent time delay predicted by TRARR. The effect of passing sight distance is better seen by examining the overtaking rates and mean speeds. Modelled mean speeds are sensitive to volume to the same degree as given in Figure 8.1 of the HCM. There does not appear to be much affect of directional split on speed. Overtaking rates for each scenario tell more about the operating conditions on a highway. Figures 5.8 to 5.12 illustrate the results[2]. Directional split plays a major role in available gaps for overtaking. The graphs show points at which overtaking rates begin to fall with increased volume for given directional split and passing sight distance. This is an indication of an increased drop in level

[1] The x-axis of these graphs represents one-way traffic volume. Example - For a one-way traffic volume of 600 vph and a directional split of 80/20, the total two-way volume would be 750 vph.

[2] One-way volumes are plotted in a similar fashion to previous figures in this chapter.

of service being encountered.

5.4.1 Inclusion of Passing Lanes

The passing opportunities along the 10 km test section were improved by the addition of 10 percent and 20 percent lengths of the highway being converted to passing lanes. The highway with 10 percent passing lanes was composed of one passing lane of one kilometre length placed at the center of the test section. The 20 percent passing lane configuration had one kilometre passing lanes placed at the third points. Passing was not allowed for the opposing traffic flow.

The same range of volumes and directional splits were simulated for these roadways. The results are presented in Figures 5.13 to 5.15.

5.5 The Unified Model and Level of Service Concept

A key element in developing the Unified Model is to construct relationships between it and the level of service concepts presented in the 1985 HCM. Given the inputs required for the Unified Model, passing demand and assured passing opportunities, one should be able to determine a corresponding L.O.S. Specifically, highway improvement options should be related to improved operating conditions and L.O.S.

Passing demand is inherent in a given volume and directional split. Actual overtakings were simulated with TRARR, with unsatisfied passing demand being reflected in the percent time spent following. The supply side of the Unified Model, or availability of passing opportunities is determined by specifying the available passing sight distance and opposing volume. Opposing volume is given indirectly by directional split.

Given the volume and directional split, one can calculate the level of service using the Unified Model by entering the proper graph (Figures 5.2 to 5.6). The estimated percent time spent following is given directly. This value can then be directly translated into a L.O.S. using criteria specified by Chapter 8 of the HCM. The effect of passing lane addition can be examined using this technique.

5.6 Summary

Using the TRARR model, actual overtaking rates for varying traffic volumes, directional split, passing sight distance and passing lane configurations were generated. The quantitative effects of each combination can be seen in the figures presented.

Rates of improvement and detriment can be noted for each combination of traffic parameters.

TRARR output allowed for relationships to be developed between the Unified model and the 1985 HCM's level of service concept. The accuracy of these relationships is discussed in the following chapter.

Figure 5.2: Unified Model Simulation Results
50/50 Directional Split
0% Passing Lanes

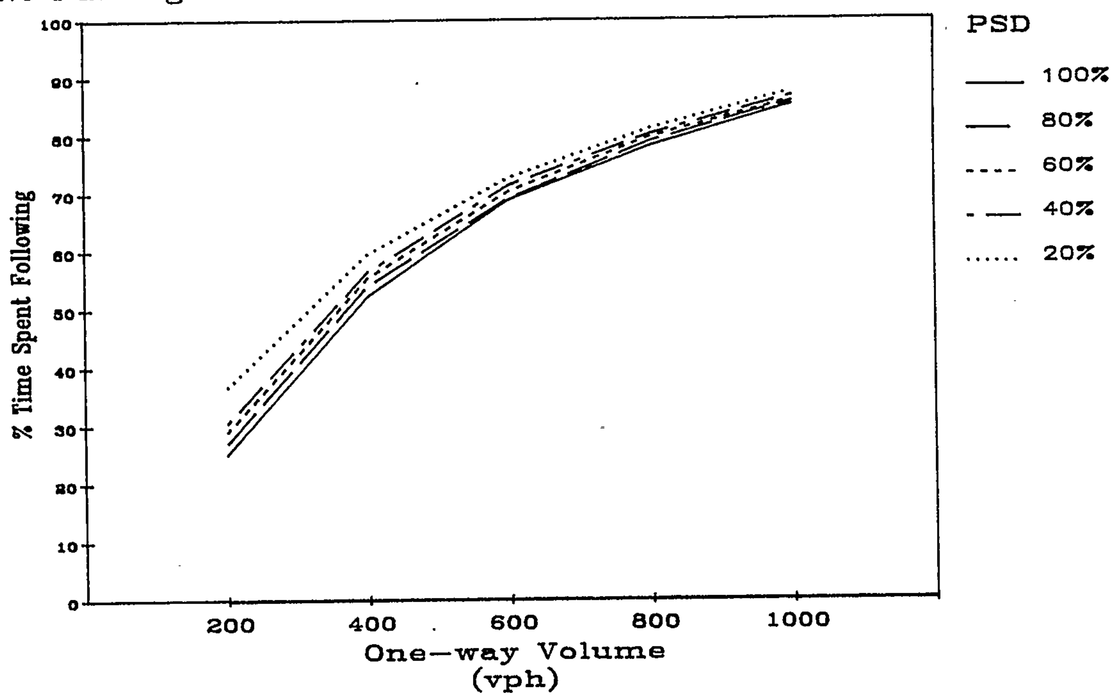


Figure 5.3: Unified Model — Simulation Results
60/40 Directional Split
0% Passing Lanes

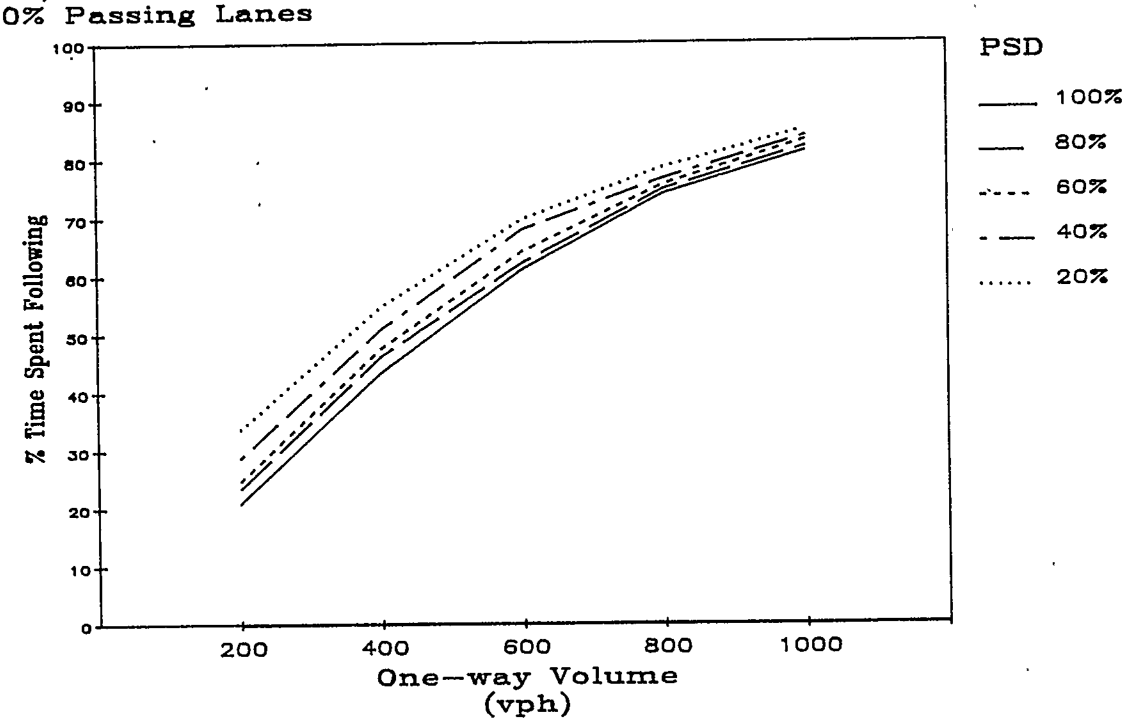


Figure 5.4: Unified Model — Simulation Results
70/30 Directional Split
0% Passing Lanes

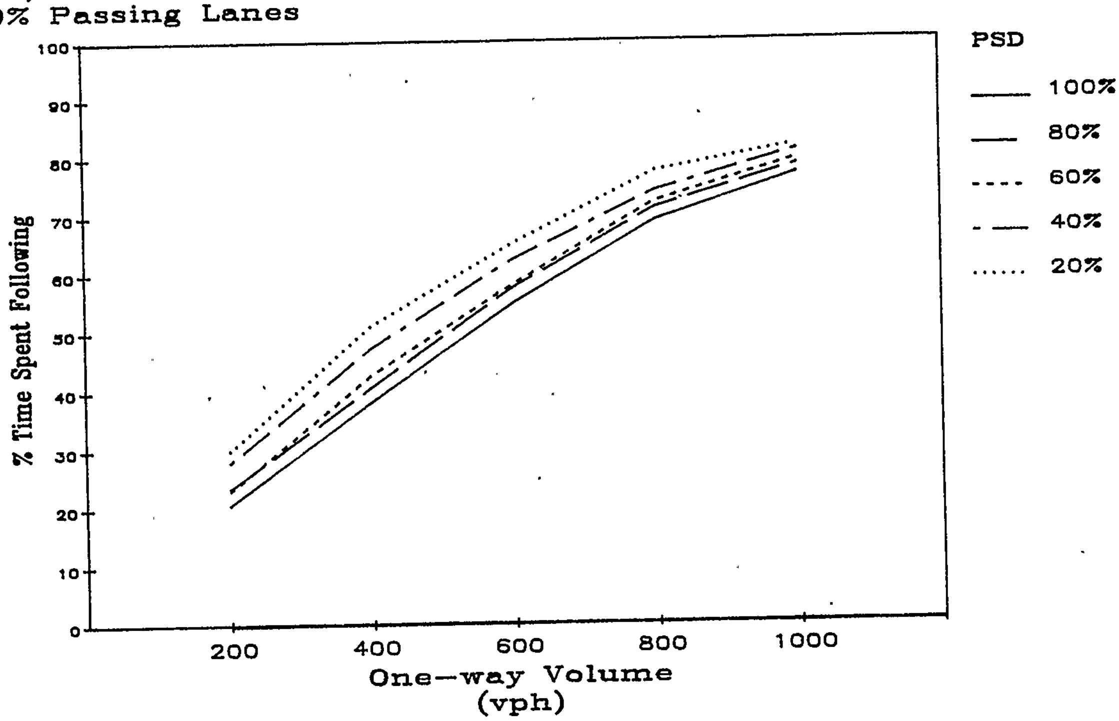


Figure 5.5: Unified Model — Simulation Results
80/20 Directional Split
0% Passing Lanes

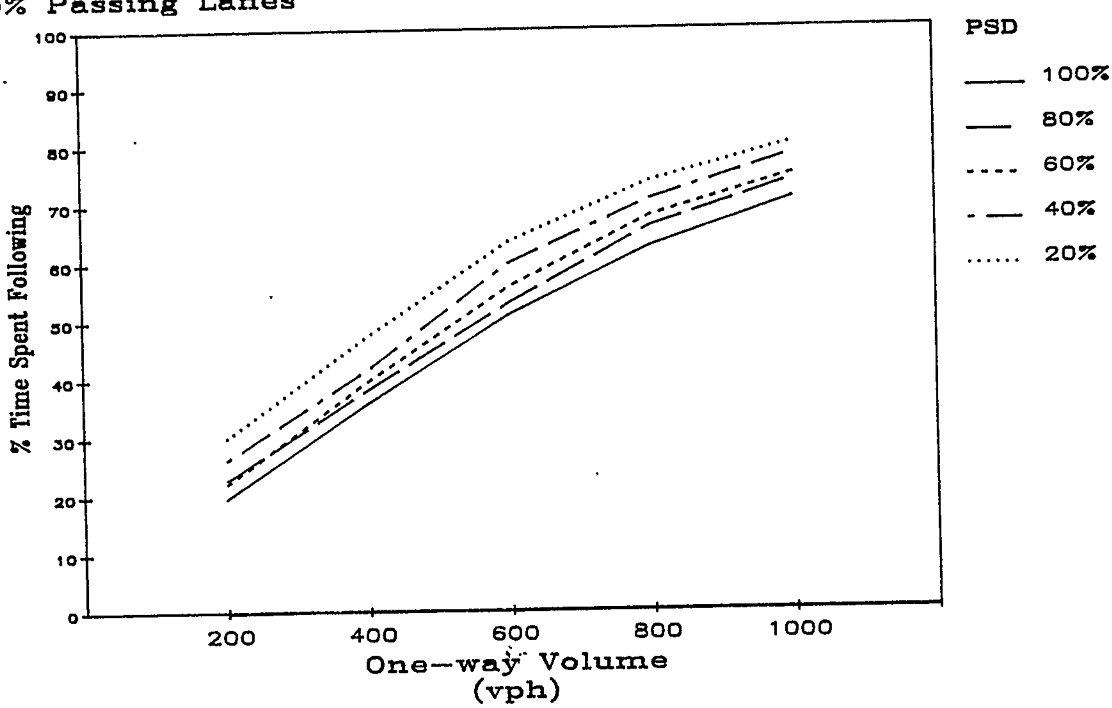


Figure 5.6: Unified Model — Simulation Results
90/10 Directional Split
0% Passing Lanes

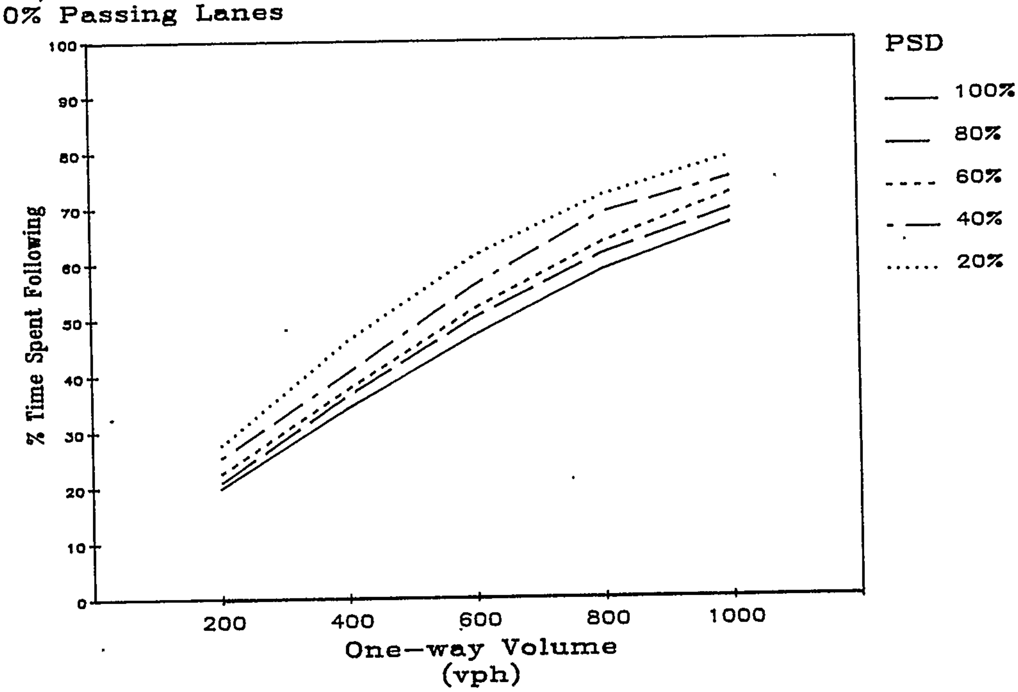


Figure 5.7: 1985 HCM / TRARR Comparison

% Time Spent Following

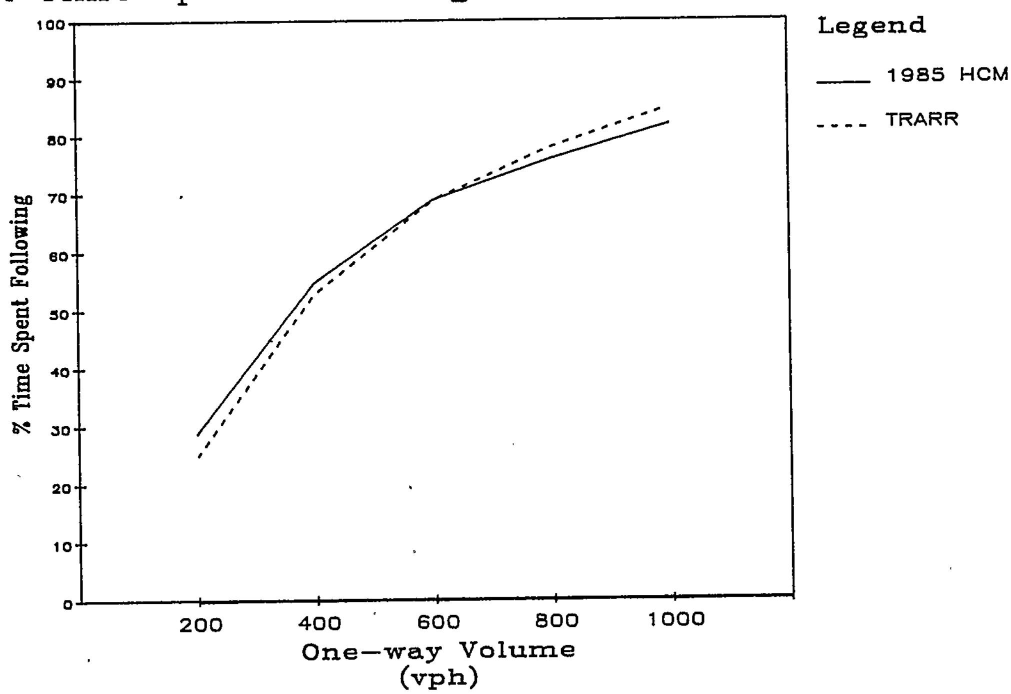


Figure 5.8: Overtaking Rates – TRARR Simulation
50/50 Directional Split
0% Passing Lanes

