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Investigating Intelligent Transportation Systems Strategies On The Santa Monica Freeway Corridor

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University of California, Berkeley

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INVESTIGATING INTELLIGENT TRANSPORTATION SYSTEMS STRATEGIES ON THE SANTA MONICA FREEWAY CORRIDOR

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Research Report
California PATH Program
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This report is part of an effort to simulate various ITS strategies on the Santa Monica Freeway Corridor (I-10) in Los Angeles. This corridor is also known as the "Smart Corridor" because of the project of the same name that is currently underway on the corridor. While many of the data used for this report were obtained from the agencies involved in the Smart Corridor project, it should be made clear that this research was conducted at the University of California at Berkeley and is not a part of the Smart Corridor project itself The results arrived at in this report do not necessarily reflect the views of the agencies involved in the Smart Corridor project.

This work was performed as part of the California PATH Program of the University of California, in cooperation with the State of California Business, Transportation, and Housing Agency, Department of Transportation; and the United States Department of Transportation, Federal Highway Administration.

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California. This report does not constitute a standard, specification or regulation.

ABSTRACT

Report Title: Investigating Intelligent Transportation Systems Strategies on the Santa Monica Freeway Corridor

Authors: Vinton W. Bacon Jr., John R. Windover, and Adolf D. May

This report describes a multi-year research effort to simulate the Santa Monica Freeway Corridor in Los Angeles using the INTEGRATION computer simulation model, and to investigate the potential benefits of Intelligent Transportation Systems (ITS). The simulation process involved a massive data collection effort, coding the collected data into the model, testing and calibration of the INTEGRATION model, and finally, performing investigations with the calibrated network.

The data collection efforts were very extensive involving over a year's efforts. The resulting data set included supply, demand and control data and is believed to be the most extensive data set compiled for the Santa Monica Freeway corridor. This data set was used to code a network consisting of 111 origin and destination nodes, 1,747 nodes, and 3,286 links. An involved demand estimation process using the QUEENSOD and INTEGRATION programs led to the creation of origin/demand matrices for each half hour time slice from 6:00 a.m. to 8:00 p.m.

Because of the novel nature of the INTEGRATION program, numerous tests were performed to determine the capabilities of the program before moving ahead with the calibration effort. The calibration of the model was divided into three parts representing the morning peak period, the mid-day period and the evening peak period. The calibration efforts began by attempting the calibration of the first time slice of the morning peak period (6:00 a.m. to 6:30 a.m.). This allowed for the identification of problems in the modeling effort, and the incorporation of refinements in the demand estimation and traffic simulation processes. The effort to calibrate the morning peak period resulted in a reasonable calibration of time slices one, two, and three. However, after an extensive effort, a reasonable calibration of time slices four through six was not attained. Similar results were obtained for the mid-day period. Calibration of the evening peak period was not attempted.

The initial investigation plan was to study the impact of various Advanced Traffic Information Systems (ATIS) and Advanced Traffic Management Systems (ATMS) control strategies. However, due to time and fund restrictions, coupled with limitations of the ATIS INTEGRATION model, the investigation period was restricted to limited ATIS studies with a modified morning peak period. The investigations performed indicated that ATIS benefits increased significantly with incident severity and significantly decreased with distorted information.

Key Words: Freeways, Intelligent Transportation Systems, Animation and Simulation

EXECUTIVE SUMMARY

This report highlights some of the major findings and conclusions of this three year, multi-person research effort in applying the INTEGRATION model to the Santa Monica Freeway corridor in Los Angeles. This simulation study, like most simulation studies, consisted of the following four major activities:

- Selecting simulation model and site for application,
- Assembling and coding model input data,
- Testing and calibrating the model,
- Undertaking investigations with the model.

The major findings of this study are presented under each of these major activities in the following sections, and the conclusions are presented at the end of the executive summary.

SELECTING SIMULATION MODEL AND SITE FOR APPLICATION

The research team had previously assessed the available simulation models which could be employed in investigating ATMS and ATIS strategies in a freeway corridor. The key required features of the models included origin/destination demand estimation, traffic assignment coupled with traffic simulation, and the ability to simulate ATIS and ATMS strategies in a freeway corridor. Three models, CONTRAM, INTEGRATION and SATURN, were found to be most applicable in these earlier studies. The research team gained experience in using the CONTRAM and INTEGRATION models. While recognizing that all models had some deficiencies, the INTEGRATION model was determined to be the most appropriate for this current study.

While many difficulties were encountered with applying the INTEGRATION model in this study, it is still considered to be the model most suitable for studies similar to this one. Two new model developments, CORFLO/CORSIM and DYNASMART, are currently underway and should also be considered for future studies of this nature.

The previous experience with models is covered in Chapter 2 and an overview of the INTEGRATION model is presented in Chapter 3. This documentation of the INTEGRATION model in Chapter 3, is considered to be one of the most comprehensive available today.

The Santa Monica Freeway corridor was selected as the site for this current study because of the previous experience of the research team in working in this corridor, and the cooperative arrangements which developed with the California Department of Transportation and the City of Los Angeles. While an excellent site for considering ATMS and ATIS implementations, the physical size, time duration of the study, and complexity of the freeway corridor provided significant challenges to the research team.

The freeway portion of the freeway corridor included over twenty directional *freeway* miles with great variability in geometric features. There were thirty on-ramps and thirty off-ramps along the freeway with varying merge and diverge design configurations, and varying intersection designs where the ramps connected to the arterial street system.

The arterial portion of the freeway corridor included over 200 direction arterial street miles and over 500 intersections. The intersections included yield sign controlled, stop sign controlled, and signal controlled intersections. The 3 12 signalized intersections included a wide variety of multi-lane configurations and multi-phase signal timing plans.

The study duration extended from 6:00 A.M. to 8:00 P.M., a total of fourteen hours which was divided into twenty-eight 30 minute time slices. The target was to obtain traffic counts on over 3200 links for each of the twenty-eight time slices which represented almost 100,000 link flows.

Previous simulation modeling experience in the Santa Monica Freeway corridor is described in Chapter 2. The general characteristics of the freeway and arterial street portions of the corridor are described near the beginning of Chapter 4.

ASSEMBLING AND CODING MODEL INPUT DATA

The flow chart of project activities contained in Figure 1.1 in Chapter 1 pictorially illustrates that the assembly and coding of model input data included freeway data and arterial data which in turn included in each supply, demand, and control sub-data sets. The effort to assemble and code input data required over a year's efforts with two to three members of the research team involved. Once the sub-data sets were assembled, data was coded and inputted into the INTEGRATION model. Data collection and preparation of model input is covered in considerable detail in Chapters 4 and 5.

The freeway and arterial street data were assembled separately, and because of the delay in obtaining the arterial data, work on the freeway-only portion of the corridor proceeded first. Delays in the arterial data collection stage of the project were due to two factors. First, the data collection efforts were hampered by delays in the installation of loop detectors along the arterial streets. Secondly, the Northridge earthquake occurred in the middle of the project which further delayed data collection efforts. Only through the cooperation of the California Department of Transportation and the City of Los Angeles could such a gigantic data set be assembled.

An important contribution of the research team efforts was the compilation of all available traffic count information for all portions of the Santa Monica Freeway by half-hour time periods from 6:00 A.M. to 8:00 P.M. These counts included all previous traffic counts over the past five years as well as comprehensive data sets collected by the research team.

Although causing a delay in the project schedule, the City of Los Angeles collected an excellent comprehensive traffic count data set for most of the arterial streets in the corridor using their recently operational ATSAC system. Unfortunately turning movements were only available at a few intersections. Another contribution of the research team efforts was the compilation of these traffic counts in a systematically developed set of spreadsheets.

The final step in this phase of the project was inputting the assembled supply, demand, and control data into the INTEGRATION model. The final coding of the freeway corridor included 111 origin and destination nodes, 1747 nodes, and 3286 links. The capacities were estimated for each link and appropriate speed-flow curves were also developed for each link. A plot of the final freeway corridor configuration is shown in Figure 5.11.

The time-of-day intersection (3 12) and ramp (30) traffic signal plans were coded into the INTEGRATION model. Because of the complex signalized intersections and the heavy traffic demands, a unique expanded intersection coding scheme was developed as shown in Figure 5.5.

A demand estimation process was developed as shown in Figure 5.17 which connected interactively the QUEENSOD program with the main program, INTEGRATION. In addition, the research team developed a large number of auxiliary programs which were used to enhance the input coding process, provide for the connection between QUEENSOD and INTEGRATION models, and later for checking and summarizing the model outputs.

The final coding for the INTEGRATION model included the following four data files: node descriptor, link descriptor, signal timing plans, and origin/destination demand matrices.

It should be noted that the effort required to collect and code data is directly proportional to the size and complexity of the network. A network as large and complex as the Santa Monica Freeway corridor required over a year's efforts to collect and code the model input data. Even with a data collection effort this large, problems were still encountered in the calibration efforts due to the need for an even more extensive and accurate data base of demand data (i.e. link flows).

TESTING AND CALIBRATING THE MODEL

Due to the continuous updating of the INTEGRATION model and limited number of real-world applications of the INTEGRATION model, thorough testing of the model was required. The effort completed by the research team in the testing of the INTEGRATION model is detailed in Chapter 6. The testing of the model analyzed the INTEGRATION model's ability to model both freeway and arterial links, however the freeway portion of the corridor was emphasized due to its importance in the corridor simulation.