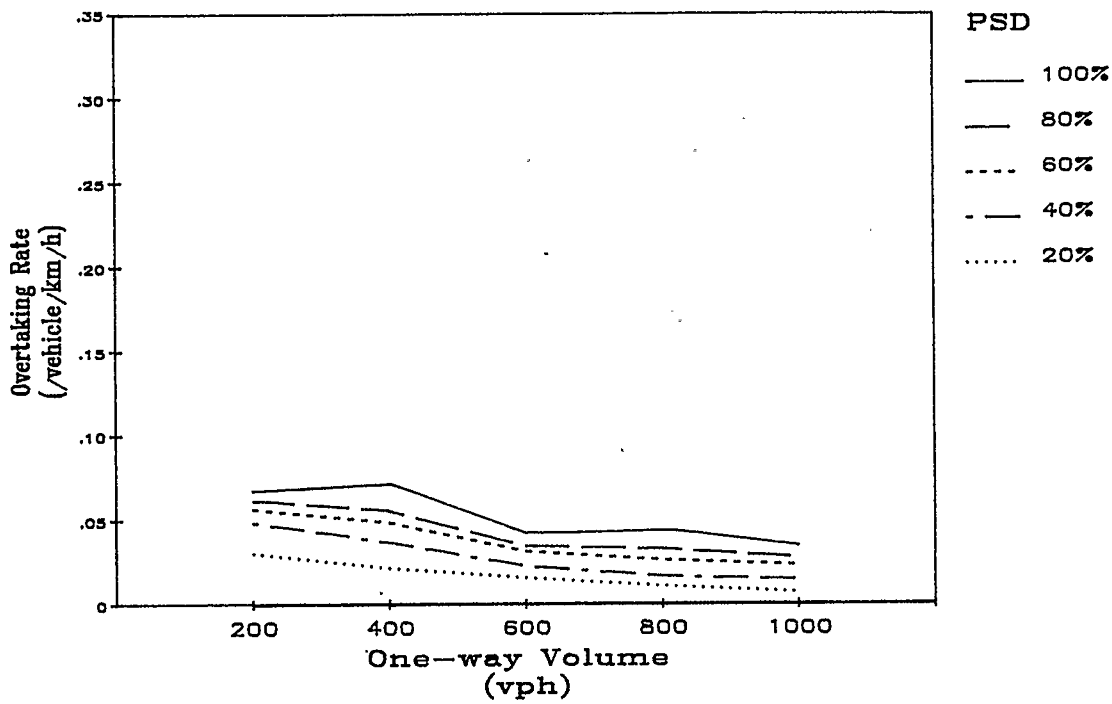


Figure 5.9: Overtaking Rates – TRARR Simulation
60/40 Directional Split
0% Passing Lanes

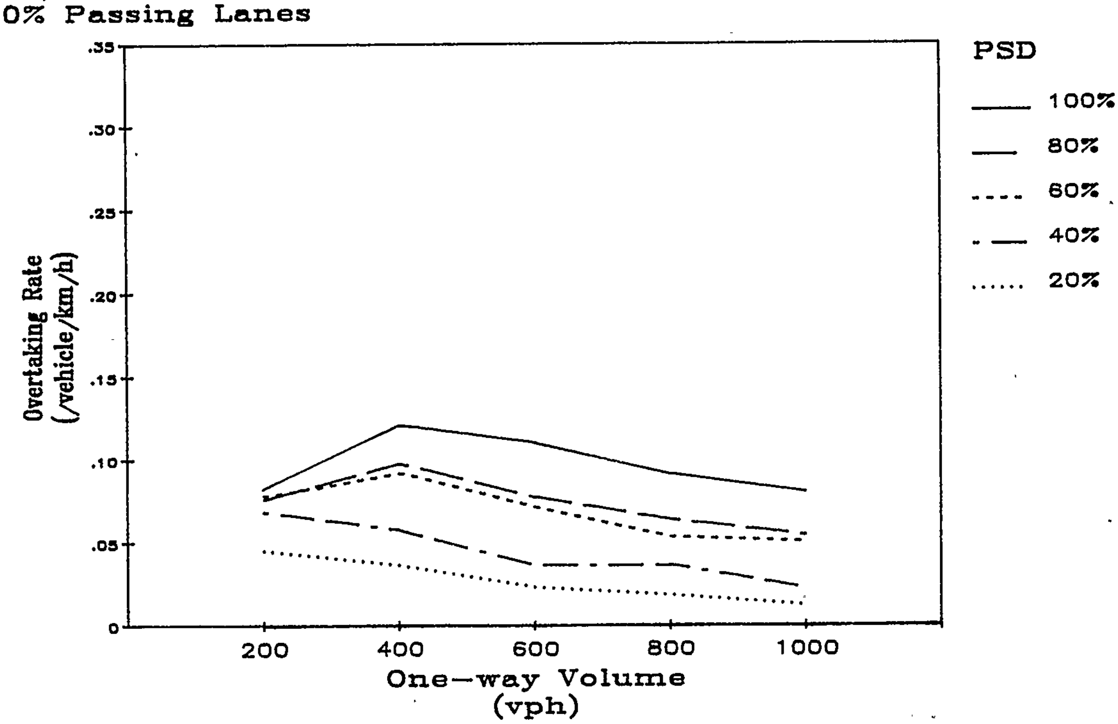


Figure 5.10: Overtaking Rates — TRARR Simulation
70/30 Directional Split
0% Passing Lanes

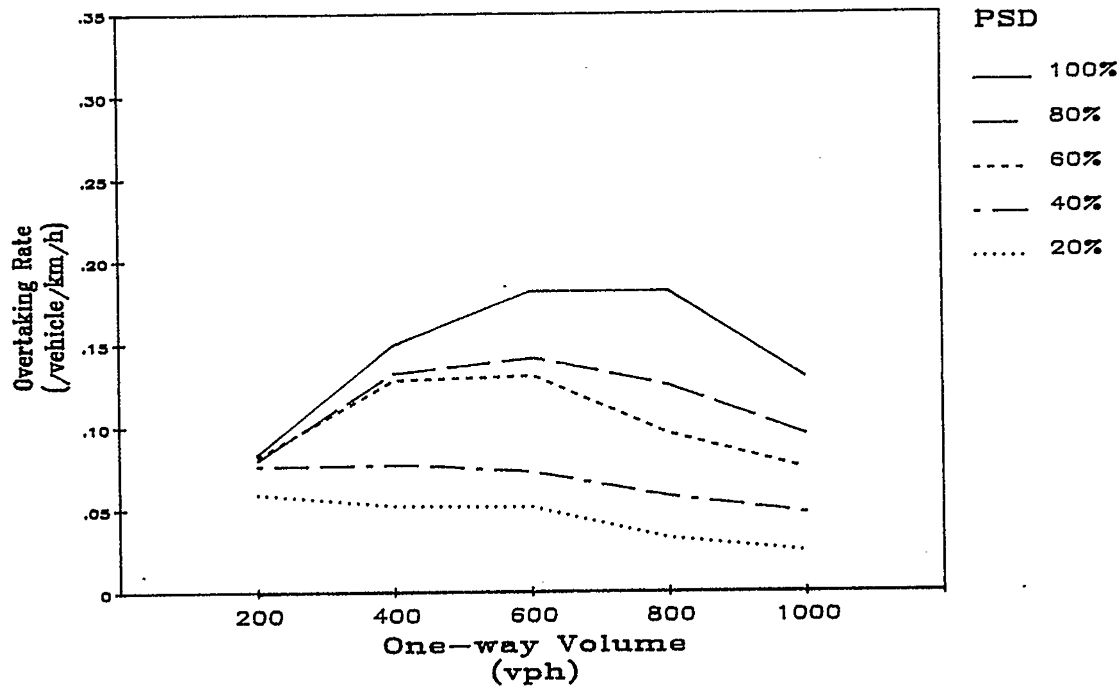


Figure 5.11: Overtaking Rates — TRARR Simulation
80/20 Directional Split
0% Passing Lanes

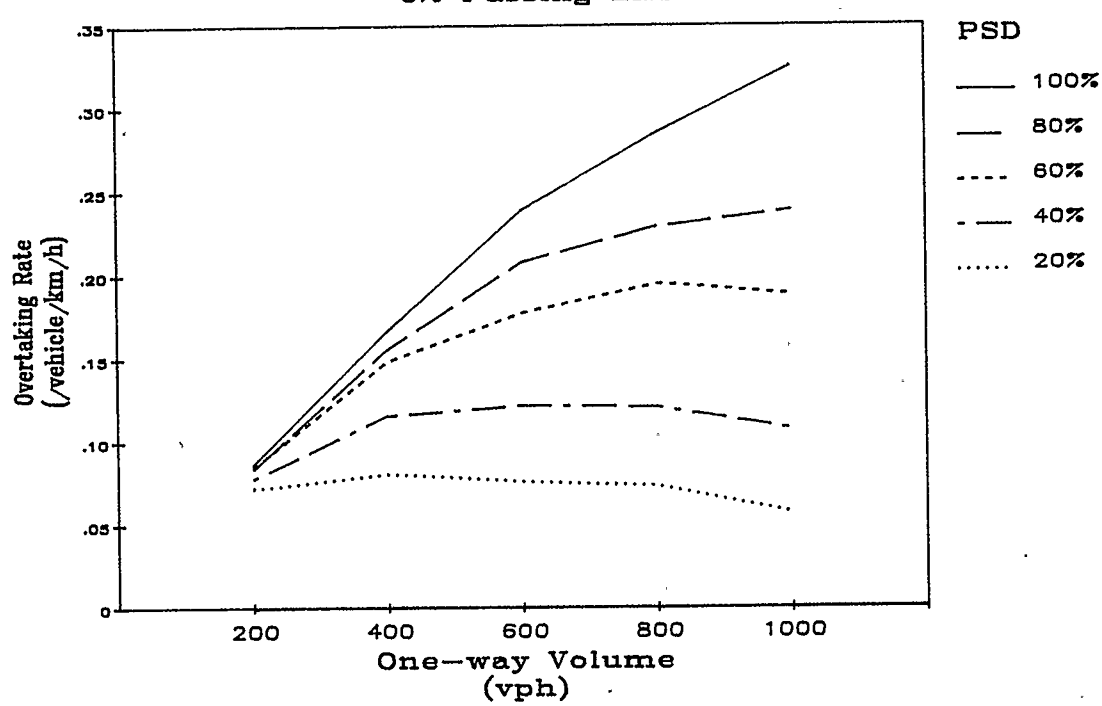


Figure 5.12: Overtaking Rates — TRARR Simulation
90/10 Directional Split
0% Passing Lanes

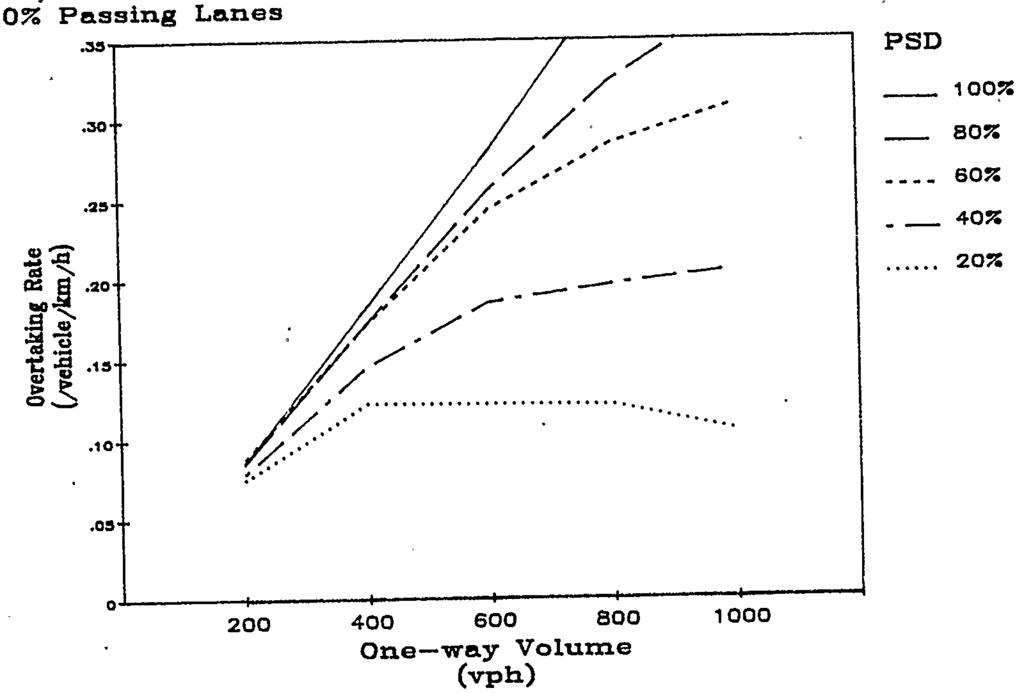


Figure 5.13: Effect of Passing Lanes on % Following
50/50 Directional Split
100% Passing Sight Distance

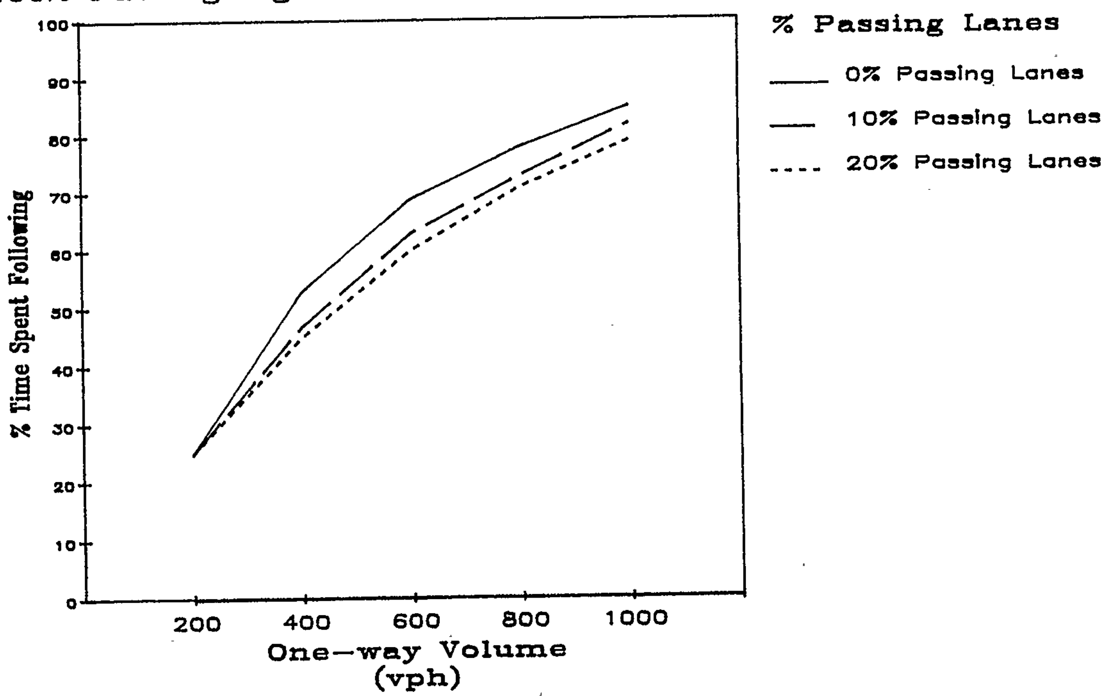


Figure 5.14: Effect of Passing Lanes on % Following
50/50 Directional Split
60% Passing Sight Distance