INTEGRATION model tests with two simple freeway networks indicated that the INTEGRATION versions 1.5d and 1.5e were predicting similar traffic performance and that these predictions were similar to traffic performance estimates from a well established simulation model (FREQ) and analytical solutions. Tests with a simple arterial network indicated that INTEGRATION versions 1.5d and 1.5e were reasonably modeling arterial links.

To test the INTEGRATION model on a larger scale the freeway-only portion of the Santa Monica Freeway corridor was coded and calibrated for both INTEGRATION versions 1.5d and 1.5e. In order to judge the performance of the INTEGRATION model, the freeway-only portion of the Santa Monica Freeway corridor was accurately coded and calibrated with the FREQ model, which is a well established freeway simulation model.

With some modifications to the coding of freeway off-ramps, the INTEGRATION 1.5e modeling of the eastbound mainline freeway was fairly reasonable. However, there were still some questions about the performance of the INTEGRATION model in simulating the freeway-only portion of the corridor. The INTEGRATION model was predicting higher congestion levels on the eastbound mainline freeway during the morning peak period. Comparison of the link flows predicted by INTEGRATION 1.5e and FREQ revealed differences on the order of plus or minus 500 vehicles per hour.

The testing of the INTEGRATION model's ability to model HOV lanes determined that the INTEGRATION model is very robust in its abilities to simulate a wide range of HOV facilities.

Future simulation projects utilizing the INTEGRATION model should attempt to obtain much more detailed documentation on the INTEGRATION model and many of its subsidiary programs than was available for this research effort. Detailed documentation on the methodologies programmed into the INTEGRATION model is crucial.

Various features of the INTEGRATION model were tested and modified as a result of this research effort. Future simulation projects should also obtain detailed reports on the testing and validation of the various features of the INTEGRATION model.

While not completely satisfied with the results of the model testing effort, a decision was made to move ahead into the calibration effort with the anticipation that these remaining problems could be overcome in the calibration effort or if not, the calibration effort and the later planned investigations would be redesigned considering these limitations.

The calibration stage of the research project was one of the most extensive stages of the project, involving over one year's efforts. Due to time and fund restrictions, coupled with problems encountered in the use of the INTEGRATION model, the calibration stage of the project only attempted the calibration of fourteen of the twenty-eight time slices to be simulated. Only six of these fourteen time slices were classified as calibrated at the conclusion of the calibration effort. The calibration effort conducted by the research team is detailed in Chapter 7.

The corridor calibration was difficult because a network of the size and complexity of the Santa Monica Freeway corridor had never been successfully calibrated using the INTEGRATION model nor an equivalent model. The calibration effort focused initially on a single expanded intersection and then on calibrating the entire Santa Monica Freeway corridor. A huge effort was undertaken during the calibration of the first time slice to refine the demand estimation process that will be utilized during the calibration of all subsequent time slices. Once the problems associated with the calibration of the first time slice were resolved the calibration of the morning peak period and midday period were undertaken.

The attempts to calibrate the INTEGRATION model for the first early-morning uncongested time slice, which included work to refine the demand estimation process, required three months of effort on the part of the research team. A major accomplishment was the identification of problems in the modeling effort, and incorporating refinements in the demand estimation and traffic simulation process in an attempt to overcome or minimize these problems. The research team received guidance from the INTEGRATION model developer's during this stage of the calibration effort and both participated in the model refinements.

An incremental approach was followed during the calibration of the morning peak period (eight time slices) and midday period (ten time slices). The effort to calibrate the morning peak period resulted in a reasonable calibration of time slices one, two and three. However, after an extensive effort, both at U.C. Berkeley and Queen's University, a reasonable calibration of time slices four, five and six was not attained. The calibration of the last two time slices of the morning peak period was not attempted. It should be noted that the model developer was not under contractual obligations to the research team, but did provide guidance and recommendations throughout the calibration of the morning peak period and did take the opportunity of attempting to calibrate the morning peak period.

The calibration effort for the midday period achieved a marginally reasonable calibration for time slices nine, ten and eleven, but not for time slices twelve through sixteen. The calibration of the last two time slices of the midday period was not attempted.

Future research should further improve the demand estimation process. Improvements in the demand estimation process would lead to the generation of more accurate origin/destination matrices, which would produce a more reasonable calibration. Further work to determine the optimal number of minimum path trees and optimal generation time of those minimum path trees, and to determine the optimal arterial link flow reliability factors, should be undertaken.

The calibration effort undertaken for this research project highlights the high correlation between network size and complexity, and the calibration effort required. A more successful calibration would surely have resulted if a more comprehensive and more accurate demand data base (i.e. observed link flows) was available. Future data collection efforts should attempt to obtain traffic volumes of all turning movements at all intersections. Observed turning movements will improve the modeling of traffic performance at signalized intersections. Since unrealistic delay at many signalized intersections in the corridor was one of the main reasons why the time slices with heavy demands could not be calibrated, improved modeling of traffic at intersections will allow a more successful calibration effort.

It should be noted that the research team devoted over a years' efforts to the data collection effort. This effort did result in one of the most comprehensive and complete corridor data bases ever assembled.

UNDERTAKING INVESTIGATIONS WITH THE MODEL

The investigations conducted by this research effort primarily focused on the impact of in-vehicle information systems (IVIS) with different levels of information quality under non-incident and various incident conditions. An evaluation of the INTEGRATION model's potential ability to model ATIS and ATMS control strategies and the results of the investigations conducted is contained in Chapter 8.

Investigations of ATIS control strategies were severely limited since attempts to establish an INTEGRATION run with a network loaded entirely with unguided vehicles were unsuccessful. Thus, the base run for the ATIS investigations was with a network loaded entirely with guided vehicles. Investigations to study the impact of supplying motorists two different levels of information quality were conducted. The INTEGRATION model was also capable of simulating variations in the number of surveillance links in the network, CMS and HAR. However, due to extremely limited project time tests to validate these features of the INTEGRATION model were not conducted by the research team. Tests to validate these features could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Thus, investigations to study these untested control strategies were not considered for this research effort.

The INTEGRATION model was only capable of limited investigations of ATMS control strategies. The model was not able to simulate optimized ramp meter timing plans or optimized signal coordination along an arterial route. The optimization of individual signals required implementation in the calibration stage of the project, thus this feature was not available to be incorporated into the final design of experiment. The INTEGRATION model had proven to be robust in simulating HOV lanes, but these control strategies were classified as lower priority investigations and were not studied during this research effort.

Investigations with supply coding modifications were possible and were conducted at two different incident locations with both a minor incident and major incident. Investigations with demand coding modifications were not possible, due to problems encountered in calibrating time slices with heavy demand patterns. The investigations conducted for this stage of the project studied the impact of IVIS with different levels of information quality under both non-incident and incident situations in a network loaded entirely with guided vehicles.

The demand pattern for the morning peak period was modified to reduce the level of congestion in the network due to problems encountered in the calibration of time slices with heavy traffic demands. The modified morning peak period was an eight time slice run (including the warm-up time slice) that was created from the origin/destination matrices for the first three time slices of the morning peak period, which were all reasonably calibrated during the calibration stage of the project.

The investigations were conducted at two incident locations. The first incident location was on the eastbound mainline freeway upstream of the major bottlenecks which were predicted along the eastbound mainline fi-eeway in the base run. Six investigations were conducted at this location; two levels of information quality (perfect information and distorted information) and three incident situations (no incident and a minor and major incident) were all studied. These investigations revealed that for a minor incident, a 100 percent IVIS system can offset the adverse effect of the minor incident. However, the adverse effect of the major incident along the eastbound mainline freeway in the Santa Monica Freeway corridor, can only be partially *offset* by all vehicles in the system being guided vehicles. Under the major incident situation, travel times on almost all sub-portions of the corridor network were increased.

The investigations with the distorted information indicated that a reduction in the level of information quality provided to motorists results in significant increases in total arterial trip distances and reductions in total freeway trip distances. The overall total trip time in the corridor when vehicles were provided distorted information under the non-incident situation was almost equivalent to that when vehicles were provided perfect information under the major incident situation. Thus, in terms of total trip times the effect of distorted information was almost as adverse as a major incident with motorists receiving perfect information.

The second incident location was on the eastbound mainline freeway at a major bottleneck location. The minor incident at this location resulted in an adverse effect to vehicles on the eastbound mainline fi-eeway and ramps, and to vehicles traveling along the arterial routes. The major incident at this second location had a larger impact on eastbound vehicles and the arterial routes than the minor incident. Thus, the costs of both the minor and major incident at a bottleneck location on the eastbound mainline freeway could only be partially offset by all vehicles in the system being guided vehicles.

CONCLUDING OBSERVATIONS AND RECOMMENDATIONS

The concluding observations and recommendations from this research report summarize the observations and discuss the potential enhancements in the data collection effort and in the INTEGRATION model's performance. Some general remarks on computer simulation modeling are also presented.

The data collection effort included the assembly and coding of both freeway and arterial data. During the model calibration for both the freeway-only simulation and the freeway corridor simulation the importance of complete and accurate freeway on-ramp and off-ramp volumes and freeway-to-freeway connector volumes was realized. During the freeway calibration work the need for accurate and proper freeway performance measures, such as temporal and spatial velocity patterns and freeway end-to-end travel times, was reconfirmed. The need to collect turning movement volumes at all signalized intersections was also realized during the calibration stage of the project. During the calibration of time slices with heavy demands unrealistic traffic performance was being simulated at a number of intersections which resulted in queue spill-back problems in the arterial network. A more successful calibration would surely have resulted if all turning movement volumes at all signalized intersections were available.

Thus, it is critical for future freeway data collection efforts to collect complete and accurate freeway on-ramp and off-ramp volumes and freeway-to-freeway volumes. It is also important that freeway performance measures be collected during the freeway data collection efforts. For detailed simulation modeling, it is critical that accurate and complete signalized intersection turning movement volumes be included in the arterial data collection efforts.

The data collection effort for this research project, which collected intersection approach volumes and only a few intersection turning movement volumes, required over a year's efforts with two to three members of the research team involved to collect and code data. It should be noted that the effort required to collect and code data is directly proportional to the size and complexity of the network.

The link flow data was used to derive a synthetic origin-destination demand matrix. This research project highlighted the difficulties these models have in determining the actual origin-destination demand patterns. Future freeway corridor simulation efforts should try to obtain actual origin-destination demand patterns from driver survey studies or from other sources. Since these studies are very time consuming and expensive, the use of a travel demand model should be considered to generate a more accurate origin-destination demand matrix.

The performance of the INTEGRATION model was determined during the model testing, model calibration and investigation stages of the research project. The five potential areas for improvements in the INTEGRATION model that were identified during this research were;

- Further testing and enhancements of the opposing link feature of the INTEGRATION model are required. The initial calibration efforts revealed that detailed simulation of a signalized intersection in INTEGRATION required an eight node/twelve link configuration, which would allow for the coding of left-turning movements which are delayed by an opposing vehicle flow. The opposing link feature of INTEGRATION should be enhanced to calculate the amount of delay imposed on left-turning movements as a function of the geometry of the signalized intersection.

- The INTEGRATION model's capability to simulate ATMS control strategies needs to be expanded. The generation and simulation of optimal signal timing plans along an arterial route and optimal freeway ramp metering plans should be incorporated into the INTEGRATION model.

- An improvement is required in the process in which a base run with a network loaded entirely with unguided vehicles for ATIS investigations is established. The ability to establish an INTEGRATION simulation of a network loaded entirely with unguided vehicles, which uses the travel time information or minimum path trees generated from a run with a network loaded entirely with guided vehicles, that reasonably matches the simulation run with the network loaded entirely with guided vehicles needs to be improved.

- The output generated by the INTEGRATION model should be enhanced in order to ease the analysis of the simulation output. The user should be able to obtain speed, density and flow contour maps for a set of user-specified links.

- A reduction in the run time of the INTEGRATION model is required. Extremely slow run times of several days hampered calibration and investigation efforts for this project.

While many difficulties were encountered with applying the INTEGRATION model in this study, it is still considered to be the model most suitable for studies similar to this one.

The effort and time required to complete a detailed and accurate fourteen hour simulation of the entire Santa Monica Freeway corridor was underestimated by the research team. Over a year's efforts were spent to assemble a comprehensive input data set, which resulted in the best available single source of these data collected to date for the Santa Monica Freeway corridor. The data requirements for a more successful calibration effort were much greater than the project was able to collect. The model testing and calibration stages of the project, which also required over a year's efforts, developed and refined a systematic procedure for the calibration process and demand estimation process. These stages also identified and corrected and/or minimized some of the problems with the INTEGRATION model, with help from the model developer. Some of the improvements still required in the INTEGRATION model are identified above. Given the time, budget and talent available for this project, only a partial calibration was achieved and only limited investigations were conducted. Future simulation projects should be very conservative in their estimation of the effort required for data collection and coding, model testing and calibration, and investigations. The effort required to simulate a network is directly proportional to the size (both temporal and spatial) and complexity of the network.

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Chapter 1: INTRODUCTION

This report summarizes the results of a multi-year, multi-person research project to investigate Intelligent Transportation Systems (ITS) on the Santa Monica Freeway Corridor (I-10) in Los Angeles, Caliiornia. The initial goal of the project was to determine the potential benefits of Advanced Traveler Information Systems (ATIS) and Advanced Traffic Management Systems (ATMS) control strategies. However, due to time and fund restrictions the project scope was scaled down to just ATIS investigations. The evaluation used the INTEGRATION traffic simulation model. This project was under the direction of Professor Emeritus Adolf D. May at the University of California at Berkeley. This project began in September of 1992 and was completed by June of 1995.

1.1 BACKGROUND

The nation's network of surface streets constitute a large portion of the transportation system that provides a basic source of mobility. However, congestion problems resulting from the growth of automobile ownership and use now threaten this mobility. In 1987, it was estimated that congestion accounts for over 2 billion vehicle hours of delay per year on urban freeways. The productivity losses from congestion only were estimated to cost the nation up to $100 billion per year [15].

The magnitude of the congestion problem worldwide has resulted in numerous research efforts aimed at improving the efficiency of the existing transportation systems. In particular, increasing attention is being paid to the potential of advanced technologies to achieve improvements in the transportation system. ITS represents a technology-based approach to improving the nation's transportation system's efficiency.

ITS involve the application of advanced information processing and communications, sensing, and control (and information) technologies to surface transportation. ITS offers the transportation system significant benefits including: increased capacity and operational efficiency, improved safety, reduced environment and energy impacts, increased productivity for motor carriers and service providers, increased comfort and convenience of travel, and improved cooperation between transportation systems operators. ITS technologies can be viewed as six interlocked systems: Advanced Traffic Management Systems (ATMS), Advanced Traveler Information Systems (ATIS), Advanced Vehicle Control Systems (AVCS), Advanced Public Transportation Systems (APTS), Commercial Vehicle Operations (CVO), and Advanced Rural Transportation Systems (ARTS). ATMS includes urban traffic control systems, incident detection systems, highway and corridor control systems, High Occupancy Vehicle (HOV) priority treatment and ramp metering systems. ATIS technologies are designed to provide the traveler with navigational information and routing advice based on real-time traffic data using audio or visual media contained in the vehicle or on the highway. AVCS technologies under development include variable speed control, radar braking, and automated headway and steering control. APTS include automatic vehicle monitoring, real-time traveler information systems, advanced payment systems, and real-time transit scheduling. CVO addresses applications of advanced technologies to commercial roadway vehicles (trucks, commercial fleets, and inner-city buses). ARTS include vehicle location, emergency signaling and traveler information.

Interest and support for ITS has been increasing dramatically in the last few years in the United States as well as in Europe and Japan. However, there are a number of unanswered issues which still have to be addressed before any large scale deployment of these new technologies can take place. Specifically, the magnitude and consistency of the potential impacts of these technologies on roadway traveling conditions must be evaluated, either through operational field tests or research simulation studies. Due to the cost and complexity of operational field tests, traffic simulation models are used to aid transportation system designers identify key operational and performance issues in testing a range of ITS scenarios.

1.2 PROJECT SCOPE AND OBJECTIVES

At the onset of the project, the main purpose was to investigate and quantify the likely benefits of various ATIS and ATMS control strategies on the Santa Monica Freeway Corridor in Los Angeles, California. However, due to difficulties encountered in the calibration stage of the project and with the simulation model, coupled with time and fund restrictions, the project scope was reduced to investigating and quantifying the likely benefits of ATIS. The ATIS investigations studied the impact of in-vehicle information systems with different levels of information quality under non-incident and various incident situations in a network loaded entirely with guided vehicles. The impact of ATIS was assessed in terms of trip distances, travel time savings and average speeds.

In order to evaluate and refine various ITS control strategies on the Santa Monica freeway corridor, a modeling approach was selected. Through the use of simulation, it is possible to investigate many potential scenarios under many different circumstances; and many statistics on the behavior of the system can be obtained. The INTEGRATION traffic simulation model, developed at Queen's University in Kingston, Canada is being used for this application. The INTEGRATION model has several characteristics that make it a powerful and rather unique tool for network analysis in the ITS context.

1.3 APPROACH

This project is a continuation of the original work by May et al. [1]. It was determined earlier that a dynamic model combining traffic simulation and traffic assignment in an integrated freeway/arterial environment was desirable for evaluating the potential benefits of ATIS and ATMS on the Santa Monica freeway corridor. Thus, a study was begun to investigate the candidate models potentially suited for the purposes of this project [17]. The CONTRAM model was initially judged to be the appropriate model for this type of analysis. An initial application of the CONTRAM model was carried out and reported in 1991 [18]. Problems were discovered regarding the ability of the CONTRAM model to accurately reflect freeway congestion.

A new phase of the project (referred to as Phase 1) started in the fall of 1991 which employed the INTEGRATION traffic simulation model. INTEGRATION was developed specifically to deal with representations of networks combining freeways and arterials. The INTEGRATION model was applied to a number of different hypothetical networks in order to test the validity of the model before investigations on the entire Santa Monica freeway corridor were conducted. A research report [20] was prepared to summarize the various experiments conducted to validate the model capabilities with regard to freeway modeling and ATIS modeling. These initial tests on hypothetical networks indicated that the INTEGRATION model was a good model to respond to the needs of a freeway corridor and exhibited a number of features that makes it well suited for modeling ATIS.

Phase 1 of the project also conducted an initial application of the INTEGRATION model to a subsection of the Santa Monica freeway corridor, leading to the development of a reference base run. This reference base assignment (without incidents and without ATIS and ATMS) was considered as the baseline to compare with the performance of the system under various ATIS and ATMS control strategies. The study area coded and calibrated in this phase of the project involved approximately six miles of the Santa Monica Freeway (I- 10) with associated ramps, two parallel arterials: Washington Boulevard and Adams Boulevard (both eastbound and westbound), and a network of other surface streets. In addition to the PATH report [20], further discussion of investigations performed in Phase 1 of the project can be found in two papers [19], [21].