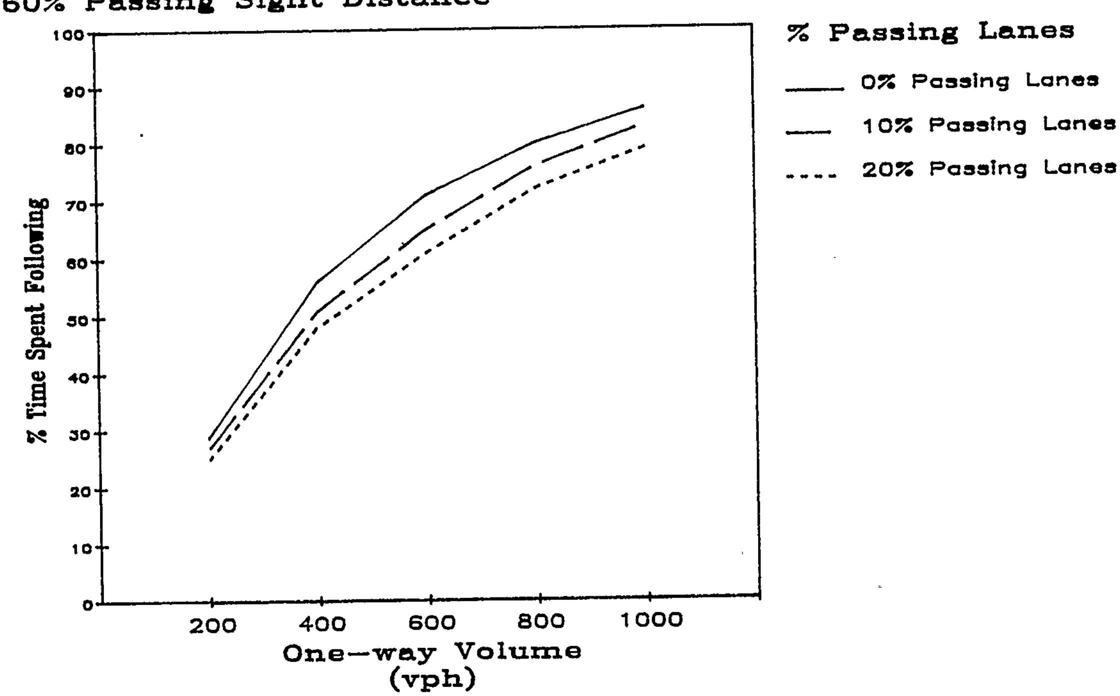


Figure 5.15: Effect of Passing Lanes on % Following
50/50 Directional Split
20% Passing Sight Distance

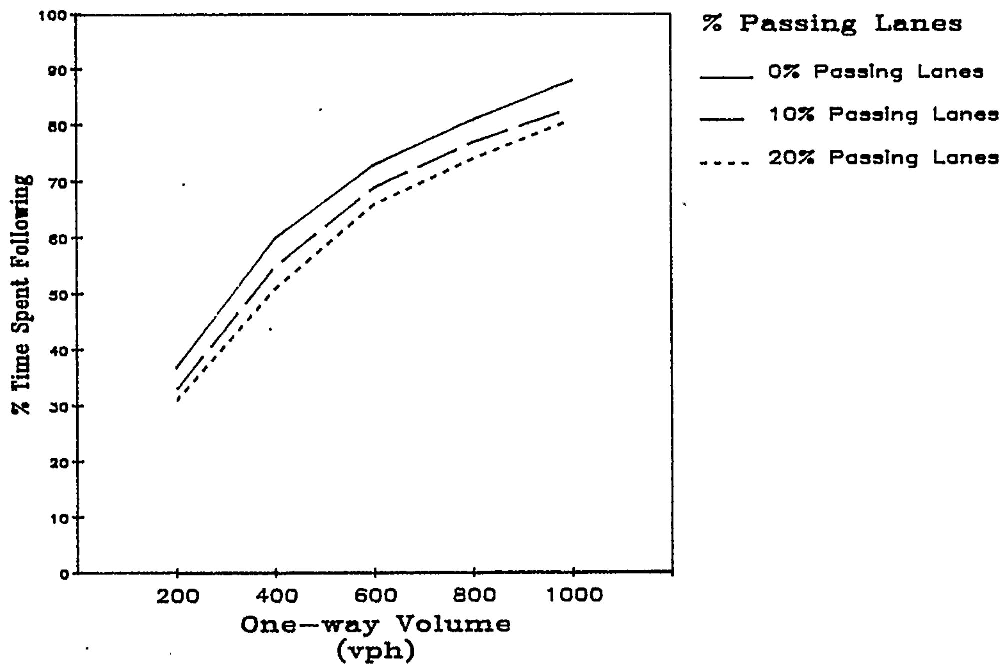


Figure 5.16: Effect of Passing Lanes on Overtaking
50/50 Directional Split
100% Passing Sight Distance

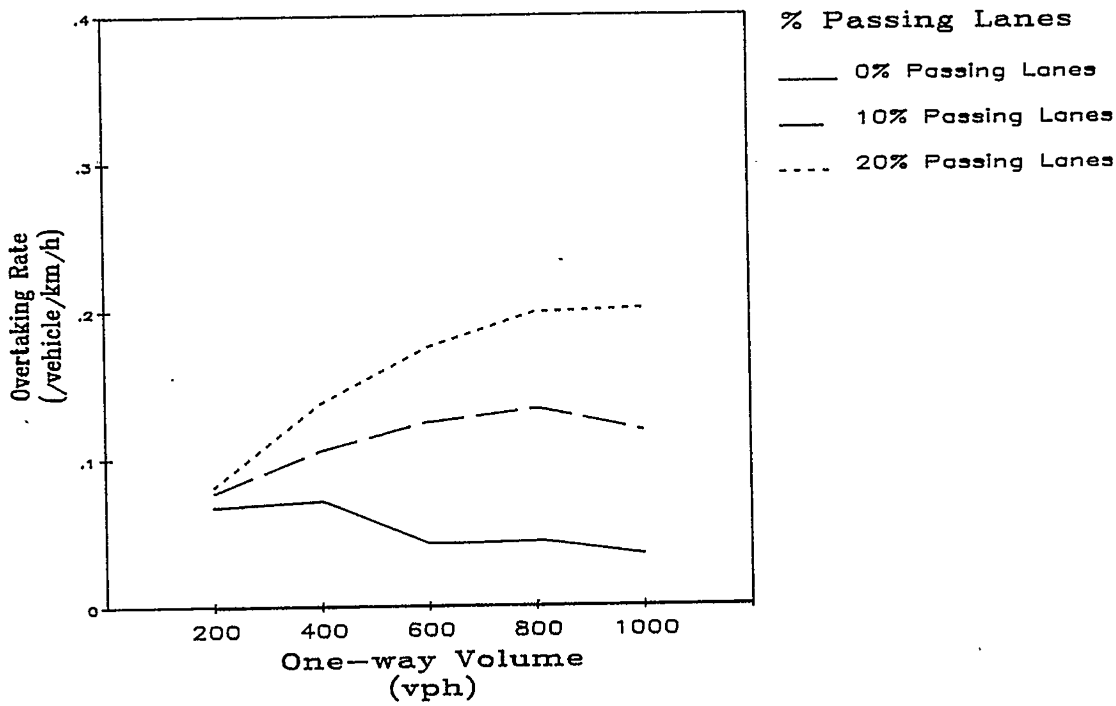


Figure 5.17: Effect of Passing Lanes on Overtaking
50/50 Directional Split
60% Passing Sight Distance

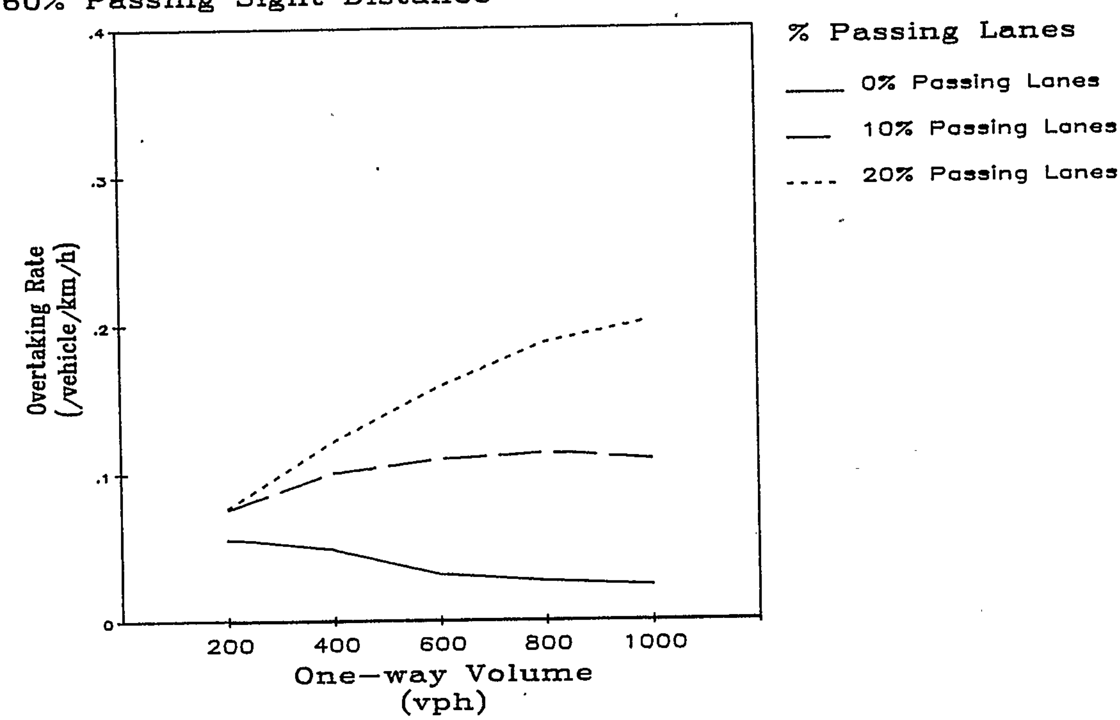
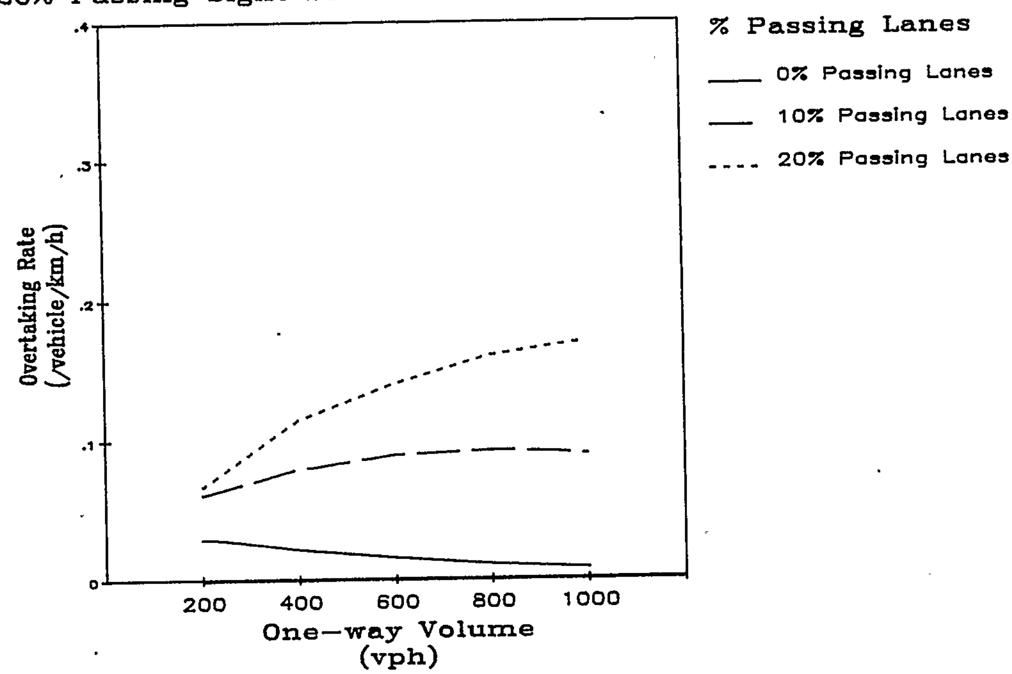


Figure 5.18: Effect of Passing Lanes on Overtaking
50/50 Directional Split
20% Passing Sight Distance



Chapter 6

Discussion of Unified Model Development

6.1 General

This chapter presents a more detailed analysis and review of the functions developed for the Unified Model. The method of data generation and corresponding limitation used in the modelling process are set forth. Implications of this method in design are given.

6.2 Unified Model Development

Interactions between traffic in the two opposing lanes on a two-lane highway are so complex that it is not practical to examine all possible scenarios. The wide range of operating conditions found on rural roads necessitates that simplifications or only subsets of the total picture be examined. It then becomes a question of how limiting or reflective the subset is of basic operations. This point is mentioned in regard to the functions generated relating the Unified Model to the 1985 HCM.

6.2.1 No Passing Lanes

Relationships for normal highway geometrics were presented in Figures 5.2 to 5.6. A number of observations can be made. Comparisons between the ideal case of traffic flow used in the HCM and that reproduced by TRARR show a good correspondence. Platooning rates increased at the same rate in both cases. This is a further validation for TRARR. One should note the percent time spent following criteria becomes relatively insensitive to passing sight distance, particularly at higher volumes and even directional splits. Lack of overtaking in these cases can be attributed to inadequate supply of gaps in the opposing traffic stream and not limitations imposed by road geometry. As directional split increases, more gaps are available for overtaking. This is reflected in the figures, with the family of curves diverging from each other as directional split increase.

The insensitivity of percent time spent following may also be due to the calibration factors of TRARR. TRARR was calibrated to account for shoulder overtakings, thus there will be more passing modelled for a given percentage of available sight distance. This would tend to decrease the platoon buildup on roads with significant portions of no-passing lanes.

Overtaking rates exhibit much more dependence on PSD and directional split as shown in Figures 5.8 to 5.12. A critical volume can be seen where overtaking rates begin to decline with increasing volume for a given set of conditions. This critical volume represents a major change in the deterioration of L.O.S. The unsatisfied passing demand increases at much faster rates. Increased overtaking rates are coupled with only moderate gains in mean speed, supporting the notion of speeds being insensitive to volume. The higher overtaking rates at higher volumes may indicate a greater proportion of the traffic on the roads in the form of high speed platoons. Road service is high yet percentage of vehicles following also remains high. Further research into the Unified Model could develop correction factors that relate high speed platoons to an increased level of service. First, a precise definition of a high speed platoon is required. Such a platoon may be defined as having the lead vehicle travelling at or above the posted speed limit.

6.2.2 Passing Lanes

Unified Model functions for analysis of passing lanes were developed and presented in Figures 5.13 to 5.15. In examining the case for 100 percent PSD, it can be seen that the improvement due to the insertion of 10 percent passing lanes is much greater than the change due to the addition of 20 percent passing lanes. This can be partially explained by the concept of effective passing lane length. Generally, passing lanes are constructed in lengths of one or two kilometres. The majority of overtakings occur during the first 500 metres. However, the benefits from the dispersion of platoons at the head of a passing lane is felt for distances of up to 10 kilometres downstream. This downstream distance is the area where operational conditions are improved over the no passing lane situation. In ideal situations, such as the case in the modelling effort, the effective length may be very long. The lack of grades and trucks in the traffic stream makes the reformation of platoons very slow after dispersion at the passing lane. Consequently, by using a test section of only 10 kilometres the complete effects of the single and to a much greater extent, double passing lane configurations may not have been recorded. An underestimation of passing lane improvements may have been inherent in design.

Addition of grades and trucks into the traffic stream would decrease the level of service afforded by the no passing lanes situation and conversely increase the benefits incurred through the use of passing lanes. Grades and trucks in the traffic stream increase the level of platooning. Passing lanes would then be more effective at dispersing these platoons and decreasing percent time spent following.