The next phase of the study (referred to as Phase 2) consisted, of initial experiments of ATIS and ATMS modeling on the portion of the Santa Monica freeway corridor that was coded and calibrated in Phase 1 of the project. A research report [22] was prepared to summarize the investigations conducted in Phase 2 of the project. These investigations were for the morning peak period (6:00 AM to 10:00 AM) under both incident-free and incident traffic conditions. Strategies tested at this stage included freeway ramp metering, traffic signal optimization, route guidance systems, and combinations of these strategies. The primary focus at this phase of the project was more on developing and testing methodologies for modeling various ITS strategies than on the quantitative simulation results. This phase of the report concluded that while INTEGRATION had some limitations it appeared to be significantly better for simulating ITS strategies than any other known simulation models.

The present report describes the next phase of the study consisting of ITS modeling on a larger portion of the Santa Monica freeway corridor. The area of study was expanded to consist of approximately 10 miles of freeway with associated ramps, five parallel arterials: Olympic Boulevard, Pico Boulevard, Venice Boulevard, Washington Boulevard and Adams Boulevard (both eastbound and westbound), and a network of other surface streets. The time frame for this phase of the project was expanded from the morning peak period (6:00 AM to 10:00 AM) to a full day of operation (6:00 AM to 8:00 PM). This expanded portion of the Santa Monica freeway corridor to be modeled is the largest freeway/arterial network simulation ever attempted. The calibration stage of the project attempted to match the simulated and real-world traffic performance, on both the freeway and arterial portions of the corridor network, as accurately as possible. Investigations conducted with this expanded network primarily considered ATIS control strategies under non-incident and various incident situations for a modified morning peak period (6:00 AM to 9:30 AM).

The stages of the present phase of the project are outlined in Figure 1.1. The goals and objectives of the project were initially to investigate and quantify the likely benefits of various ATIS and ATMS control strategies, but were later scaled back to just consider ATIS control strategies. To accurately quantify these benefits the study area coded for Phases 1 and 2 of the project and the time frame of the simulation had to be expanded. This expansion was required in order to realistically model vehicle routing behavior throughout the Santa Monica freeway corridor and throughout the entire day. This expansion required the collection of supply, demand and control data for the entire corridor. The freeway and arterial data were collected in two separate efforts, since Caltrans collected the freeway data and the City of Los Angeles collected the arterial data. The freeway data was collected in three facets: supply (geometry, capacity and speed/flow relationship of all freeway and ramp links), demand (observed freeway and ramp link flows) and control (freeway ramp metering plans). The arterial data was also collected in three facets: supply (arterial network designation, and geometry, capacity and speed-flow relationship of all arterial links), demand (observed arterial link flows) and control (signal timing plans, stop sign locations and turning restrictions). The freeway data collected was manipulated and then summarized into three spreadsheets (freeway supply, demand and control data). The arterial data collected was also manipulated and summarized into three spreadsheets (arterial supply, demand and control data). The data in the freeway and arterial spreadsheets represent one of the most complete and accurate data sets available for the Santa Monica freeway corridor.

The next stage of the project was the coding of the freeway and arterial data into input files for the INTEGRATION model. The freeway and arterial supply spreadsheets were combined and manipulated to create the INTEGRATION node and link files. The freeway and arterial demand data were combined to create a list of observed link flows on both the arterial and freeway portion of the corridor. The observed link flows were used in a synthetic origin/destination (O/D) matrix estimation model called QUEENSOD to derive O/D tables for each 30 minute time slice. The arterial control data was coded into the signal timing input file for INTEGRATION. The freeway control data (ramp metering plans) were then coded into the signal timing input file. The result of this stage of the project was a set of input tiles for the INTEGRATION model containing all the supply, demand and control data necessary to simulate a full day of operation on the Santa Monica freeway corridor. The collection and INTEGRATION input coding effort for the freeway only portion of the network was detailed in a PATH report [9].

The calibration of the INTEGRATION model was the next stage of this project. Due to the complexity of the calibration procedure, the simulation was separated into three portions: a morning peak period (6:00 AM to 10:00 AM), midday period (10:00 AM to 3 :00 PM) and evening peak period (3 :00 PM to 8:00 PM). The calibration effort focused

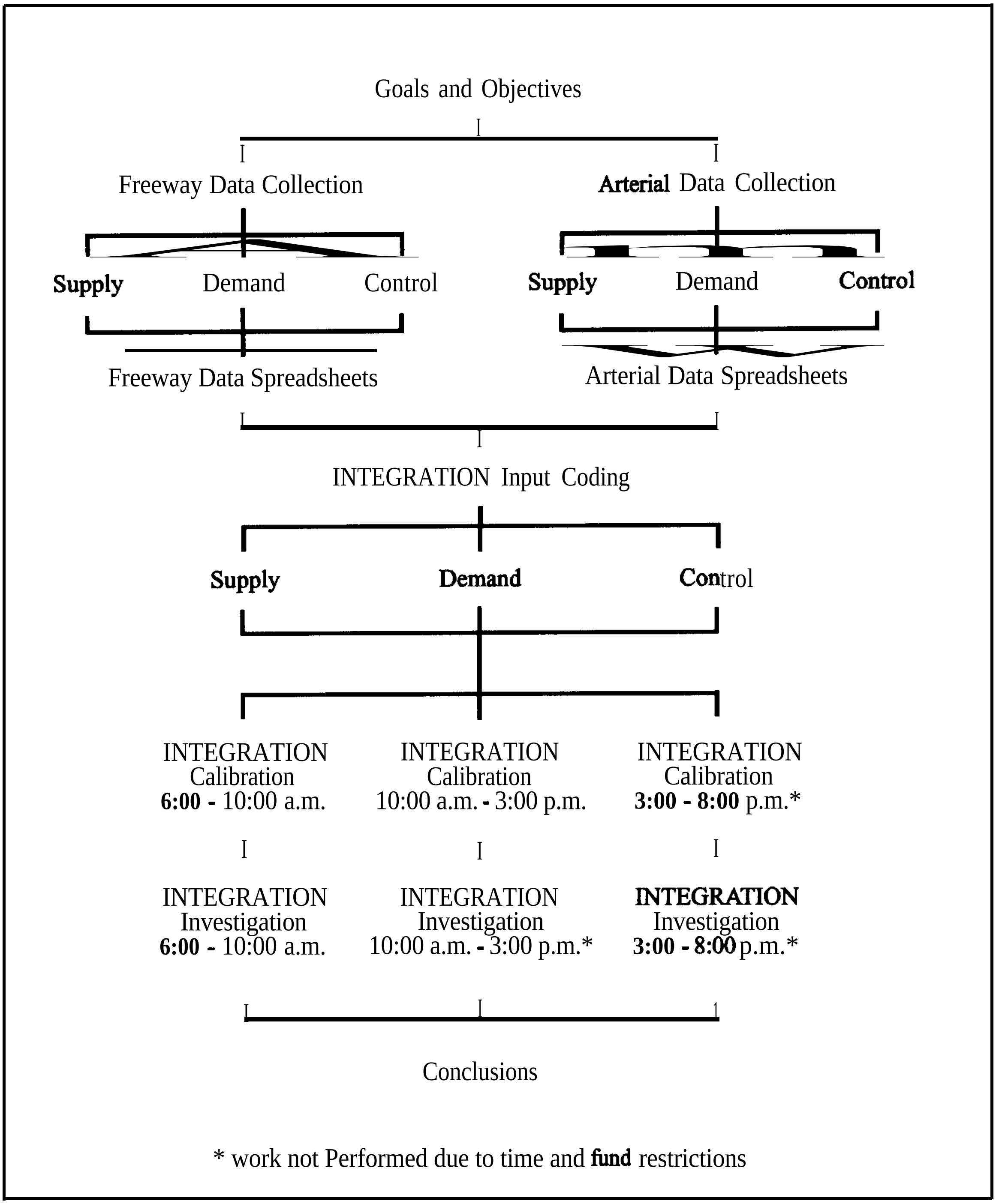

Figure 1.1: Project Outline

on adjusting the supply, demand and control input data to match the INTEGRATION simulated link flows to the observed link flows for each time slice. Care was taken to adjust only those input data values within ranges of acceptable values. The calibration of the freeway portion of the network was emphasized during the calibration stage due to the importance of correct freeway performance. However, accurate matching of simulated and real-world traffic performance on both the freeway and arterial portions of the freeway corridor was stressed throughout the entire calibration stage of the project. Due to time and fund restrictions, the calibration of the evening peak period was not attempted.

The next stage of the project was the INTEGRATION investigations. The investigations were to be conducted within each of the three time periods used for the calibration effort (morning peak period, midday period and evening peak period). However, the investigations were limited to the morning peak period due to time and fund restrictions. The demand pattern used for these morning peak period investigations was modified to reduce the level of congestion in the network. The level of congestion in the network had to be reduced from the observed levels due to problems encountered in calibrating time slices with heavy traffic demands. The morning peak period investigations primarily focused on the impact of in-vehicle information systems with different levels of information quality, under non-incident and various incident conditions, in a network loaded entirely with guided vehicles. The potential benefits of various ATMS strategies were not analyzed in the morning peak period due to model limitations, and time and fund restrictions.

The final stage of the project was the conclusions of this study. This phase summarizes INTEGRATION's modeling performance and the impact of in-vehicle information systems with different levels of information quality during non-incident and several incident situations in the modified morning peak period. This phase concludes with some recommendations for ATIS implementation and a discussion of potential areas of future research.

It should be noted that the INTEGRATION model was continually improved and enhanced during the course of this study. Before the calibration of the Santa Monica Freeway corridor was undertaken, the version of the model to be employed for calibration and investigations was tested to determine any remaining model problems. These tests focused on INTEGRATION's ability to model both freeway and arterial performance. During the calibration and investigation stages further problems were found in the INTEGRATION model that required fixing by the developer. Determining the source of the problems and verifying the corrections through model testing and re-testing, required significant project time and resources. During the final investigations, further uncorrected modal deficiencies were encountered which significantly reduced the scope and content of the final investigations.

1.4 ORGANIZATION OF THE REPORT

Chapter 2 of this report outlines the history and background of this study. Work conducted worldwide and within the California PATH Program is summarized. Chapter 3 presents an overview of the INTEGRATION traffic simulation model. Chapter 4 outlines the data collection efforts for the simulation, for both the freeway and arterial portions of the Santa Monica Freeway Corridor. Chapter 5 considers how those data were coded into the INTEGRATION model. Chapter 6 presents the various tests that were conducted on the

INTEGRATION model as part of this research effort. Chapter 7 describes the calibration process, and Chapter 8 outlines the results of the ATIS investigations. Finally, Chapter 9 provides some conclusions, and the references complete the first volume of this report. The second volume of this report contains the technical appendices.

Chapter 2: PREVIOUS RESEARCH

This chapter discusses previous research efforts that have attempted to evaluate the potential benefits of Advanced Traveler Information Systems (ATIS) and Advanced Traffic Management Systems (ATMS). Due to the vast amount of research conducted in this area over the past decade, this chapter only presents a sampling of this research. The first section of the chapter lists research that has been performed in Japan, Europe and North America. The second section of the chapter discusses previous Intelligent Transportation Systems (ITS) research conducted at UC-Berkeley. Finally, the chapter summarizes the previous research and presents the transition from the previous research to the current research effort.

2.1 PREVIOUS ATIS EVALUATION STUDIES

2.1.1 Japan

Some of the earliest reported research in this area was done by Kobayashi [32] in 1979. This research involved a simulation model to assess the potential benefits of the Japanese CACS route guidance system. The model compared the shortest routes for guided drivers with routes for non-guided drivers. It was estimated that the total network travel time could be reduced by 6% at a level of market penetration[1] of 50 to 75%.

Tsuji [54, 55] in 1985 developed mathematical models for estimating the effectiveness of a route guidance system. The models were justified by comparison from actual data collected from the CACS system. The models predicted that the travel time savings for guided vehicles would be approximately 1 1%, compared to the 12% savings observed from the CACS system. One drawback to the models developed was that the percentage of guided vehicles was assumed to be so small that there would be no influence on non-guided vehicles. That is, diversion due to route guidance was assumed to be small enough that it did not affect travel times in the network.

Uno [56] et al. in 1994 created a model for evaluating the benefits of real-time traffic information that consists of three submodules; one for dynamic traffic flow, one for route choice and one to assign traffic information based on traffic conditions. The route choice submodule is somewhat unique in that in analyzing whether a vehicle will divert or not, it allows for the consideration of factors such as scenery and safety, as opposed to solely travel time.

2.1.2 Europe

The LISB[2] system in West Berlin is a route guidance system where a number of in-vehicle units transmit traffic information to a central computer. The central computer processes all of the information in the network and provides guidance to all of the equipped vehicles

[1] Market penetration refers to the percentage of vehicles that are equipped with route guidance capabilities.
[2] LISB is the Berlin Route Guidance and Information System

in the network. Initial analysis of the LISB system was done by Sparmann [49] in 199 1. The study concluded that the potential safety and travel time improvements were significant but offered no specific values of travel time savings. May et al. [37] developed a novel method of analyzing the LISB system that consisted of comparing travel times for guided and non-guided vehicles with the same origin and destination, and traveling at the same time. The analysis with this method found that travel time savings from route guidance were almost negligible. They surmise that the benefits of familiarity with the network may override those from the LISB system.

In 1989, JMP Consultants [30] conducted an analysis of the AUTOGUIDE system in London that is functionally similar to the LISB system. For a sample of 100 trips made, the researchers compared actual paths to those of viable alternative routes and determined that a travel time savings of 8 to 11% was possible.

Breheret et al. [10] used the CONTRAM traffic assignment model which routes unguided drivers based on an approximate stochastic user equilibrium. The CONTRAM model routes guided drivers using their optimum routes on the basis of current traffic conditions. The findings of the research were that during incident conditions, the travel time savings could be on the order of 15% when the guided vehicles were given numerous alternate routes. When only a single alternate route was provided to guided drivers, the total network travel times increased indicating that the provision of a single alternate route may be problematic. Also, most of the travel time savings occurred with less than 10% guided vehicles.

Smith and Russam [46] also used the CONTRAM model to evaluate the AUTOGUIDE system. They reported that a travel time savings of 7% for guided vehicles was possible. Non-guided vehicles would also benefit resulting in a total network travel time savings of 3-6%. Smith and Ghali [47] used a modified version of CONTRAM to evaluate the effect of four traffic responsive control strategies in combination with two route guidance strategies. They found that by combining a local system optimal route guidance strategy with some of the familiar traffic control strategies, a total network travel time reduction of at least 20% could be obtained.

Van Vuren et al. [63] in 1991 used the SATURN traffic simulation model to simulate a network with multiple user classes, one class of non-guided vehicles and three classes of guided vehicles. Simulation runs were made with three different demand levels and nine levels of market penetration for an actual network near Leeds, England. The results of the study were that in simulating user equilibrium routing, the travel time savings were on the order of 5% for guided vehicles and only 1% for non-guided vehicles. The authors stressed the need for further model development particularly the development of more accurate dynamic route guidance.

Watling [64] provides a recent (1994) overview of the present state of simulation models to evaluate real time information systems. He classifies the available models into three groups. The first group is pure assignment models such as CONTRAM, JAM, and

ASSIGN. A second category is microscopic simulation models with the ability to simulate items such as lane changing and car-following. NETSIM and SIMNET are examples of this second category. Finally, there are models that consist of an assignment submodule and a traffic simulation submodule such as INTEGRATION and SATURN. He notes that of all the models reviewed, only INTEGRATION, CONTRAM, and SATURN allow for the re-optimization of route selection due to significant diversion of guided vehicles. Among his recommendations for future research are simulation of driver behavior and shifts in demand over time.

2.1.3 South Africa

In 1986, Brown et al. [11] undertook a study to determine the potential benefits of improvements to the SIGNPOST program in Johannesburg that consists of numerous changeable message signs. The methodology consisted of developing a suite of computer programs that determine the minimum cost path trees, route vehicles accordingly, and determine the travel costs for all of the vehicles. The study concluded that improvements to the SIGNPOST system would reduce total travel costs by 1.63%, with one third of the savings in the form of reduced travel time.

2.1.4 North America

In 1991, Reiss et al. [41] discussed the use of the SCOT simulation model to assess the performance of a freeway corridor control system, which includes signal timing optimization using TRANSYT, ramp metering and route guidance. A freeway corridor in Long Island, New York was simulated under nine scenarios with varying levels of demand and incidents. The results showed significant travel time savings under incident scenarios but almost no savings in a non-incident scenario, implying that drivers already use their best routes under recurrent congestion.

Mahmassani and Jayakrishnan [34] of the University of California at Irvine created a simulation and assignment framework that consist of a traffic simulator and a user decisions component. Simulation runs were conducted using a CRAY computer on a hypothetical network of three parallel freeways that service a central business district (CBD). Sensitivity analyses were done with market penetration and a factor to indicate a user's willingness to change routes. The results showed a maximum travel time savings of 5% with almost all of the benefits occurring with only 50% market penetration. However, the authors cautioned that the results were only a starting point for further study. The study is interesting as it emphasized the need to consider the user's willingness to divert. Attempts were also made to expand the dynamic model to explicitly model the driver's information acquisition process.

In 1991, Kaysi et al. [31] proposed a framework that involved three components; a surveillance system, a control and routing module, and a congestion prediction module, the last being the novel suggestion. Also of interest is the recommended use of time-varying demand in response to incidents.

The Federal Highway Administration (FHWA) developed the CORFLO model [27] to simulate construction scenarios for the I-405 freeway in Renton, Washington. The model attempted to examine the effect of numerous lane reductions that were expected on the corridor resulting from the construction of additional freeway lanes. However, the model could not be adequately calibrated to represent existing traffic flow. Use of the model was not recommended due to the extensive data requirements of the mainframe version of the program. According to conversations with professionals in the Los Angeles area, the CORFLO model was also used to simulate the Santa Monica Freeway corridor without success.

Researchers at the California Polytechnic State University have developed the METS (Mesoscopic Event-driven Traffic Simulation) model [50, 51] to simulate a wide range of routing behavior including “smart” and “obedient” vehicles. In selecting a model for the research, the INTEGRATION model was listed as the best existing model but was rejected because of the proprietary status of the program. The mesoscopic nature of the METS program means that it is a hybrid of a true microscopic model and a macroscopic model. The model simulates individual vehicles but calculates vehicle performance based on macroscopic flow relationships. The model is still in the development stage.

Chang [13,14] discussed the development of a simulation network that uses a parallel computing architecture that allowed for increased computing speed. The model consists of macroscopic and microscopic submodules that perform tasks independently of one another. The model is still in the experimental stage. The authors argue that the increased computing speed may allow the model to simulate the provision of real-time information in real-time. It appears that the computer needs of the framework are so intensive that they may significantly restrict its usage.