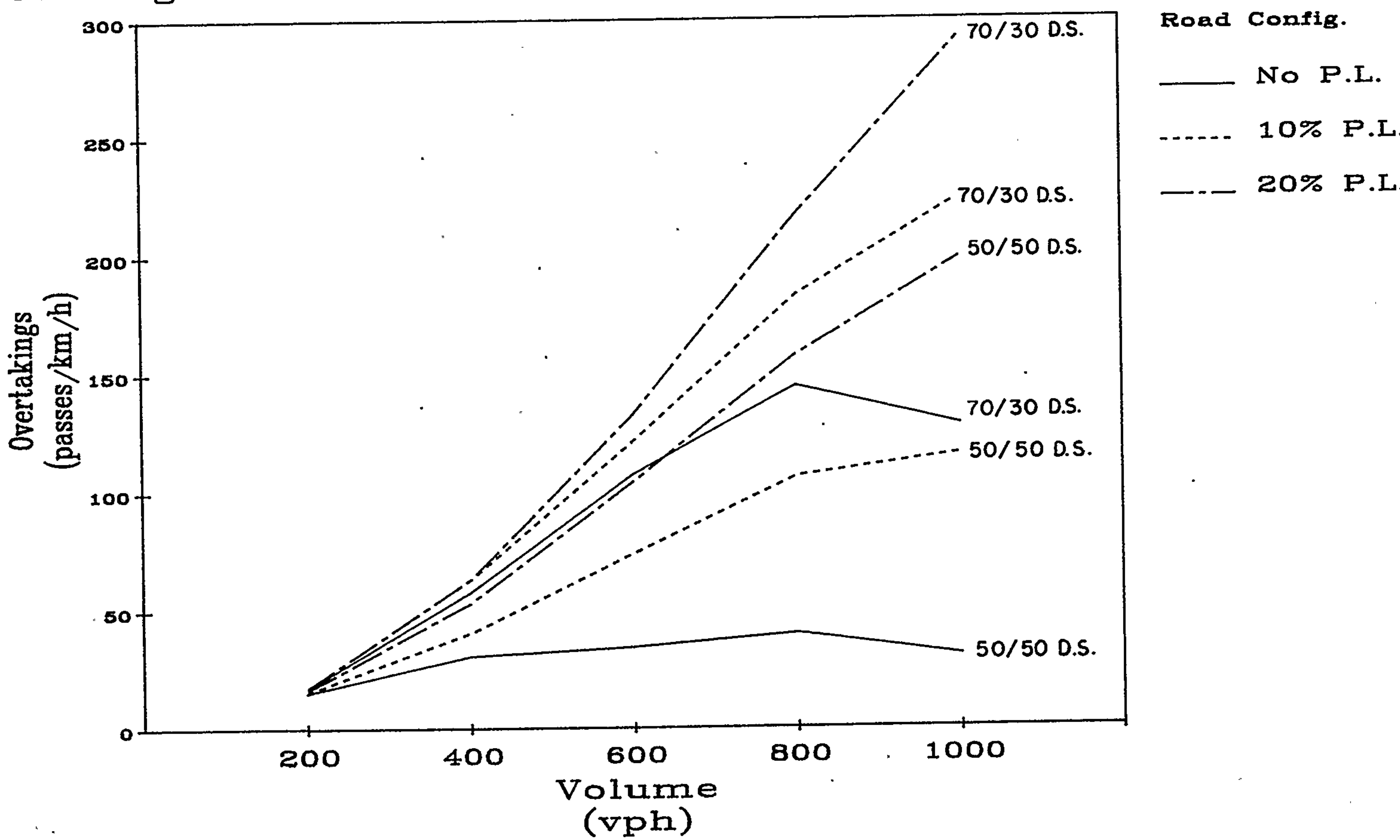
The overtakings modelled by TRARR are presented in Figure 6.1. The great effect on overtakings that both directional split and passing lanes have is clearly illustrated. Even though the modelling procedure showed passing lanes do not offer substantial decreases in the percent time spent following criteria, Figure 6.1 shows a great increase in vehicle mobility. These two areas appear to be contradictory but they may be explained by high speed platoons. Passing lanes may allow for vehicles to remain at a relatively high speed by allowing groups of vehicles to pass a slower vehicle.

Counterbalancing the improvements made through passing lane use, there are also some negative effects referring to operations. Double stripped barrier lines are often used through a passing lane length. Passing lanes can then take away passing opportunities for the opposing direction. This is especially true if the passing lane is located on a section with good PSD in the opposing direction. The passing opportunity due to road geometry is lost, thereby increasing the platooning level. Similarily, passing opportunities in the opposing lanes can also be lost due to the dispersion of vehicles in the primary lane. Passing lanes break up platoons, thus decreasing the number of gaps in the traffic stream for opposing vehicles. The percent following criteria may actually increase in the opposing direction. Actual quantification of this effect requires more study.

6.3 Range of Applicability

Development of the Unified Model is still at primary stages. Relationships have been developed to the HCM's L.O.S. system. For the range of volumes modelled,

Figure 6.1: TRARR Overtaking
100% Sight Distance.



200 to 1000 vph, the Unified Model functions can be said to give an accurate reflection of two-lane operating conditions.

Application of functions developed for two-lane highways with passing lanes show the general trends of improvement that comes with their installation. The idealized traffic conditions examined were not conducive to showing improvement from passing lanes, thus the functions actually underestimate the effectiveness of their inclusion.

This can be noted for all cases simulated. The idealized situation does not totally reflect the operating conditions on typical rural roads. The lack of trucks and grades decreases the percent time spent following for all cases.

Chapter 7

Conclusions

7.1 General

The objectives of this study were to calibrate the TRARR simulation model and then further develop the Unified Traffic Flow Theory. Calibration of a simulation model as complicated as TRARR is an on-going process. The TRARR calibration effort in this study can serve as the starting point for further testing. The Unified Traffic Flow Theory was developed to a point where the effect of changes in roadway geometry can be reflected in level of service. Further development of the Unified Theory should expand its relationships to cover more cases of traffic flow.

7.2 Calibration and Validation of TRARR

Examination of current work on two-lane rural roads has shown TRARR to be an excellent choice for this project. It has the flexiblity and detailed level of analysis suitable for examination of traffic operations. The calibration procedure used attempted to test most facets of interest in the model. Results of the testing have shown the model to represent two-lane rural roads in Alberta very well. The major areas of study, speed, percent time spent following, overtakings and gaps in the traffic stream were modelled with little error. Differences between the observed and modelled values were well below the errors of the input data and observed traffic flow.

Modelling of passing lanes was also accomplished with good results. The effects of platoon dispersion could again be closely matched between observed and modelled data. However, the test site did not allow for a complete examination of TRARR's modelling capabilities in this area. A major intersection located between the set of passing lanes made data inconsistent from one collection site to another. This greatly limited the number of data sets available for calibration. This also caused a major problem in that at the higher, more critical volumes, it became difficult to find hourly volumes which were consistent throughout the test section. For the range of volumes tested, model results fell within acceptable limits.

Validation of TRARR revealed some weaknesses in the model which were attributed to inadequate compensation for shoulder overtakings. Adjustments were made to the model with satisfactory results. Model validation increased the confidence in TRARR's use. The ability of TRARR to recreate conditions on the validation test site demonstrated its robust modelling. Ideally, though, a higher upper limit of volumes would have been preferred for testing.

7.3 The Unified Model

Using the calibrated TRARR model, demand functions were generated for varying road geometry, volumes, directional splits and passing lane configurations. An idealized traffic stream of 100 percent cars was used and a simplistic approach was also used in distributing sight distance. No grades were modelled.

Relationships were developed between the Unified Model and HCM's level of service based on the percent time spent following. Comparison of the HCM's idealized case of all cars, 100 percent sight distance and no grades compared favourably to that generated by TRARR.

The functions developed show all the expected trends between the various parameters, but using simplified approaches to traffic stream and sight distance distribution tended to underestimate the sensitivity of the Unified Model. A pure car traffic stream is not as susceptible to platoon buildup as a mixed traffic stream. It can therefore be stated that the Unified Model functions developed are accurate but have limited applicability due to the simplifications made in their development.

The mobility of the traffic stream was measured by overtaking rates. Findings showed that over the volumes tested, overtaking rates were much more sensitive to sight distance and directional split than the percent following criteria. Mobility measured through overtakings indicated a greater level of service than predicted using the percent time spent following. This may be due to vehicles following in high speed platoons. The perceived level of service of these vehicles may be higher than originally predicted as their speeds are very high and they have freedom to maneuver, yet they are happy to follow.

7.4 Further Research

As with most projects of a developmental nature, this work has brought forward many areas of further research. TRARR has been calibrated and validated to allow more simulation of rural highway operating conditions.

Continuation of Unified Model development should consider the following areas:

1. Development of curves for traffic streams containing a varying percent of trucks

   - The effect of heavy trucks in the traffic flow have been discussed. An appropriate extension of this work would be to further examine the effect of various percentages of trucks on level of service. The benefits of passing lanes could then be more fully analysed.

2. Account for not only percentage of sight distance but the distribution of this sight distance along the roadway

   - Distribution of sight distance plays an important part in overall availability of passing opportunities. This area should be further examined so that correction factors could be applied to account for this effect.

3. Model the effect of grades

   - The Unified Model functions could easily be developed to account for varying grades. This would require more data on truck performance. The TRARR model could be modified to account for decreases in truck speeds on downgrades. This again would require more data for calibration. The effect of grades could take two forms, either a parallel approach to the 1985 HCM, using level, rolling or mountainous terrain, or specific grades could be analysed.

4. Revalidate the model for passing lanes

   - TRARR was not fully calibrated for passing lanes due to a limited data collection effort. Ideally, a passing lane should be fully instrumented to collect not just input and output data, but record the overtaking rates as a function of downstream passing lane length. More data and calibration is definitely required in this area.

5. Check the TRARR model at higher volumes

    - Data at the upper volume ranges is very difficult to collect but is required to determine the top range of TRARR's modelling ability. Interest would be particularly in the effect of adding a passing lane scheme as an intermediate solution to four-laning a highway. A number of highways may be suitable for instrumentation to collect the upper volumes. The Trans-Canada Highway and urban commuter roads would experience these higher volumes.

6. Incorporate the effect of turn delays into the Unified Model

    - To make the Unified Model a more complete capacity method, the delays caused by traffic turning at intersections should be incorporated. This could be accomplished by modification of the TRARR model or use of a separate turn delay simulation.

7. Adjustments to L.O.S. for high speed platooning.

    - The insensitivity of percent time spent following has been attributed to the formation of high speed platoons. Even though these vehicles are in platoon, the level of service perceived by these drivers would be higher than that given by the percent following criteria. More research is required to evaluate this effect.

The main requirement of the majority of these research areas lies in data collection. As the newly available traffic statistics recorders become more wide spread, the Unified model should be supplied with a more adequate data base.

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Appendix A

TRARR Input Files

Examples of the four TRARR input files follow.

A.1 Example VEHS File

The VEHS file contains the calibrated safety factor values.

```
(13(/),2(9I7/),8(//2(9F7.2/))//2(4P9F7.2/)) FILE VEHS
VEHICLE AND DRIVER CHARACTERISTICS ARE SPECIFIED FOR 18 VEHICLE TYPES:
ROW 1:                                   ROW 2:
1  EXTRAORDINARY VEHICLE                 10  SMALL TRUCK
2  LARGE ROAD TRAIN                      11  CAR AND CARAVAN
3  SMALL ROAD TRAIN                      12  UNAGRESSIVE CAR
4  LOW POWERED 38 TONNER                 13  LOW POWERED CAR
5  HIGH POWERED 38 TONNER                14  AVERAGE CAR
6  UNLOADED ARTIC. TRUCK                 15  LARGE CAR
7  BUS                                   16  AVERAGE CAR
8  4 TONNE TRUCK (LOADED)                17  AVERAGE CAR
9  1 TONNE TRUCK (LOADED)                18  SPORTS CAR

LIF: DESIRED SPEED GROUP
      1      2      2      2      2      2      2      2      2
      3      3      2      3      3      3      3      3      4

VOSFN: OVERTAKING SPEED FACTOR WHEN THERE IS NO AUXILIARY LANE
   1.00   1.06   1.06   1.06   1.10   1.10   1.15   1.20   1.20
   1.10   1.05   1.05   1.05   1.15   1.10   1.15   1.15   1.15

VOSFA: OVERTAKING SPEED FACTOR WHERE THERE IS AN AUXILIARY LANE
   1.00   1.03   1.03   1.03   1.04   1.04   1.18   1.10   1.10
   1.07   1.05   1.05   1.07   1.07   1.05   1.07   1.06   1.03

VHSFN: HAPPY SPEED FACTOR WHERE THERE IS NO AUXILIARY LANE
   0.96   0.96   0.96   0.96   0.96   0.96   0.96   0.96   0.96
   0.96   0.94   0.93   0.96   0.96   0.98   0.97   0.98   1.05
```

```
VHSFA: HAPPY SPEED FACTOR WHERE THERE IS AN AUXILIARY LANE
   1.00   1.00   1.00   1.00   1.00   1.00   1.00   1.00   1.00
   1.00    .98    .98    .99    .99    .98    .98   0.98    .99

VDE: MAXIMUM DECELERATION (AS AN ACCELERATION)
  -3.00  -3.00  -3.00  -3.00  -3.00  -6.00  -6.00  -3.00  -4.00
  -6.00  -4.00  -6.00  -6.00  -6.00  -7.00  -6.00  -6.00  -8.00

VXA: MAXIMUM ACCELERATION
  10.00  10.00  10.00  10.00  10.00   8.40   7.60   8.00   7.60
   9.00   6.70   8.40   9.80   7.00   7.00   8.00   8.00   9.00

VNP: MAXIMUM POWER TO BE USED WHILE NOT OVERTAKING
   5.00   3.00   4.00   5.00   7.00  12.00  10.00  18.00  20.00
  55.00  20.00  45.00  57.00  60.00  65.00  63.00  60.00  85.00

VXP: MAX POWER TO BE USED WHILE OVERTAKING
   5.00   3.00   4.00   5.00   8.00  19.00  12.00  27.00  29.00
  65.00  25.00  47.00  67.00  75.00  87.00  75.00  75.00 116.00

VWRC: RESISTANCE COEFF. OF SPEED**2 DUE TO WIND ETC. (10**-4 M**-1)
   -.40   -.40   -.60   -.70   -.70  -1.50  -1.00  -1.00  -1.50
  -2.00  -3.50  -1.50  -3.50  -2.50  -2.00  -2.50  -2.50  -1.80

(15(//2(9F7.2/)))

VLN: LENGTH OF VEHICLE
  60.00  50.00  35.00  16.00  16.00  16.00  12.00  12.00  12.00
   7.00  12.00   5.00   4.00   5.00   5.50   5.00   5.00   4.40

VFA: ACCELERATION TO BE USED FOR SMOOTH FOLLOWING BEHAVIOUR
    .50    .50    .50    .50    .50    .50    .50    .50    .50
    .50    .50    .50    .50    .50    .50    .50    .50    .50

VFB: (REL. SPEED)/(DIST. TO BE MADE UP) FOR SMALL FOLL. DEVIATIONS
    .10    .10    .10    .10    .10    .15    .15    .10    .10
    .15    .10    .10    .15    .15    .15    .15    .15    .15

VFDA1: DISTANCE SPACING FOR BASIC LANE WHEN NOT THINKING OF OVERTAKING
   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00
   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00

VFDB1 TIME SPACING
   2.00   1.50   1.50   1.00   1.00   1.00   1.00   1.00    .70
    .90   3.00   3.00   2.00   2.00   2.00   2.00   2.00   2.00

VFDA2: DISTANCE SPACING WHEN IN BASIC LANE, CONSIDERING OVERTAKING
   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00
```

```
   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00   2.00

VFDB2 TIME SPACING
   2.00   1.50   1.50   1.00   1.00   1.00   1.00   1.00    .70
    .90    .70   1.00    .40    .50    .50    .50    .50    .30

VFDA3: DISTANCE SPACING FOR OVERTAKING LANE WHERE THERE IS NO AUX. LANE
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00

VFDB3: TIME SPACING IN SAME SITUATION
   3.00   2.00   2.00    .70    .70    .70    .90    .70    .70
    .70   1.00   1.30    .70    .70    .70    .70    .70    .60

VFDA4: DIST. SPACING WHEN IN O'TAKING LANE WHERE THERE IS AN AUX. LANE
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00

VFDB4: TIME SPACING
   4.50   2.50   2.50   1.00   1.00   1.00   1.30   1.00   1.00
   1.00   2.00   2.00   1.00   1.00   1.00   1.00   1.00   1.00

VSHA: FIXED DISTANCE COMPONENT OF MINIMUM DESIRED FOLLOWING DISTANCE
    .80    .80    .80    .80    .80    .80    .80    .80    .80
    .80    .80    .80    .80    .80    .80    .80    .80    .80

VSHB: TIME COMPONENT OF FOLLOWING SPACE WHEN HASSLED
   1.00    .50    .50    .20    .20    .20    .40    .20    .20
    .20    .50    .50    .20    .20    .20    .20    .20    .20

VFDF: FOLLOWING DISTANCE FACTOR (PER SECOND)
    .00    .00    .00    .00    .00    .00    .00    .00    .00
    .00   .005    .01    .01    .01    .01    .01    .01   .015

VFDFC: FOLLOWING DISTANCE FACTOR CUTOFF VALUE (MAXIMUM FACTOR)
   1.00   1.00   1.00   1.00   1.00   1.00   1.00   1.00   1.00
   1.00   1.60   2.00   3.00   3.00   3.00   3.00   3.00   5.00

(3(//2(9I7/)),4(//2(9L7/)),11(//2(9F7.2/)))

LAG: AGGRESSION NUMBER  (0 = ONLY OVERTAKES WHERE THERE IS AN AUX. LANE)
      3      3      3      3      3      3      3      3      3
      3      3      3      3      3      6      3      3      8

LAGB: WAIT FOR VEHICLE BEHIND IF ITS AGGRESSION NUMBER EXCEEDS THIS
      7      7      7      7      7      7      7      7      7
      7      5      4      7      7      7      7      7     10
```

```
LAGF: WAIT FOR VEHICLE IN FRONT IF ITS AGGRESSION NUMBER EXCEEDS THIS
      4     4     4     4     4     4     4     4     4
      4     4     4     4     4     9     4     4     9

LBS: WHETHER OVERTAKE WHEN BEYOND SECOND IN A PLATOON
      F     T     T     T     T     T     T     T     T
      T     T     F     T     T     T     T     T     T

LLA: WHETHER OBEYS OPTIONAL OVERTAKING RESTRICTIONS (T = YES, F = NO)
      T     T     T     T     T     T     T     T     T
      F     T     T     T     F     F     F     F     F

LRO: WHETHER DO RISKY OVERTAKINGS
      T     T     T     T     T     T     T     T     T
      T     T     F     T     T     T     T     T     T

LUO: WHETHER USE OPPOSING AUXILIARY LANE TO ADVANTAGE
      F     T     T     T     T     T     T     T     F
      T     T     F     T     T     T     F     T     T

VTO: CONSIDER YOURSELF BEING O'TAKEN IF VEH WILL REACH YOUR REAR IN VTO
   5.00  5.00  5.00  5.00  5.00  5.00  5.00  5.00  5.00
   5.00  5.00  5.00  5.00  5.00  5.00  5.00  5.00  5.00

VSFSA: SAFETY FACTOR: SIGHT-RESTRICTED OVERTAKING, AUXILIARY LANE STARTS
   2.00  2.00  1.80  1.60  1.60  1.60  2.00  1.60  1.60
   1.60  1.80  2.50  1.50  1.70  1.70  2.00  1.40  1.40

VSFSN: SAFETY FACTOR FOR OTHER SIGHT-RESTRICTED OVERTAKINGS
   2.00  2.00  1.90  1.70  1.70  1.70  2.00  1.70  1.70
   1.60  1.90  1.90  1.80  1.70  1.60  1.70  1.60  1.50

VSFVA: SAFETY FACTOR: OPPOSING VEHICLE VISIBLE, AUXILIARY LANE STARTS
   2.20  2.20  2.00  1.80  1.80  1.80  2.20  1.80  1.80
   1.80  2.00  2.70  1.70  1.90  1.90  2.20  1.60  1.60

VSFVN: SAFETY FACTOR: OPPOSING VEHICLE VISIBLE, NO AUXILIARY LANE
   2.10  2.10  2.10  2.10  2.00  2.00  2.00  2.00  2.00
   1.90  2.10  2.10  2.10  1.90  1.80  1.90  1.80  1.70

VSOA: DIST. COMPONENT OF SPACE TO BE LEFT AFTER O'TAKING (INCL. LENGTH)
  70.00 60.00 35.00 20.00 20.00 20.00 15.00 10.00 10.00
  10.00 20.00 13.00 10.00 10.00 10.00 10.00 10.00 10.00

VSOB: TIME COMPONENT OF SPACE TO BE LEFT AFTER OVERTAKING
    .20   .20   .20   .20   .20   .20   .20   .20   .20
    .20   .20   .20   .30   .30   .30   .30   .30   .30
```

```
VEXA: LEAST SPACING SUCH THAT NO EXTRA OVERTAKING TIME IS ALLOWED
 100.00  80.00  65.00  45.00  45.00  45.00  40.00  30.00  30.00
  30.00  40.00  35.00  25.00  25.00  25.00  25.00  25.00  15.00

VEXB: EXTRA OVERTAKING TIME PER METRE OF INSUFFICIENT SPACING
    .10    .10    .10    .10    .10    .10    .10    .10    .10
    .10    .10    .10    .10    .10    .10    .10    .10    .10

VCLB: CHANGE LANE TIME WHEN SOMEONE BEHIND
  15.00  15.00  15.00  15.00  15.00  15.00  15.00  15.00  15.00
  10.00  15.00  15.00  15.00  10.00  10.00  10.00  10.00  10.00

VCLN: CHANGE LANE TIME WHEN NOONE BEHIND
  20.00  20.00  20.00  20.00  20.00  20.00  20.00  20.00  20.00
  20.00  20.00  20.00  20.00  20.00  20.00  20.00  20.00  20.00

(6(//2(9F7.2/)))

VAM: TIME ALLOWED FOR MERGING AFTER OVERTAKING
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00
   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00   5.00

VMGA: DISTANCE COMPONENT OF END-OF-AUX.-LANE MERGING DISTANCE ALLOWED
  50.00  50.00  30.00  20.00  20.00  20.00  20.00  20.00  20.00
  20.00  50.00  20.00  30.00  20.00  20.00  10.00  30.00  20.00

VMGB: TIME COMPONENT OF SAME
  10.00  10.00  10.00   8.00   8.00   8.00   8.00   5.00   6.00
   7.00   5.00   3.00   8.00   4.00   4.00   2.00   6.00   4.00

VTS: TIME UNTIL SETTLE AFTER MERGING
  30.00  30.00  30.00  30.00  30.00  30.00  10.00  30.00  30.00
  30.00  10.00  10.00  30.00  30.00  30.00  20.00  40.00  30.00

VSS: SPEEDS FOR WHICH END-OF-AUXILIARY-LANE MERGING IS STOP-START
   5.00   5.00   5.00  10.00  10.00  10.00  10.00  10.00  10.00
  10.00  14.00  14.00  10.00  10.00  10.00  13.00   7.00  10.00

VFE: IF ACCELERATION FOR SMOOTH MERGING EXCEEDS THIS, DO NOT DECELERATE
  -1.00  -1.00  -1.00  -1.00  -1.00  -1.00  -1.00  -1.00  -1.00
  -1.00   -.80   -.50  -1.00  -1.00  -1.00  -1.20   -.80   -.50

(6(//2(9F7.2/))//2(12F5.2/)//6F8.2,I8)

VFCA: FUEL CONSUMPTION PER SECOND WHEN IDLING (ML/S)
    .00    .00    .00    .00    .00    .00    .00    .00    .00
    .00    .40    .30    .20    .15    .65    .50    .40    .60
```

VFCB:FUEL CONSUMPTION EFFICIENCY FACTOR BETA1 (ML/KJ)

.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.08	.09	.10	.13	.07	.09	.10	.11

VFCC:FUEL CONSUMPTION EFFICIENCY FACTOR BETA2 (ML/(KJ.M/S^2))

.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.05	.06	.05	.07	.04	.03	.06	.05

VFCD:DRAG FORCE PARAMETER (KN), MAINLY RELATED TO ROLLING RESISTANCE

.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.50	.28	.30	.33	.40	.30	.35	.25

VFCE:DRAG FORCE PARAMETER (KN/(M/S)^2), MAINLY RELATED TO AEROD. RESIST.

.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	1.60	.80	1.20	1.80	1.40	1.00	1.10	.80

VFCF:VEHICLE MASS (KG*1000)

.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	2.50	.80	1.10	.95	1.65	1.40	1.20	1.10

VFCG:MULT. FACTOR FOR ROLLING RESISTANCE ACCORDING TO ROAD SPEED INDEX

1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

SMX	CIOV	RTUHN	SMN	XMFH	XMIPH	MPS
30.00	.80	5.00	15.0	2.9	5.0	30

A.2 Example ROAD File

```
(///3F10.2,I10//////)

       DSS      DENDS        DUR       NURD
   2000.00    1000.00     100.00        121

SAMPLE ROAD FILE.

 CHAINAGE   BARRIER  AUXILIARY   ROAD     SIGHT DISTANCE     GRADE
    KM       LINES     LANES     SPEED     M         M         %
           (1 OR -1) (T OR F)   INDICES                      (DIR 1)
(11X,I2,3X,I2,2(4X,L1),2(3X,I2),2F10.2,F10.2)
    0.0      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.1      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.2      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.3      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.4      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.5      1    1    F    F    1    1  2000.00   2000.00     0.00
    0.6     -1   -1    F    F    1    1   100.00    100.00     0.00
    0.7     -1   -1    F    F    1    1   100.00    100.00     0.00
    0.8     -1   -1    F    F    1    1   100.00    100.00     0.00
    0.9     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.0     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.1     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.2     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.3     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.4     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.5     -1   -1    F    F    1    1   100.00    100.00     0.00
    1.6      1    1    F    F    1    1  2000.00   2000.00     0.00
    1.7      1    1    F    F    1    1  2000.00   2000.00     0.00
    1.8      1    1    F    F    1    1  2000.00   2000.00     0.00
    1.9      1    1    F    F    1    1  2000.00   2000.00     0.00
    2.0      1    1    F    F    1    1  2000.00   2000.00     0.00
```

A.3 Example TRAF File

```
(//3(F8.1/)/I8/4(F8.1/)/I8//F8.1//) FILE TRAF:  TRAFFIC FLOW CHARACTERISTICS
WHERE NOT SPECIFIED UNITS ARE IN SECONDS, METRES AND KM/H.

     1.0  BASIC TIME UNIT FOR THE SIMULATION (TUN)
  1000.0  SETTLING DOWN TIME FOR THE SIMULATION (TSE)
  3600.0  DURATION OF THE SIMULATION (TSI); NOTE THAT THE PROGRAM KEEPS RUNNING
            UNTIL ALL VEHICLES WHICH ARRIVED IN THIS TIME HAVE DEPARTED.
       1  OPTION: 1=STD; 2=USE ITRAF; 3=USE PBAYS; 4=PLOT; 5=GRAFIC DISPLAY
     0.0  LENGTH OF NO OVERTAKING TO CREATE BUNCHING IN DIRECTION 1 (DTS1)
     0.0  LENGTH OF NO OVERTAKING TO CREATE BUNCHING IN DIRECTION 2 (DTS2)
     0.0  PERCENT FOLLOWING IN PLATOONS ON ARRIVAL IN DIRECTION 1 (PFOL1)
     0.0  PERCENT FOLLOWING IN PLATOONS ON ARRIVAL IN DIRECTION 2 (PFOL2)
            NOTE ZERO %FOLL GIVES RANDOM ARRIVALS; NEG %FOLL USES DEFAULTS.
       1  NUMBER OF VEHICLE GENERATION CATEGORIES (NSTR); CHECK FORMATS IN THIS
            FILE IF NSTR IS CHANGED.  ONLY NSTR OF THE COLUMNS ARE READ.
   213.0  RANDOM SEED NUMBER (NSEEDO); RANGE IS 0. TO 999999.

THE REMAINING PARAMETERS DESCRIBE THE SIMULATED TRAFFIC STREAM
(////A8/18(F8.2/)//4(F8.2/)//F8.2/2(//F8.2/)//I8///2(8F8.1/))
ADTV@D PROPORTIONS OF VEHICLE TYPES IN VARIOUS CATEGORIES
**********************************************************************
*                 TRAFFIC GENERATION CATEGORIES                * TYPE  *
**********************************************************************
CARS    EXTRA   EXTRA   EXTRA   SPARE   EXTRA1  EXTRA2  EXTRA3   *       *
0.      0.      0.      0.      0.      1.      0.      0.       *   1   *
0.      0.      0.      0.      0.      0.      1.      0.       *   2   *
0.      0.      0.      0.      0.      0.      0.      1.       *   3   *
0.      0.      0.      0.      0.      0.      0.      0.       *   4   *
0.      0.      0.      0.      0.      0.      0.      0.       *   5   *
0.      0.      0.      0.      0.      0.      0.      0.       *   6   *
0.      0.      0.      0.      0.      0.      0.      0.       *   7   *
0.      0.      0.      0.      0.      0.      0.      0.       *   8   *
0.      0.      0.      0.      0.      0.      0.      0.       *   9   *
0.      0.      0.      0.      0.      0.      0.      0.       *  10   *
0.      0.      0.      0.      0.      0.      0.      0.       *  11   *
0.1     0.      0.      0.      0.      0.      0.      0.       *  12   *
0.1     0.      0.      0.      0.      0.      0.      0.       *  13   *
0.2     0.      0.      0.      0.      0.      0.      0.       *  14   *
0.      0.      0.      0.      0.      0.      0.      0.       *  15   *
0.2     0.      0.      0.      0.      0.      0.      0.       *  16   *
0.3     0.      0.      0.      0.      0.      0.      0.       *  17   *
0.1     0.      0.      0.      0.      0.      0.      0.       *  18   *
```

```
ADVGC@D TWO-DIRECTIONAL TRAFFIC VOLUME (VEH/H) FOR EACH CATEGORY
0.50    0.      0.      0.      0.      0.      0.      0.   DIR1 BASIC LANE
0.0     0.      0.      0.      0.      0.      0.      0.        AUX. LANE
0.50    0.      0.      0.      0.      0.      0.      0.   DIR2 BASIC LANE
0.      0.      0.      0.      0.      0.      0.      0.        AUX. LANE

VMIT@D TWO-DIRECTIONAL TRAFFIC VOLUME(VEH/H) FOR EACH CATEGORY
  2000.0    0.0     0.0     0.0     0.0     0.0     0.0     0.0

VMF@D MEAN DESIRED SPEED(KM/H)
   100.0    0.0     0.0     0.0     0.0     0.0     0.0     0.0

VSDF@D STANDARD DEVIATION OF DESIRED SPEEDS(KM/H)
     8.3    0.0     0.0     0.0     0.0     0.0     0.0     0.0

LFSD@D INDICES INDICATING TYPE OF SPEED DISTRIBUTION
       1       1       1       1       1       1       1       1

PFQ1@D DEFAULT PLATOONING-FLOW DISTRIBUTION USED WHEN PFOL IS INPUT AS -1
0.      200.    400.    800.    1200.   1600.   2000.   2800.
0.       15.     30.     50.      65.     75.     90.    100.
```

A.4 Example OBS File

```
 26      NOB   NUMBER OF OBSERVATION POINTS
 13      NOB1  NUMBER OF OBSERVATION POINTS IN DIRECTION 1

  2      LMSP  INTERVAL STARTING POINT DIRECTION 1
 15      LMSP  INTERVAL STARTING POINT DIRECTION 2

 12      LMFP  INTERVAL FINISHING POINT DIRECTION 1
 25      LMFP  INTERVAL FINISHING POINT DIRECTION 2

 12      KOBS1 GAP DISTRIBUTION RECORDING POINT DIRECTION 1
 25      KOBS2 GAP DISTRIBUTION RECORDING POINT DIRECTION 2

  1      NCT   NUMBER OF VEHICLE OBSERVATION CATEGORIES

5.0      TFOL  TIME COMPONENT OF DEFINITION OF FOLLOWING
0.0      DFOL  DISTANCE COMPONENT OF DEFINITION OF FOLLOWING

  2      IFILE OPTION TO GENERATE ADDITIONAL OUTPUT INFORMATION

VEHICLE CATEGORY NAMES
1=CARS   ;2=        ;3=        ;4=        ;5=        ;6=

VEHICLE CATEGORIES FOR THE VARIOUS VEHICLE TYPES (LVC)
 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1

OBS PTS FOR FIRST DIRECTION (RELATIVE TO START OF SIM. SEGMENT)
  100. 1000. 2000. 3000. 4000. 5000. 6000. 7000. 8000. 9000.
10000.11000.11900.

OBS PTS FOR SECOND DIRECTION (FROM START OF SIM. SEG. IN DIR 2)
  100. 1000. 2000. 3000. 4000. 5000. 6000. 7000. 8000. 9000.
10000.11000.11900.

/EOF
```

Appendix B

Sensitivity Analysis

Figure B.1: Sensitivity Analysis — VSFVN
Overtaking Rates
50/50 Directional Split