The DYNASMART model [28, 29], originally developed at the University of Texas at Austin, was used to assess the benefits of ATIS and ATMS strategies during special events. The network simulated was near the Anaheim Stadium in the Los Angeles Area. DYNASMART is a macroscopic model in determining shock-wave propagation and traffic flow dynamics but also keeps track of individual vehicles. The model is also capable of simulating capacity-reducing incidents as well as different levels of vehicles with route guidance capabilities.

2.2 PREVIOUS PATH RESEARCH

Research concerning the benefits of route guidance began at the University of California’s PATH Program in the late 1980’s. The present research is a continuation of these research efforts.

2.2.1 Initial ATIS Studies

In 1989, Al-Deek and May [1, 2] estimated the potential benefits of route guidance on the Santa Monica Freeway corridor in Los Angeles using three separate simulation models; FREQ, TRANSYT and PATHNET. FREQ simulated the freeway portion of the corridor, TRANSYT the arterials, and PATHNET determined the shortest paths given the travel times in the network. The results from this study were that potential travel time savings were marginal in non-incident situations with normal demand. The savings increased with increased demand and/or with the presence of incidents to a high of 4 to 14 minutes on a 30 minute trip with an incident scenario and higher demand.

The researchers noted that there were a number of assumptions that were made in this study that need to be relaxed before a more comprehensive analysis of the potential benefits of route guidance can be made. For example, the study assumed that the number of vehicles diverting from the freeway to the arterial in the event of an incident was considered to be negligible. This assumption is valid only when market penetration is quite small. As the percentage of guided vehicles increases, the network equilibrium would need to be determined. Also, a number of experienced drivers may divert even without the benefit of advanced route guidance. The amount of the total diversion to the arterial streets and the resulting traffic conditions needed to be determined. Also, the minimum path trees were calculated every 15 minutes. This may be too coarse a time period given the dynamic nature of the minimum path trees during an incident scenario.

2.2.2 Investigations Using CONTRAM

The use of multiple simulation models in the Al-Deek study proved time consuming and problematic, leading to the search for a single model that could perform the entire analysis. In 1991, Gardes, Haldors and May [18] began such a search by analyzing a total of 24 simulation models with respect to their ability to simulate ATIS and ATMS. The initial list of 24 consisted of seven transportation models, six freeway operations models, five signalized network models, and six freeway/arterial corridor models. The initial screening considered three major factors; the operating environment (freeway, arterial streets), traffic assignment capabilities, and the ability to represent congested traffic conditions. This initial screening reduced the list from 24 to only 5 models. Of these five; CONTRAM, JAM, and SATURN are signalized network models while CORQ and INTEGRATION are freeway/arterial corridor models. The CORQ and JAM model were found to be unavailable for use and hence not considered for detailed analysis.

A detailed analysis of the three models; CONTRAM, SATURN, and INTEGRATION, consisted of a scoring system where each model was given a score based on a number of model attributes that would be critical to an accurate simulation of route guidance. The CONTRAM and SATURN models were strong in that they performed dynamic traffic assignment, but weak in their ability to simulate freeways and queues resulting from traffic congestion. INTEGRATION, on the other hand, was developed specifically to perform traffic assignment in an integrated freeway/arterial network. The main weaknesses of the INTEGRATION program was its early stage of development and a dearth of previous investigations using the model. The originality and complexity of INTEGRATION's approach was also a concern. None of the models were found to be complete regarding the simulation of route guidance. The conclusion of the research was that the three models all should be tested on a small network to assess their potential ability to simulate route guidance.

Because of model availability, CONTRAM was chosen as the first model to investigate. In 1991, Gardes, Haldors, and May [18] modeled a portion of the Santa Monica Freeway corridor with CONTRAM. The network consisted of the mainline freeway, two parallel arterials; Washington and Adams, and nine crossing arterials. The boundaries of the network were Fairfax on the west and the Harbor Freeway (SR-110) on the east. Once a calibrated base run was created, simulation runs with two types of incidents and many levels of market penetration were conducted.

The results from the investigation conformed with what would be expected in the field, travel time savings were greatest for guided vehicles with the system travel times decreasing as the percentage of guided vehicles rises. However, the researchers note that the results should be viewed with caution due to a number of problems with the CONTRAM model. The main problem with the model was its inability to accurately represent congested freeway conditions. Comparison of CONTRAM with manual calculations, and with the FREQ model, showed that the CONTRAM model was predicting radically different speeds and queuing patterns. For future research the authors recommended resolving this problem and also investigating the INTEGRATION model which had recently became available and had undergone significant development.

2.2.3 Initial Testing of INTEGRATION - Freeway Straight Pipe Network

In January of 1992, the ITS/PATH research team switched to the INTEGRATION model. Due to the untested nature of the INTEGRATION model it was necessary to perform a number of investigations to ensure that the model could accurately simulate traffic performance and route guidance. Gardes and May [19, 20] performed a number of tests with the INTEGRATION model prior to using the model to simulate the Santa Monica Freeway corridor.

The first test of INTEGRATION consisted of simulating a simple straight-pipe section of freeway containing a bottleneck where the roadway capacity dropped from 6,000 vehicles per hour to 4,000 vehicles per hour. This network is shown in Figure 2.1. Demand was such that the capacity of the bottleneck was exceeded for a 10 minute period which resulted in the formation of a queue upstream of the bottleneck. Simulation runs were made on this network with both the INTEGRATION and the FREQ models and the results of these runs were compared to the results from an analytical solution. The FREQ model was used because it is widely accepted as a model that is capable of simulating traffic performance on a freeway under non-congested and congested conditions. Table 2.1 shows the results of this comparison.

The fit between the FREQ model, INTEGRATION model and the analytical solution were all reasonably close. The slight discrepancy between the FREQ and INTEGRATION models was hypothesized to result from the fact that in FREQ congestion can only begin and end at boundaries between time slices. A comparison of FREQ and INTEGRATION was also made using the travel times, the volume/capacity ratio, and the density of all the links in the network for each time slice. Again, the results between the two models were considered to be relatively close.

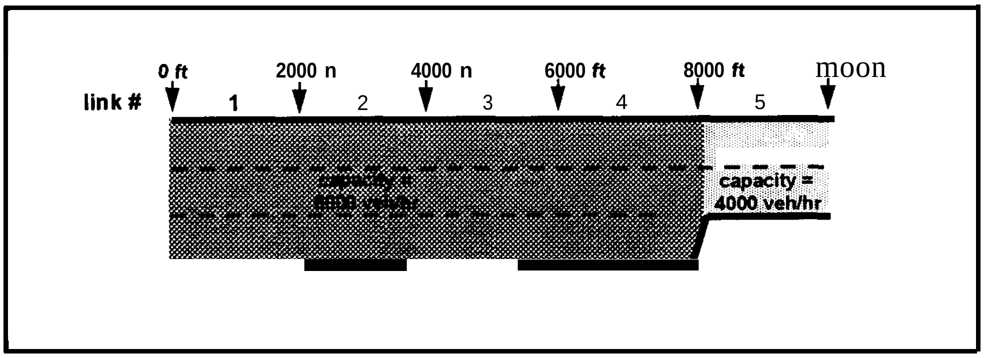

Figure 2.1: Freeway Straight-Pipe Bottleneck Network

Model	Queue Start Time	Maximum Queue Time	Maximum Queue Length	Queue End Time
Analytical	8:22	8:31	0.87 Miles	8:42
INTEGRATION	8:22	8:32	0.90 Miles	8:44
FREO	8:20	8:30	1.10 Miles	8:44

Table 2.1: Freeway Straight-Pipe Bottleneck - Summary of Modeling Results

The next test of the INTEGRATION model involved the straight-pipe network with two on-ramps added. This network is shown in Figure 2.2. Again, the demand was such that the capacity of the bottleneck was exceeded, resulting in a queue upstream of the bottleneck. Similar comparisons were made and the results showed that the models predicted the same traffic conditions with a reasonable degree of error.

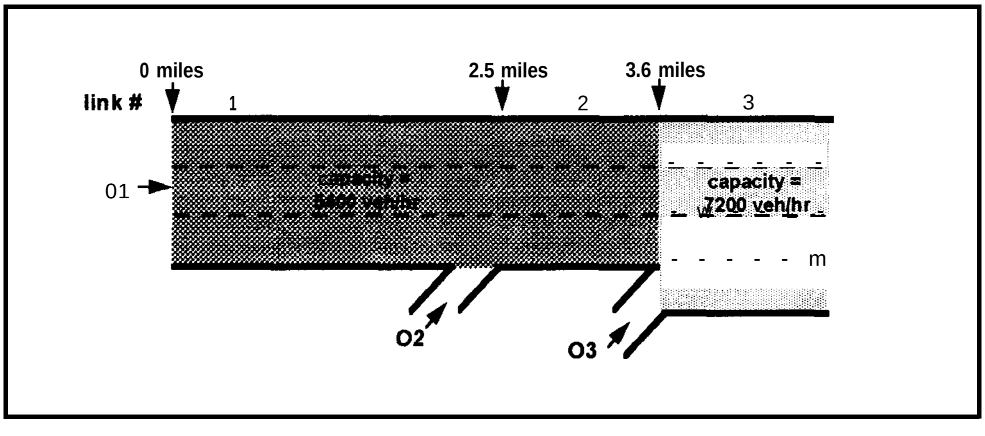

Figure 2.2: Freeway Straight-Pipe Bottleneck with Two On-Ramps

2.2.4 Initial Testing of INTEGRATION - Santa Monica Freeway

A network representing approximately nine miles of the eastbound portion of the Santa Monica Freeway was coded using the FREQ and INTEGRATION models [20]. The network encompassed 32 subsections of mainline freeway with 16 on-ramps and 15 off-ramps. The demand for the network represented eight 30 minute time slices from 6:00 am to 10:00 am.