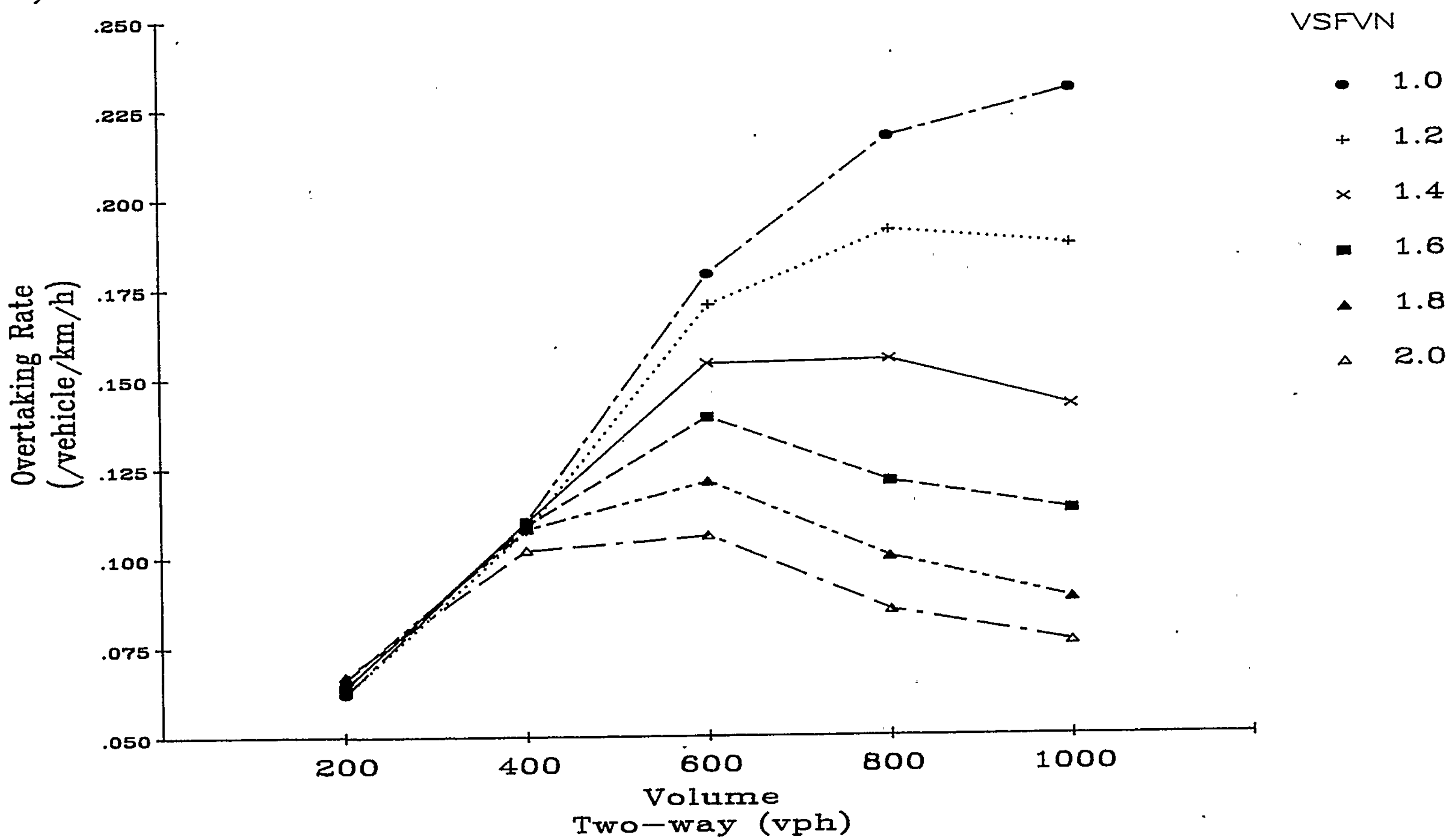


Figure B.2: Sensitivity Analysis — VSFVN
Mean Speed
50/50 Directional Split

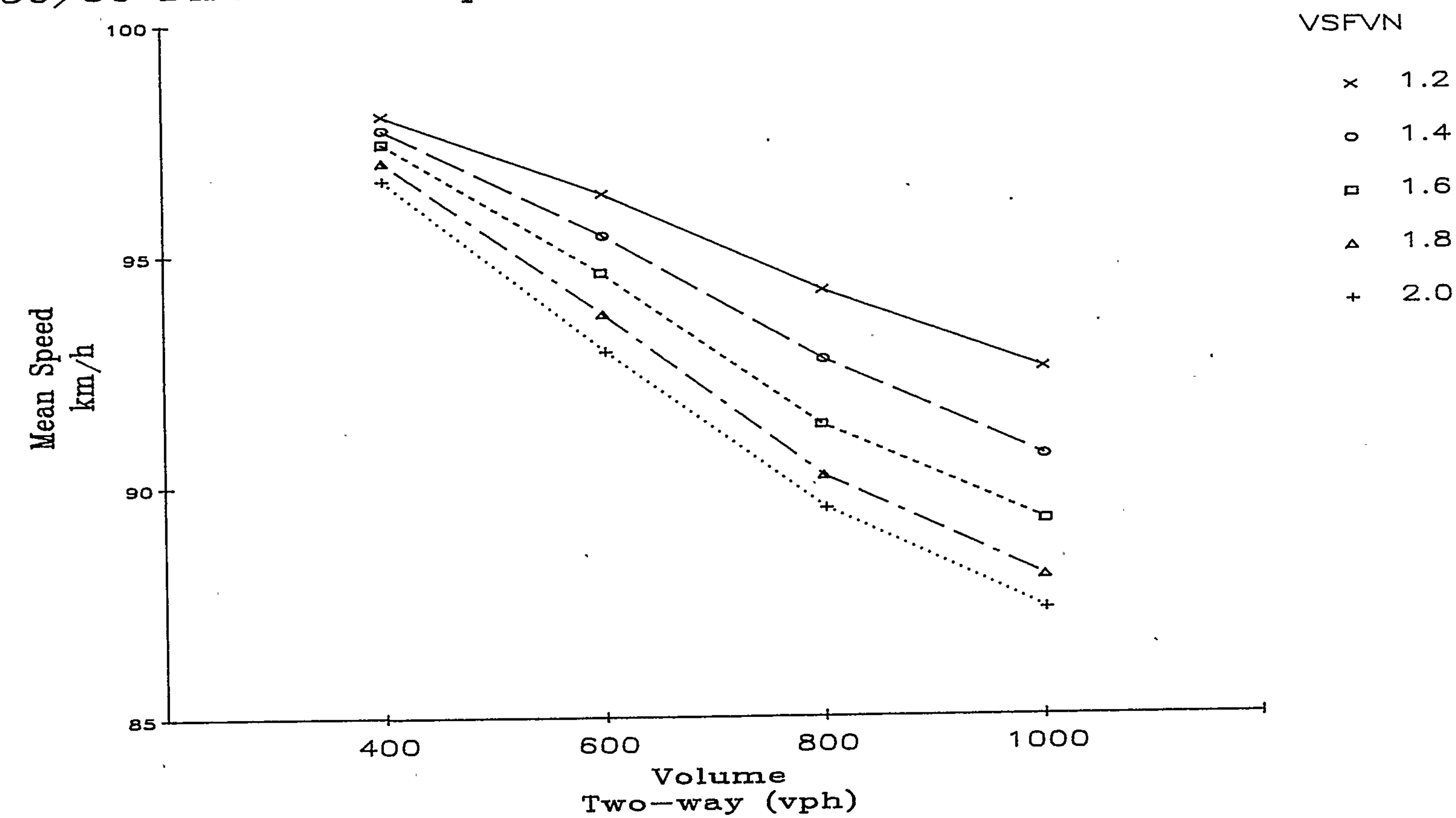
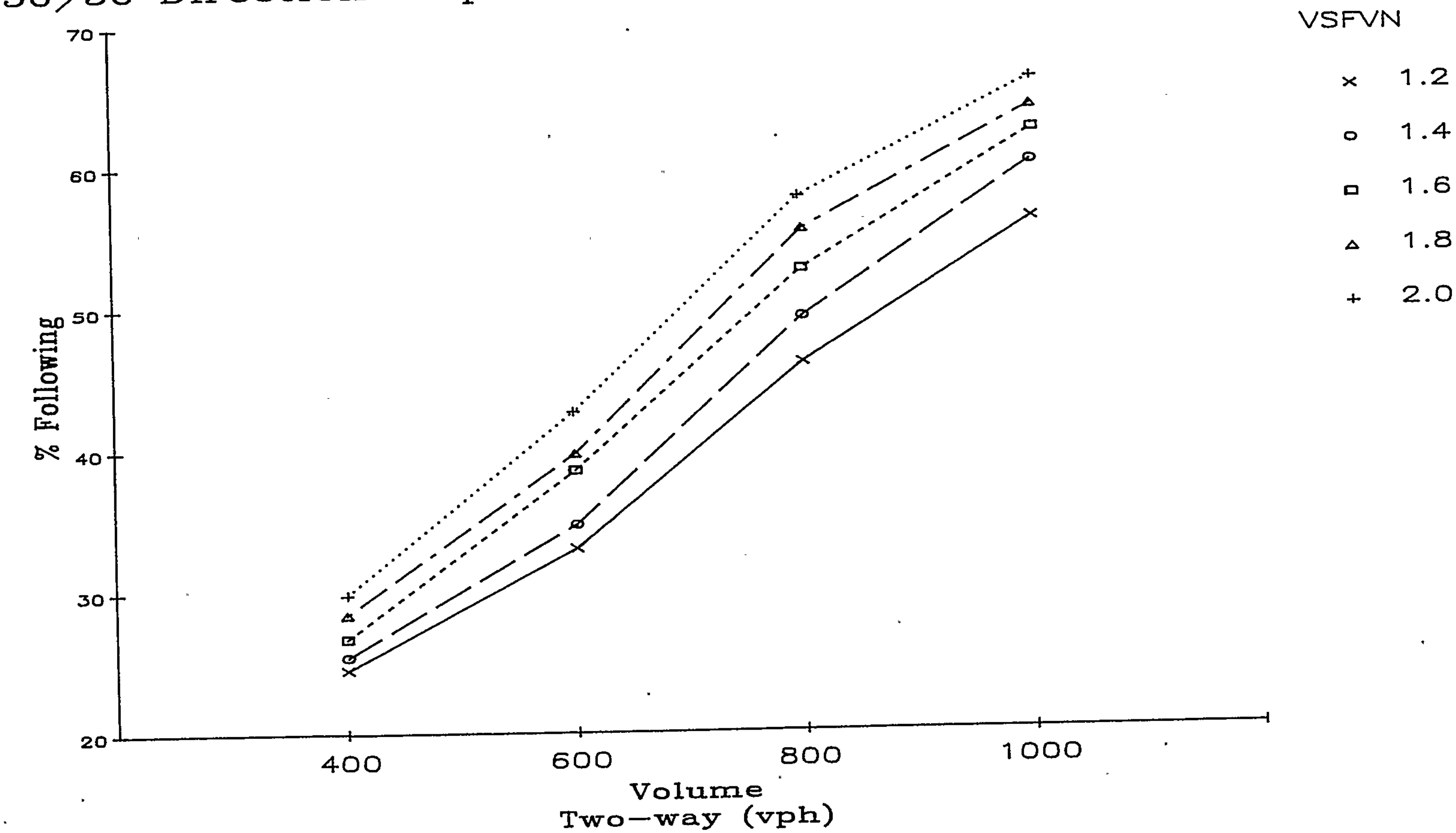


Figure B.3: Sensitivity Analysis — VSFVN
% Following
50/50 Directional Split



VSFSN Safety Factor Analysis

To reduce the number of simulation runs required in this analysis, three values were used for the VSFSN. Case A used the original factors determined at ARRB, Case C was with VSFSN = 1.0 and Case B took a value between Case A and B. Six traffic streams were modelled with a 50/50 directional split.

Table B.1: VSFSN Safety Factor - Test Cases

Vehicle Type No.	Case A	Case B	Case C
1	2.0	1.5	1.0
2	2.0	1.5	1.0
3	1.8	1.4	1.0
4	1.6	1.3	1.0
5	1.6	1.3	1.0
6	1.6	1.3	1.0
7	2.0	1.5	1.0
8	1.6	1.3	1.0
9	1.6	1.3	1.0
10	1.6	1.3	1.0
11	1.8	1.4	1.0
12	2.5	1.8	1.0
13	1.5	1.3	1.0
14	1.7	1.4	1.0
15	1.7	1.4	1.0
16	2.0	1.5	1.0
17	1.4	1.2	1.0
18	1.4	1.2	1.0