Output from the two models including, travel times, observed speeds, and congestion patterns, were compared and found to be relatively similar. However, it was noticed the travel times predicted by the INTEGRATION model tended to be higher during periods of congestion in comparison with the FREQ model. The researchers hypothesized that the reason for this may be due to the differences in the ways in which the two models dissipate queues.

2.2.5 Initial Testing of INTEGRATION - Diamond Network

To test the route guidance features of the INTEGRATION model, a simple network named the diamond network was developed [20]. This network is shown in Figure 2.3. Traffic in the network moves from left to right. The simplicity of the network allows the analyst to observe INTEGRATION's vehicle routing without the complications of a larger network.

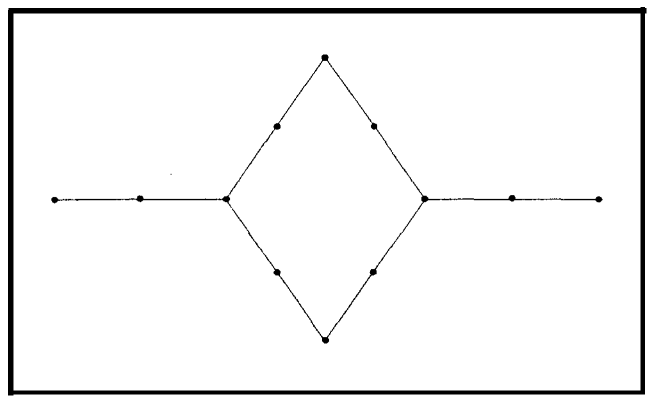

Figure 2.3: The Diamond Network

The network represents a situation where, under normal conditions, there are two identical paths, the upper and lower portions of the diamond, that can be taken to traverse the network. The supply characteristics of the upper and lower portions are identical. Thus, with no incidents in the network, vehicles will be distributed equally among the two paths. An incident was introduced into the lower path of the network resulting in demand exceeding capacity. A number of scenarios were simulated with variations made to the severity of the incident, the quality of the information provided to guided vehicles, and the percentage of guided vehicles in the network.

The first sensitivity analysis varied the percentage of guided vehicles in the network from 0 to 100% in 5% intervals. As expected, the system travel times decreased rapidly as the percentage of guided vehicles was increased from 0% to 20% with little increase in benefits beyond 20% market penetration. However, as the percentage of guided vehicles increased beyond 60% the total travel time actually increased beyond the time for 0% guided vehicles. The source of this aberration was found to be that, with heavy market penetration, the routing for the guided vehicles would oscillate between the upper and lower paths of the diamond network, resulting in significant congestion. It was found that this problem could be mitigated by decreasing the accuracy of the information given to guided vehicles.

Overall, the model was found to be quite useful in simulating the provision of route guidance. Based on the success of this test and the testing with the two hypothetical freeway networks, the model was considered to be a potential tool in the simulation of route guidance on a freeway corridor.

2.2.6 Initial Santa Monica Freeway Corridor Investigations

The next step in the investigations process was to code a portion of the Santa Monica Freeway corridor. The network was similar to the network modeled by the CONTRAM model discussed in section 2.2.2. It consisted of eight miles of bi-directional mainline freeway, two parallel arterials and nine major crossing arterials. There are 308 uni-directional links in the network and 171 nodes. 30 of the 171 nodes are zones, i.e. origins and/or destinations. This network is shown in Figure 2.4.

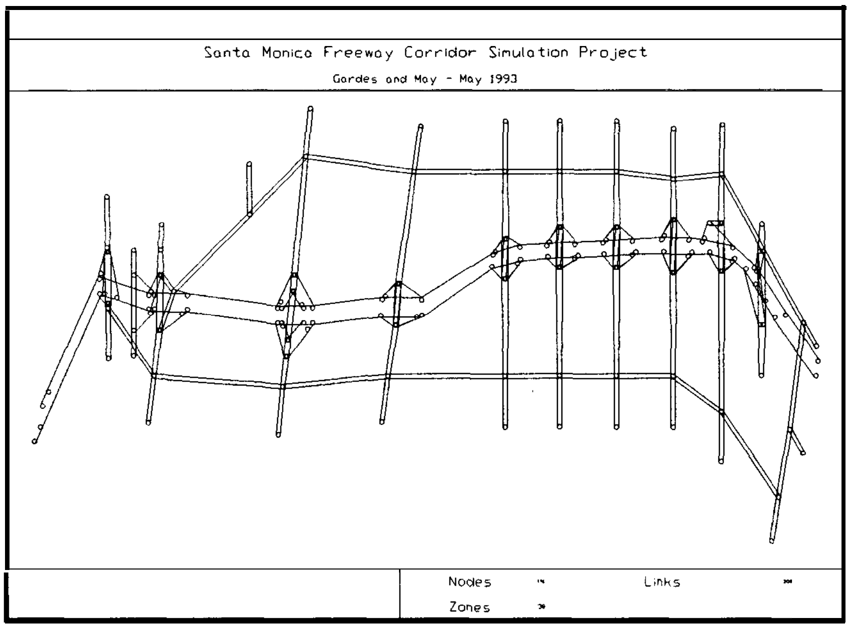

Figure 2.4: Santa Monica Freeway Network Coded by Gardes and May

Phase 1 of Gardes and May's research concluded with the calibration of a base run with this network. Phase 2 of the research [22] consisted of experiments using ATIS and ATMS strategies. Six scenarios were simulated that consisted of a base run, and five other scenarios that modeled ramp metering, signal optimization, route guidance and combinations of these. All six scenarios were run under incident and non-incident conditions. The results for non-incident conditions showed that ramp metering and signal optimizations modeled by themselves resulted in travel time savings of less than two percent each.

The presence of 25% guided vehicles resulted in an average travel time savings of 5.6% for all vehicles in the network. A sensitivity analysis varying the percentage of guided vehicles in the network revealed that more than half of the system benefits resulting from the presence of guided vehicles was possible with the first 5% of guided vehicles. The presence of 5% guided vehicles resulted in an average travel time savings of 3.8%, where guided vehicles saved 5.7% while unguided vehicles saved 3.7%. Increasing the percentage of guided vehicles to 100% resulted in a travel time savings of 6.2% for the system as a whole.

The simulation runs under incident conditions generally resulted in slightly larger travel time savings for all ATIS and ATMS strategies. Under incident conditions, travel time savings increased to 3.0% when only modeling ramp metering, and 5.6% when only modeling signal optimization. Interestingly, the travel time savings from modeling route guidance did not increase appreciably from non-incident to incident scenarios.

2.2.7 Expansion of the Network

Gardes and May concluded that the INTEGRATION model was a powerful tool in the analysis of the potential benefits of numerous ATIS and ATMS strategies. However, to more fully analyze these strategies on the Santa Monica Freeway a larger network would need to be simulated. The INTEGRATION model was also advantageous because of its ability to handle a network of much greater size than the one coded by Gardes and May.

In September of 1992, the present research project was begun. The goal of the project was to continue the simulation of the Santa Monica Freeway corridor using the INTEGRATION model. The network to be simulated was to be significantly expanded. It was felt that a more accurate representation of the network would yield more credible results as to what the potential benefits of route guidance and other traffic management strategies would be in a real freeway corridor. It is towards this goal that the present research is directed.

2.3 SUMMARY OF PREVIOUS RESEARCH

The above discussion indicates that there is not at present a single model that is a proven tool regarding its ability to simulate all aspects of ATIS and ATMS. The requirements of such a model are difficult to meet as a proper simulation of ATIS and ATMS requires microscopic simulation of individual vehicles so that traffic performance can be determined accurately at the level of individual links. This not only requires massive computing requirements for large networks, but a number of algorithms to assess items such as lane changing, car-following, and complex traffic signals. The questions of changes in demand due to various ATMS and ATIS strategies, and human behavior in response to real-time information only complicate the matter further.

The INTEGRATION model offers much promise as a model that can simulate many aspects of ATIS and ATMS. Many of the simulation models discussed in this chapter adequately simulate certain items required for ATIS and ATMS research, but fail in some other item critical to a thorough analysis. INTEGRATION appeared to have the ability to simulate a wide range of ATIS and ATMS strategies alone and in combination with one

Chapter 3: OVERVIEW OF THE INTEGRATION MODEL

The INTEGRATION model was developed in response to the need to simulate entire traffic networks; arterial streets combined with freeways, under non-congested as well as congested traffic conditions [57]. For his doctoral dissertation in 1985, Michel Van Aerde proposed a modeling approach that considers the behavior of individual vehicles which have self-assignment capabilities. This research led to the development of the INTEGRATION model, a microscopic computer simulation model, at the University of Waterloo in Ontario, Canada. Continued development of the model is being performed at Queen's University in Ontario, Canada. The model development has been sponsored, in part by the Ontario Ministry of Transportation, General Motors Research Labs, Queen's University, and the Natural Sciences and Engineering Research Council of Canada.

The purpose of this chapter is to provide a general description of the INTEGRATION modeling approach, the model input data requirements, the model outputs, an overview of the supporting models used with INTEGRATION, INTEGRATION's modeling capabilities for ITS investigations, and applications of the model.

3.1 GENERAL MODEL CHARACTERISTICS

While the INTEGRATION model has evolved substantially over the past few years, much of the base functionality of the model has remained the same. Several characteristics of the INTEGRATION model make it a powerful and rather unique tool for network analysis of various ITS infrastructure. Some of the fundamental features of the model are discussed in this section.