Figure B.4: Sensitivity Analysis — VSFSN
Overtakings
Volume = 400

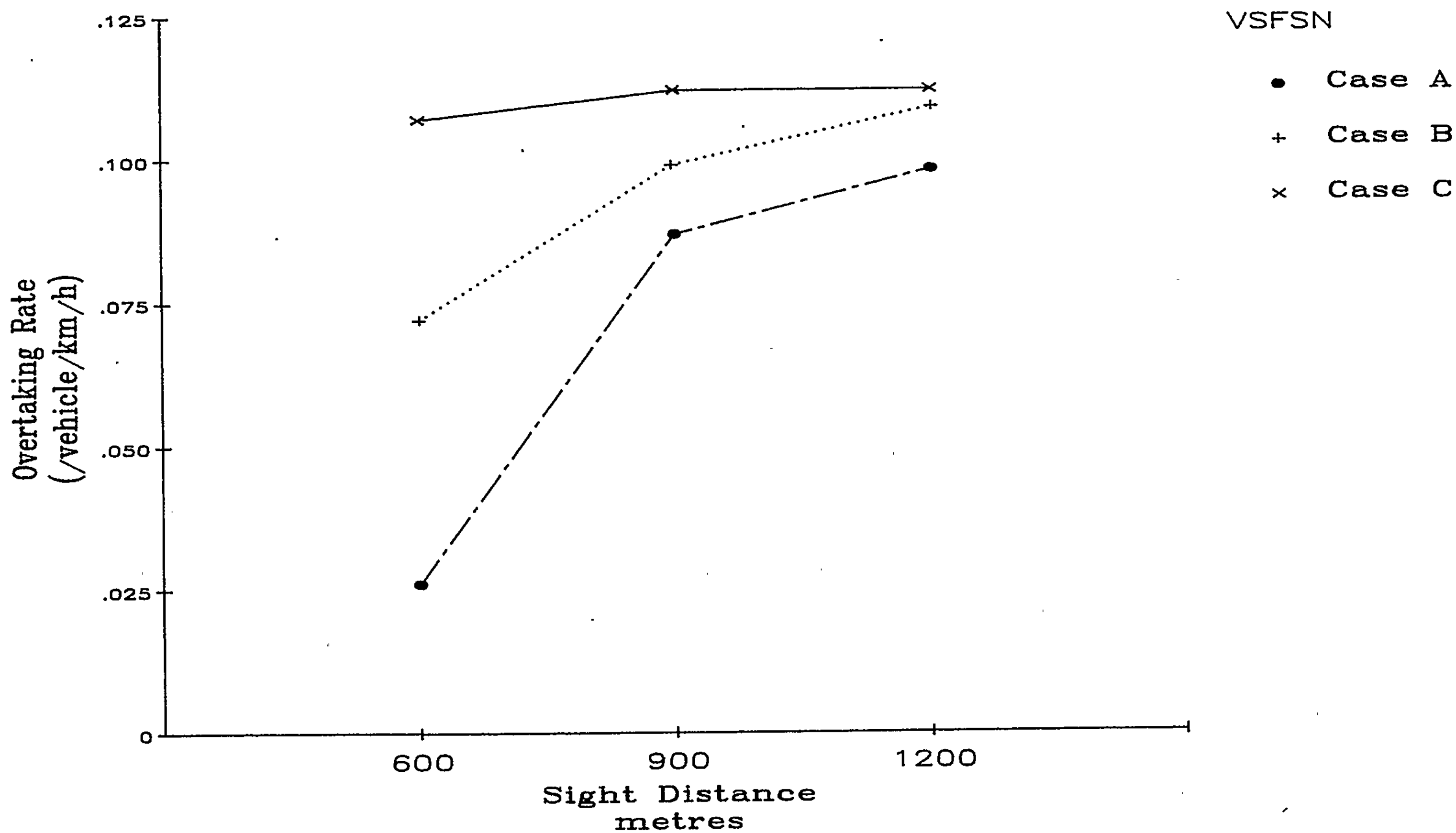


Figure B.5: Sensitivity Analysis — VSFSN
Overtaking Rates
Volume = 800 vph

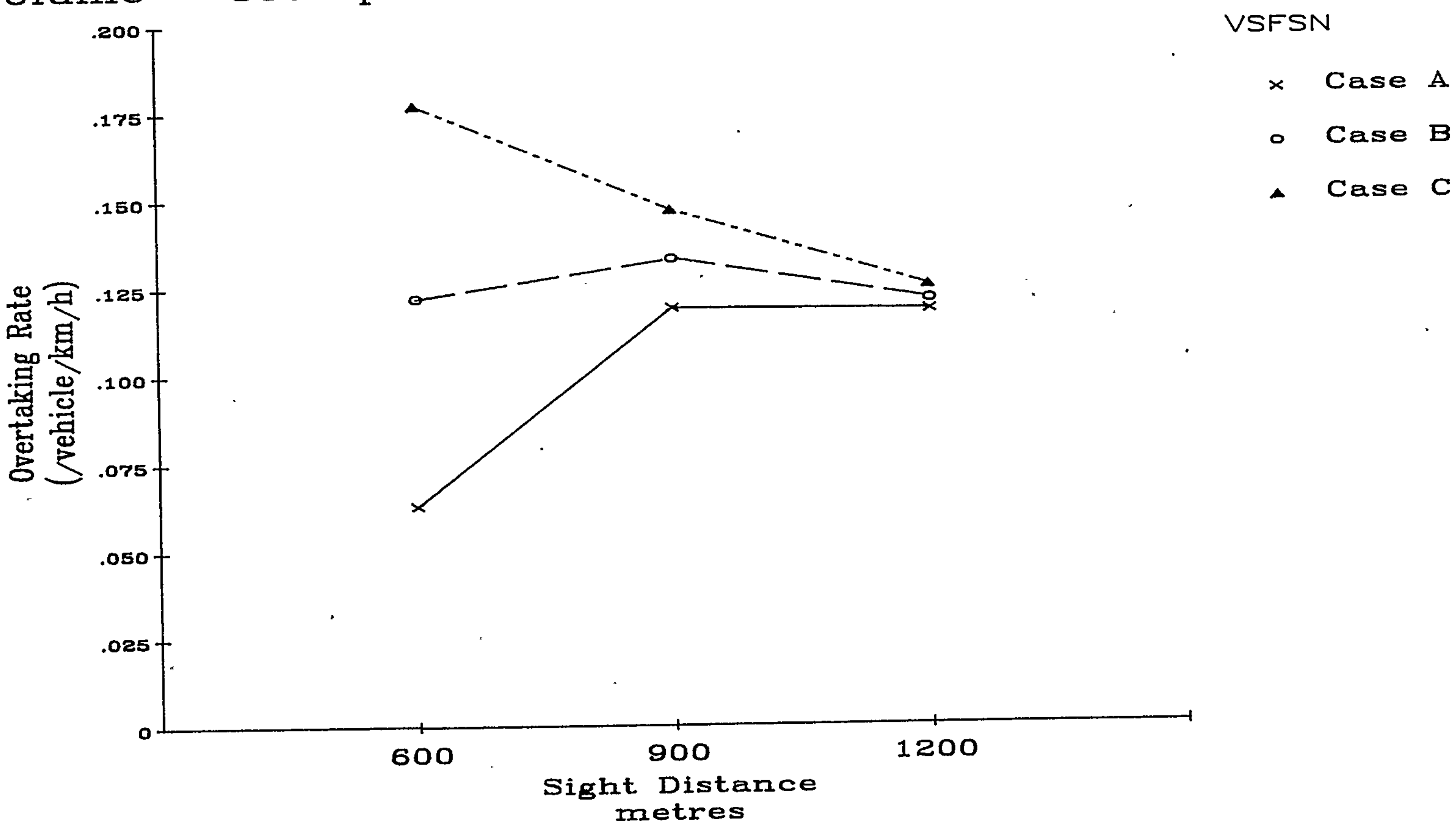


Figure B.6: Sensitivity Analysis — VSFSN
Mean Speed
Volume = 400

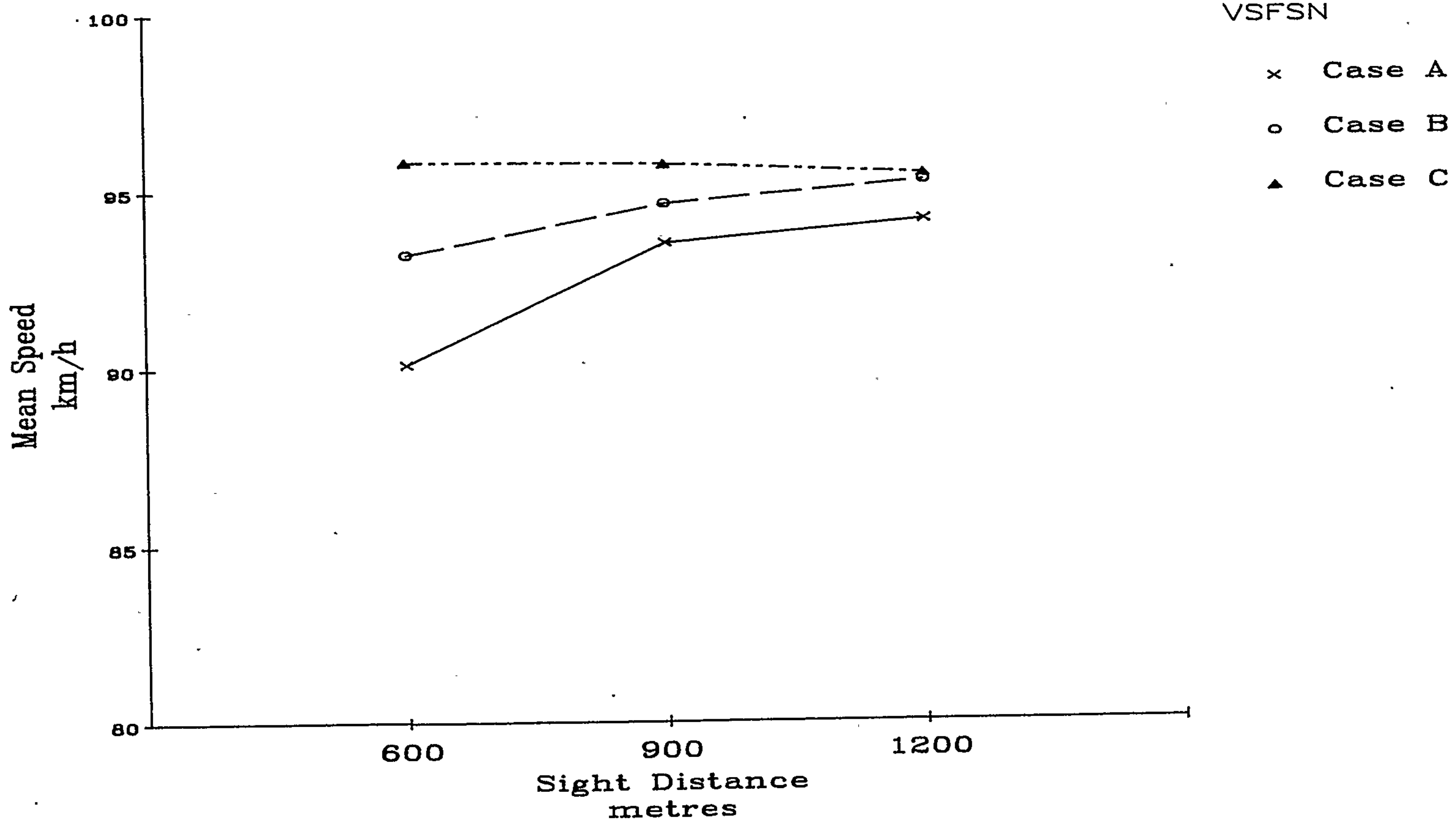


Figure B.7: Sensitivity Analysis
Mean Speed
Volume = 800 vph

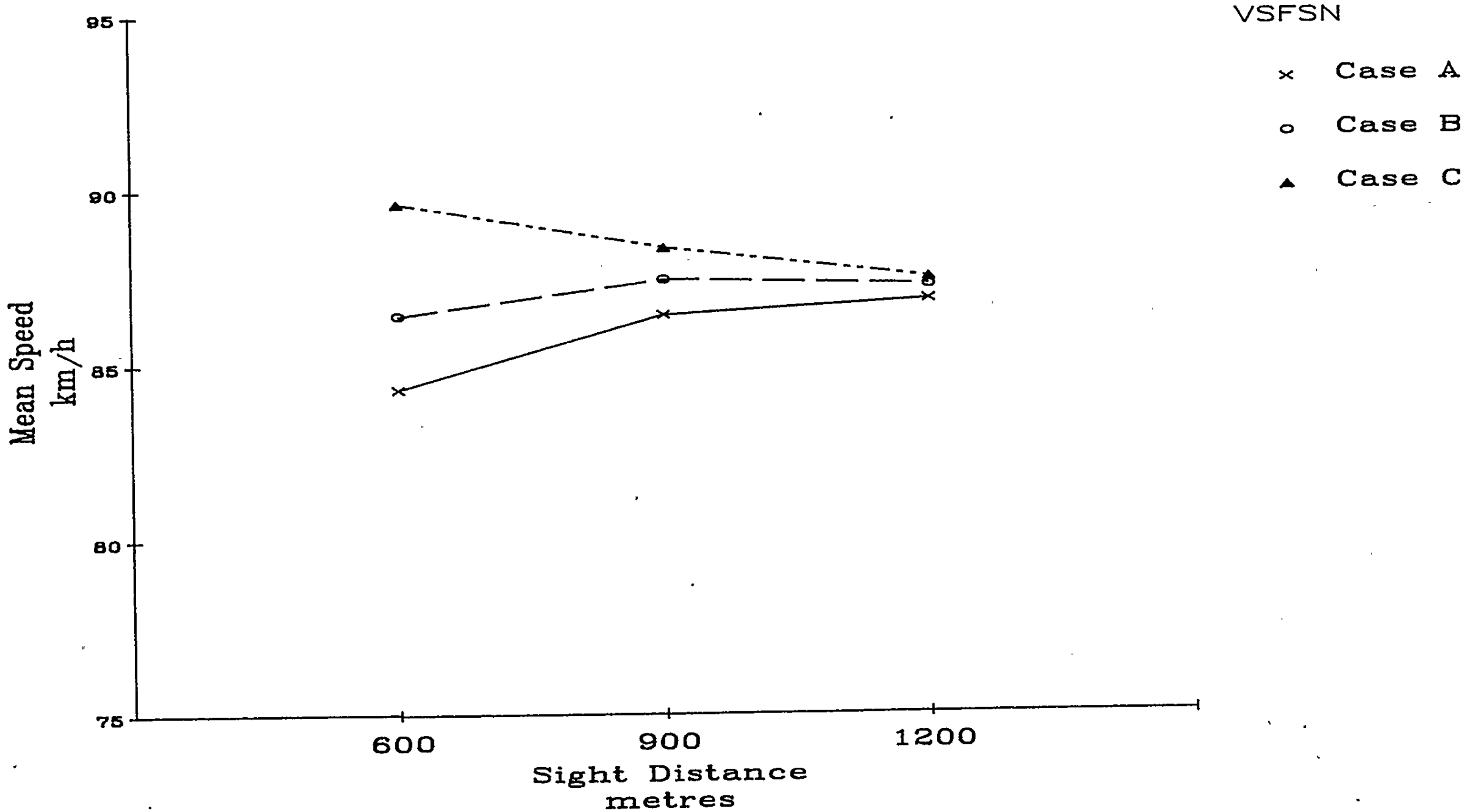


Figure B.8: Sensitivity Analysis — VSFSN
% Following
Volume = 400 vph

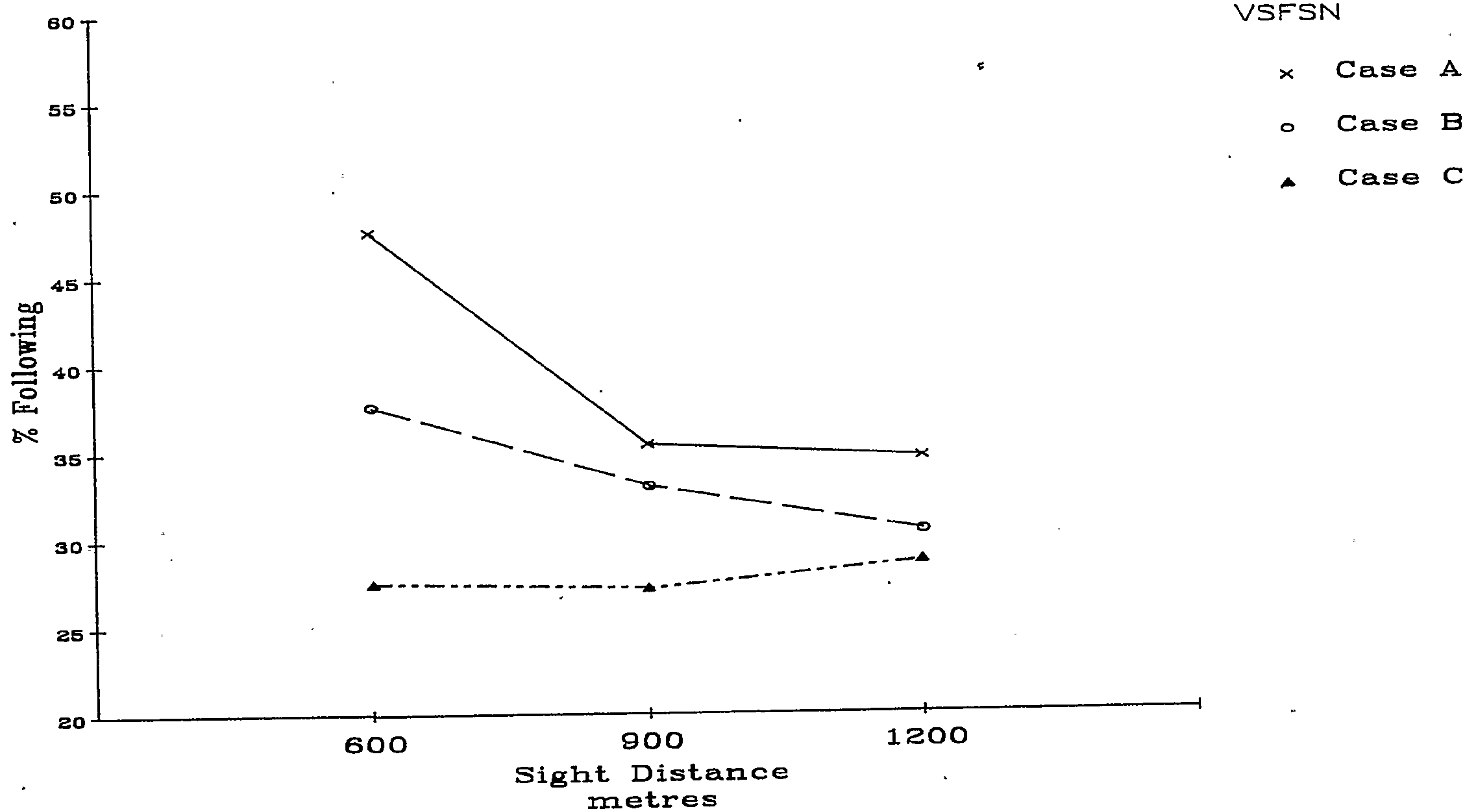
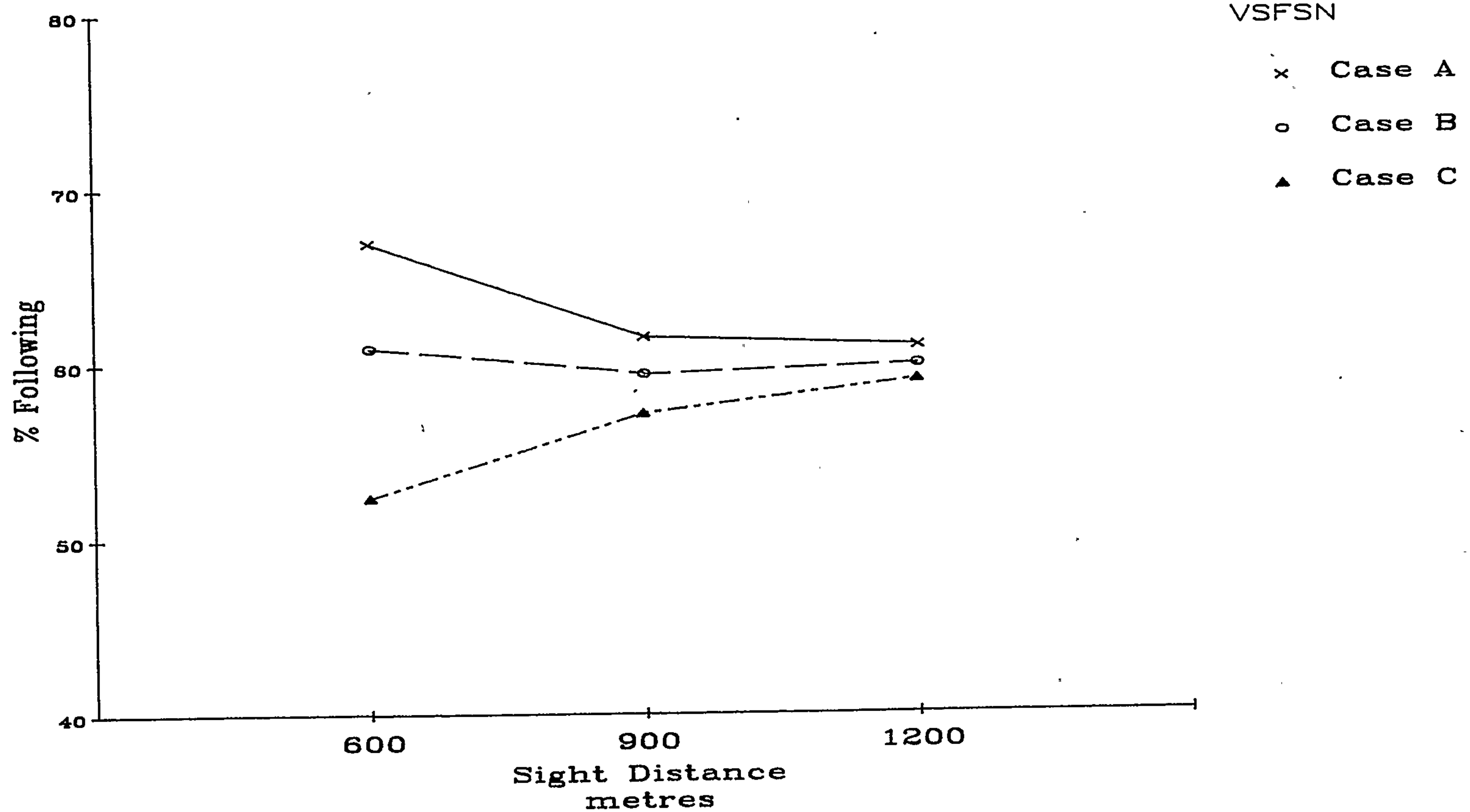


Figure B.9: Sensitivity Analysis ä VSFSN
% Following
Volume = 800 vph



Appendix C

Traffic Recording System

To facilitate the collection of calibration data a traffic monitoring system was installed on the test sites. This equipment was supplied by International Road Dynamics (IRD) of Saskatoon, Saskatchewan. The system consisted of permanent inductive loops and a traffic statistics recorder (TSR). Data was recovered with a reader programmer unit (RPU) or communications software (TELECOM package) supplied by the vendor, operated through a portable computer.

The IRD system is designed to record traffic capacity and congestion statistics. An example of output from the counters follows. Vehicle classification was determined by length bins. Statistics output by the counting equipment include volume, mean speed and standard deviation, percentage in platoons, average headways, number of gaps greater than 25 seconds and the percentage of the hour with gaps greater than 25 seconds. The IRD equipment provides the flexibility of allowing the user to input values to define following vehicles and gap size. In this study, a vehicle was defined to be in a following state if it was travelling at a headway of 5 seconds or less. Gap sizes in the opposing traffic stream greater than or equal to 25 seconds were recorded to correspond with previous work on the Unified Model. These data were used directly for input and output comparison with the TRARR model results.

PAGE: 9

SITE ID: HWY 1 BANFF PAGE 9 for LANE NUMBER 1 OF 2

Census Start Time: 12:00 Mar 10/86

Census End Time: Continuous

Data Extracted: 10:03 Mar 18/86

Printed: 9:41 Mar 19/86 Traffic Monitor: TSR-1040

Stream: Free Moving= 10.00 sec Platoon= 4.50 sec Upper Speed= 100 Km/hr Gap= 30.00 sec

			(___Speed Statistics___)					(________Free Moving________)					(___All Speeds Platoons___					___Gap___)		
Rec#	Date	Time	Total Count	Class Count	Mean Km/hr	StDev Km/hr	Skew	1/Spd Hr/Km	Class Count	Mean Km/hr	StDev Km/hr	Skew	1/Spd Hr/Km	Pass Count	#Plat	ALL Avg#	SPDS AvHdy	%Plat	# of Gaps	Mean Gap	% of Intvl
00180	860318	0:00	25	25	94.4	11.0	+0.51	0.27	25	94.4	11.0	+0.51	0.27	0	0	0.0	0.00	0.0	20	173.50	96.5
00181	860318	1:00	22	21	88.4	4.9	-0.40	0.24	21	88.4	4.9	-0.40	0.24	1	0	0.0	0.00	0.0	17	205.00	97.0
00182	860318	2:00	10	10	100.7	11.5	+1.34	0.10	10	100.7	11.5	+1.34	0.10	0	0	0.0	0.00	0.0	8	441.00	98.0
00183	860318	3:00	11	10	91.3	5.3	+0.82	0.11	8	91.8	5.8	+0.57	0.09	0	0	0.0	0.00	0.0	9	398.75	99.5
00184	860318	4:00	10	10	94.9	6.1	-0.50	0.11	10	94.9	6.1	-0.50	0.11	0	0	0.0	0.00	0.0	10	357.50	99.5
00185	860318	5:00	13	13	97.6	10.8	+1.28	0.13	10	99.3	11.7	+0.92	0.10	0	1	2.0	2.70	7.5	9	395.25	99.0
00186	860318	6:00	31	30	98.0	8.9	+1.26	0.31	27	98.3	9.4	+1.12	0.28	0	1	2.0	4.13	3.5	26	137.00	99.0
00187	860318	7:00	83	83	96.1	8.2	+0.34	0.87	57	96.6	8.9	+0.31	0.59	0	10	2.1	2.83	13.5	38	81.50	86.0
00188	860318	8:00	323	322	94.1	7.2	+0.34	3.44	103	95.4	7.9	+0.39	1.09	0	73	3.2	2.30	50.0	30	51.50	43.0
00189	860318	9:00	270	270	91.7	6.8	-0.10	2.96	114	92.3	7.1	+0.01	1.24	0	57	3.0	2.10	41.5	35	45.25	44.0

---------- END OF TABLES FOR LANE#1 ------------

Figure C.1: Sample IRD Output

Appendix D

Calibration Tables

Table D.1: Calibration (volume = 191)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	97.1	99.6	2.5	2.6
σ (Km/h)	11.6	8.7	-2.9	-25.0
No. of Platoons	38	40	2.0	5.3
% Following	35	39	4.0	11.4
No. of Gaps > 30s.	46	45	-1.0	-2.2
% of Hour with Gap > 30s	59	68	9.0	15.3

Table D.2: Calibration (volume = 173)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	100.9	103.1	2.2	2.2
σ (Km/h)	10.8	10.3	-0.5	-4.6
No. of Platoons	29	30	1.0	3.4
% Following	25	26	1.0	4.0
No. of Gaps > 30s.	44	41	-3.0	-6.8
% of Hour with Gap > 30s	69	72	3.0	4.3

Table D.3: Calibration (volume = 193)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	98.4	99.6	1.2	1.2
σ (Km/h)	10.7	9.1	-1.6	-15.0
No. of Platoons	36	43	7.0	19.4
% Following	38	43	5.0	13.2
No. of Gaps > 30s.	37	42	5.0	13.5
% of Hour with Gap > 30s	61	60	-1.0	-1.6

Table D.4: Calibration (volume = 193)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	87.2	102.7	15.5	17.8
σ (Km/h)	15.1	9.3	-5.8	-38.4
No. of Platoons	36	41	5.0	13.9
% Following	41	36	-5.0	-12.2
No. of Gaps > 30s.	47	44	-3.0	-6.4
% of Hour with Gap > 30s	70	70	0.0	0.0

Table D.5: Calibration (volume = 121)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	102.4	102.5	0.1	0.1
σ (Km/h)	10.7	8.3	-2.4	-22.4
No. of Platoons	17	16	-1.0	-5.9
% Following	19	19	0.0	0.0
No. of Gaps > 30s.	47	47	0.0	0.0
% of Hour with Gap > 30s	76	81	5.0	6.6

Table D.6: Calibration (volume = 63)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	104.7	103.3	-1.4	-1.3
σ (Km/h)	12.1	9.7	-2.4	-19.8
No. of Platoons	6	9	3.0	50.0
% Following	11	14	3.0	27.3
No. of Gaps > 30s.	34	35	1.0	2.9
% of Hour with Gap > 30s	87	91	4.0	4.6

Table D.7: Calibration (volume = 286)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	100.2	99.9	-0.3	-0.3
σ (Km/h)	10.2	8.8	-1.4	-13.7
No. of Platoons	61	63	2.0	3.3
% Following	47	53	6.0	12.8
No. of Gaps > 30s.	32	40	8.0	25.0
% of Hour with Gap > 30s	45	51	6.0	13.3

Table D.8: Calibration (volume = 187)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	99.6	104.4	4.8	4.8
σ (Km/h)	11.2	10.7	-0.5	-4.5
No. of Platoons	28	40	12.0	42.9
% Following	26	35	9.0	34.6
No. of Gaps > 30s.	44	42	-2.0	-4.5
% of Hour with Gap > 30s	62	66	4.0	6.5

Table D.9: Passing Lane Calibration - Southbound (volume = 125)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	102.9	105.5	2.6	2.5
σ (Km/h)	8.0	8.8	0.8	10.0
No. of Platoons	22	18	-4.0	-18.2
Ave. Platoon	2.1	2.4	0.3	14.3
% Following	19	18	-1.0	-5.3
No. of Gaps > 30s.	43	46	3.0	7.0
% of Hour with Gap > 30s	73	74	1.0	1.4

Table D.10: Passing Lane Calibration - Southbound (volume = 155)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	100.9	106.0	5.1	5.1
σ (Km/h)	10.8	8.7	-2.1	-19.4
No. of Platoons	28	21	-7.0	-25.0
Ave. Platoon	2.6	2.4	-0.2	-7.7
% Following	29	17	-12.0	-41.4
No. of Gaps > 30s.	41	45	4.0	9.8
% of Hour with Gap > 30s	67	74	7.0	10.4

Table D.11: Passing Lane Calibration - Southbound (volume = 327)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	104.2	105.6	1.4	1.3
σ (Km/h)	8.6	9.3	0.7	8.1
No. of Platoons	70	72	2.0	2.9
Ave. Platoon	3.0	3.0	0.0	0.0
% Following	43	35	-8.0	-18.6
No. of Gaps > 30s.	19	26	7.0	36.8
% of Hour with Gap > 30s	31	33	2.0	6.5

Table D.12: Passing Lane Calibration - Southbound (volume = 276)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	107.2	105.8	-1.4	-1.3
σ (Km/h)	9.5	9.2	-0.3	-3.2
No. of Platoons	54	66	12.0	22.2
Ave. Platoon	3.1	2.7	-0.4	-12.9
% Following	41	33	-8.0	-19.5
No. of Gaps > 30s.	30	31	1.0	3.3
% of Hour with Gap > 30s	40	39	-1.0	-2.5

Table D.13: Passing Lane Calibration - Northbound (volume = 136)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	105.0	102.6	-2.4	-2.3
σ (Km/h)	11.3	9.1	-2.2	-19.5
No. of Platoons	19	18	-1.0	-5.3
Ave. Platoon	2.5	2.3	-0.2	-8.0
% Following	21	18	-3.0	-14.3
No. of Gaps > 30s.	41	39	-2.0	-4.9
% of Hour with Gap > 30s	73	73	0.0	0.0

Table D.14: Passing Lane Calibration - Northbound (volume = 168)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	105.3	103.7	-1.6	-1.5
σ (Km/h)	10.5	8.6	-1.9	-18.1
No. of Platoons	17	17	0.0	0.0
Ave. Platoon	2.4	2.4	0.0	0.0
% Following	18	21	3.0	16.7
No. of Gaps > 30s.	42	42	0.0	0.0
% of Hour with Gap > 30s	73	75	2.0	2.7

Table D.15: Passing Lane Calibration - Northbound (volume = 263)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	107.6	103.2	-4.4	-4.1
σ (Km/h)	13.1	8.6	-4.5	-34.4
No. of Platoons	47	59	12.0	25.5
Ave. Platoon	3.6	3.1	-0.5	-13.9
% Following	47	49	2.0	4.3
No. of Gaps > 30s.	38	35	-3.0	-7.9
% of Hour with Gap > 30s	51	54	3.0	5.9

Table D.16: Passing Lane Calibration - Northbound (volume = 196)

Traffic Characteristic	Observed	Modelled	Δ	% Difference
Speed (Km/h)	105.6	103.3	-2.3	-2.2
σ (Km/h)	8.4	8.4	0.0	0.0
No. of Platoons	45	37	-8.0	-17.8
Ave. Platoon	2.6	2.5	-0.1	-3.8
% Following	36	34	-2.0	-5.6
No. of Gaps > 30s.	42	38	-4.0	-9.5
% of Hour with Gap > 30s	63	64	1.0	1.6

Appendix E

Validation Tables

Table E.1: Validation - Highway 93 (volume = 149)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	89.0	90.2	89.1	0.1	0.1
σ (Km/h)	10.5	10.9	12.9	2.4	22.9
% Following	33	25	34	1.0	3.0
No. of Overtakings	11	19	17	6.0	54.5

Table E.2: Validation - Highway 93 (volume = 167)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	95.0	90.6	89.3	-5.7	-6.0
σ (Km/h)	8.8	12.7	12.9	4.1	46.6
% Following	40	36	41	1.0	2.5
No. of Overtakings	28	46	37	9.0	32.1

Table E.3: Validation - Highway 93 (volume = 143)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	88.0	90.5	90.6	2.6	3.0
σ (Km/h)	11.7	12.5	11.5	-0.2	-1.7
% Following	25	28	30	5.0	20.0
No. of Overtakings	16	15	18	2.0	12.5

Table E.4: Validation - Highway 93 (volume = 109)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	92.0	88.2	87.7	-4.3	-4.7
σ (Km/h)	10.5	9.3	10.0	-0.5	-4.8
% Following	29	34	40	11.0	37.9
No. of Overtakings	9	10	12	3.0	33.3

Table E.5: Validation - Highway 93 (volume = 177)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	95.0	84.6	81.6	-13.4	-14.1
σ (Km/h)	9.7	13.3	4.8	2.1	21.6
% Following	44	54	63	19.0	43.2
No. of Overtakings	16	36	21	5.0	31.3

Table E.6: Validation - Highway 93 (volume = 221)

Traffic Characteristic	Observed	Model	Adjusted Model	Δ	% Difference
Speed (Km/h)	92.0	87.4	90.1	-1.9	-2.1
σ (Km/h)	10.5	12.2	12.1	1.6	15.2
% Following	45	38	35	-10.0	-22.2
No. of Overtakings	38	59	54	16.0	42.1

Table E.7: Validation Check - RDSTAV3 (volume = 191)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	97.1	100.5	3.4	3.5
σ (Km/h)	11.6	9.0	-2.6	-22.4
No. of Platoons	38	41	3.0	7.9
% Following	35	39	4.0	11.4
No. of Gaps > 30s.	46	42	-4.0	-8.7
% of Hour with Gap > 30s	59	66	7.0	11.9

Table E.8: Validation Check - RDSTAV3 (volume = 173)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	100.9	103.9	3.0	3.0
σ (Km/h)	10.8	10.6	-0.2	-1.9
No. of Platoons	29	26	-3.0	-10.3
% Following	25	28	3.0	12.0
No. of Gaps > 30s.	44	44	0.0	0.0
% of Hour with Gap > 30s	69	67	-2.0	-2.9

Table E.9: Validation Check - RDSTAV3 (volume = 193)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	98.4	102.2	3.8	3.9
σ (Km/h)	10.7	9.4	-1.3	-12.1
No. of Platoons	36	45	9.0	25.0
% Following	38	40	2.0	5.3
No. of Gaps > 30s.	37	42	5.0	13.5
% of Hour with Gap > 30s	61	55	-6.0	-9.8

Table E.10: Validation Check - RDSTAV3 (volume = 193)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	87.2	102.6	15.4	17.7
σ (Km/h)	15.1	9.4	-5.7	-37.7
No. of Platoons	36	39	3.0	8.3
% Following	41	40	-1.0	-2.4
No. of Gaps > 30s.	47	45	-2.0	-4.3
% of Hour with Gap > 30s	70	77	7.0	10.0

Table E.11: Validation Check - RDSTAV3 (volume = 121)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	102.4	102.9	0.5	0.5
σ (Km/h)	10.7	8.8	-1.9	-17.8
No. of Platoons	17	10	-7.0	-41.2
% Following	19	17	-2.0	-10.5
No. of Gaps > 30s.	47	47	0.0	0.0
% of Hour with Gap > 30s	76	81	5.0	6.6

Table E.12: Validation Check - RDSTAV3 (volume = 63)

Traffic Characteristic	Observed	Validated Model	Δ	% Difference
Speed (Km/h)	104.7	103.4	-1.3	-1.2
σ (Km/h)	12.1	9.1	-3.0	-24.8
No. of Platoons	6	13	7.0	116.7
% Following	11	13	2.0	18.2
No. of Gaps > 30s.	34	33	-1.0	-2.9
% of Hour with Gap > 30s	87	90	3.0	3.4

Table E.13: Validation Check - RDSTAV3 (volume = 286)

Traffic Characteristic	**Observed**	**Validated Model**	**Δ**	**% Difference**
Speed (Km/h)	100.2	102.2	2.0	2.0
σ (Km/h)	10.2	8.7	-1.5	-14.7
No. of Platoons	61	64	3.0	4.9
% Following	47	50	3.0	6.4
No. of Gaps > 30s.	32	34	2.0	6.3
% of Hour with Gap > 30s	45	44	-1.0	-2.2

Table E.14: Validation Check - RDSTAV3 (volume = 187)

Traffic Characteristic	**Observed**	**Validated Model**	**Δ**	**% Difference**
Speed (Km/h)	99.6	105.7	6.1	6.1
σ (Km/h)	11.2	11.0	-0.2	-1.8
No. of Platoons	28	37	9.0	32.1
% Following	26	30	4.0	15.4
No. of Gaps > 30s.	44	45	1.0	2.3
% of Hour with Gap > 30s	62	67	5.0	8.1