3.1.1 Individual Vehicles

The INTEGRATION model is a microscopic model in that it simulates the behavior of individual vehicles. The model allows for vehicles to enter the network at any given second of simulation time as opposed to only at the beginning of a time slice. New vehicles enter a network that is already loaded with previously departed vehicles that have not yet reached their destination.

These individual vehicles also have self-assignment capabilities. At each node the vehicle will decide which link to take next based on the current routing tree. The routing trees may change throughout the simulation and may vary with vehicle type resulting in multiple paths for vehicles with the same origin/destination pair.

However, the use of individual vehicles does not mean that the user has to enter input data or analyze output data at the level of individual vehicles. For a given period of time, the model reads a demand for a given origin/destination pair and generates the appropriate number of vehicles for this origin/destination pair. These vehicles are then loaded onto the network throughout the entire time period. The user can specify the degree of randomness of the vehicle departures from an origin in the time period. The output data is also aggregated into various traffic performance statistics based on a user-specified interval of time.

3.1.2 Modeling Freeways and Signalized Arterials

One of the unique aspects of the INTEGRATION model is its ability to simulate a network composed of signalized arterial links as well as mainline freeway links. INTEGRATION models the behavior of all links as pipes which have a certain capacity, speed-flow curve, length, etc. From this input data, storage capacity on the link, link travel time and other data can be derived. The speed for any vehicle on the link is determined by the number of vehicles on the link and the shape of the speed-flow curve for that link. Both arterials and freeways are coded in this manner.

The presence of traffic signals is represented by allowing the link to discharge traffic only during the effective green period. If the light is red, vehicles are not allowed to exit the link and must queue in a first-in, first-out (FIFO) stack. During the effective green period, the discharge pattern of one link automatically becomes the arrival pattern at the upstream end of the next link, subject to any time delays associated with travel along the link. This consideration for platooning allows for the analysis of the effect of coordination offsets to be modeled. Ramp metering strategies are modeled as traffic signals where the cycle time corresponds to the desired ramp metering rate with one vehicle discharging per cycle. INTEGRATION's capabilities to model traffic signals is discussed further in Sections 3.6.1 and 3.6.2.

3.1.3 Continuous Dynamic Queuing-Based Traffic Assignment

INTEGRATION models traffic congestion in a network by its explicit account of queue growth and decay while maintaining a dynamic equilibrium traffic assignment. The explicit account of queue size and delay through the tracing of individual vehicles permits direct modeling of queue spill-back from downstream links, continuous modeling of traffic signal progression, and automatic delay of downstream link arrivals if they are held up at an upstream bottleneck. The model also determines the change in travel time on a link based on any queuing occurring on the link. As the travel times change between competing routes, the model is equipped to simulate the shift in traffic between these routes automatically.

3.1.4 Car-Following Analysis

A recent modification to the model was the determination of vehicle speeds using standard car-following theory. For every tenth of a second interval, the model determines the speed of each vehicle, the speed of the vehicle in front of it, and the distance between the two vehicles. Using these data the model determines the speed of the vehicle for the next tenth of a second. By determining speeds in such a manner, queues are created and dissipated in a manner consistent with field observations and standard car-following theory.

3.1.5 Different Driver/Vehicle Types

Another rather unique feature of the INTEGRATION model is its ability to assign a number of vehicle types to the network. The main difference between the various vehicle types pertains to their ability to access real-time traffic information. The model also has a vehicle type that has exclusive access to certain links in the network. The specific nature of each of these vehicle types is provided in Section 3.6.3.

3.2 INPUT DATA REQUIREMENTS

The INTEGRATION model uses nine different input tiles, five mandatory and four optional, for each simulation run. While the user is free to choose any name for these files, it is recommended that the last character of the name of the file end in the number of the data file and that the extension ".DAT" be used. For example, a node file might be named "FILE1 .DAT". These input files are listed in Table 3.1 and described below. The entire input files developed in this research are included in the Appendices and will be discussed in a later chapter. The user's manual for INTEGRATION [61] also provides greater detail on the structure of these files.

File Name	Status	Description
Data File 1	Required	Node descriptor
Data File 2	Required	Link descriptor
Data File 3	Required	Signal timing plans
Data File 4	Required	Origin/destination demand matrices
Data File 5	Required	Incident descriptor
Data File 6	Optional	Average link travel times for entire simulation
Data File 7	Optional	Time series of anticipated travel times
Data File 8	Optional	Static path tree for type 5 vehicles
Data File 9	Optional	Time series of multipath background traffic routings

Table 3.1: INTEGRATION Input Data Files

3.2.1 Data File 1: Node Descriptor File

This file lists the x and y coordinates, node type, macro cluster number, and changeable message sign (CMS) flag for each node in the network. The coordinates listed in this file are used solely for the purpose of graphical display. This is useful in that small areas of great detail, such as complex signalized intersections and freeway/arterial interfaces, can be coded larger than they actually are to reveal the detail of the area. Node type tells the model if the node is an origin, a destination, neither or both. A node that is either an origin, a destination, or both is referred to as a zone.

Macro cluster numbering is a recent feature of the INTEGRATION model that allows destinations to be grouped together in an effort to reduce the time required to compute minimum path trees. For all links that are farther than a certain distance from a given macrocluster of destinations, the model will only calculate path trees to the macro cluster and not to each individual destination. If the link is within a certain distance of the macro cluster of destinations, the program calculates path trees to each of the destinations within the macro cluster. The CMS flag indicates whether there is a changeable message sign at this node or not.

The numbering of nodes in the network does not need to be consecutive and there may be gaps in the numbering of nodes. An organized numbering scheme can assist the user in error checking the network. It is recommended that the first node identification numbers be reserved for zones. This is because the maximum zone number allowed by the version of the model is likely much less that the maximum number of nodes allowed.

3.2.2 Data File 2: Link Descriptor File

This file lists the start and end node for each link, supply aspects of the link, any turning restrictions, traffic signals, and other data for all of the links in the network. The supply characteristics of a link include the length, saturation flow, number of lanes, free flow speed, and the shape of the speed-flow curve. As with node identification numbers, link numbers need not be sequential and there may be gaps in the numbering scheme. Again, an organized numbering scheme is highly recommended.

The speed-flow curve for each link may be specified in one of two ways. For both of these methods, the free flow speed and basic saturation flow are used to determine the bounds of the curve. For the first method, two coefficients (A and B below) from a simplified version of the standard Bureau of Public Roads travel time function are specified by the user. This function is shown in Equation 3.1.

$$tt_i = tf_i \{1 + A(\frac{v_i}{c_i})^B\} \qquad \textbf{Eq. 3.1}$$

Where: tt_i = travel time on link i (sec.)
tf_i = free flow travel time on link i (sec.)
v_i = volume on link i (vph)
c_i = capacity of link i (vph)
A = increase in link travel time at capacity
B = change in rate of increase in link travel time

The second method of specifying a speed flow curve is to use the speed at capacity (km/h) and the jam density of the link (veh./km.) instead of the A and B coefficients. The model will automatically use this method if either the A or the B parameter is greater than 10. This latter method is the more recently developed of the two and is recommended by the developer of the model. The creation of the speed-flow curve using this method is discussed by Van Aerde [62].

The model allows the user to specify for any link a turning restriction that may be in effect for any period of time during the simulation. The user specifies the link that traffic exiting a specific link may not travel to, and the beginning and ending time of the restriction. The model also allows the capacity of a link to be decreased based on the volume/capacity ratio of an opposing link. The reduction in capacity on the link due to traffic on the opposing link is based on equations found in the Canadian Capacity Guide [52]. For signalized links, the model checks to ensure that the opposing link discharges on the same phase as the current link. This allows for the modeling of left turn movements that have a permitted and a protected phase. Further discussion of the opposing link feature is provided in Chapter 6.

Signalized links at an intersection are given the signal number that controls the intersection as well as the phase or phases during which the link discharges. Finally, the file contains flags that determine whether the link is reserved for use by type 5 vehicles only and whether the link is part of the real-time travel time surveillance network. This determines if the real-time travel time information from this link will be used in the calculation of real-time minimum path trees.

3.2.3 Data File 3: Signal Timing Plans

The signal timing plans are specified in terms of the cycle length, offset, number and sequence of phases, and the green time and lost time for each of the phases. The user may specify a number of signal timing plans that will be used throughout the simulation period. The plans are simply listed in order of use and the model changes plans at a user specified interval. The program has the capability to optimize individual intersections based on the volume/capacity ratio of each approach. The optimization of signalized intersections is discussed in greater detail in Section 3.6.2.

3.2.4 Data File 4: Origin/Destination Demand Matrices

Traffic demands are specified to the model in terms of origin/destination traffic flow rates between specific origin/destination nodes. The time periods for which these rates are assumed to prevail, and the distribution of the demand between different vehicle types for each origin/destination pair is specified. The user may also specify a growth factor that will uniformly increase or decrease all demands by a given amount.

The specified origin/destination hourly rates are translated internally within the model into corresponding departure times for individual vehicles. The individual vehicle departure headways may be entirely uniform, entirely exponential or derived from a user-specified shifted-exponential distribution.

3.2.5 Data File 5: Incident Descriptor File

This file provides a description of the incidents to be modeled. The file indicates the incident number, the link impacted, the number of lanes affected, the start time and the end time of the incident. Several incidents are allowed for consideration at the same time on different links, or at different times on the same link. Incidents can be arranged to represent a single incident with varying levels of severity.

3.2.6 Data File 6 (optional): Average Travel Times for Entire Simulation

This file provides a listing of the average link travel times that are expected to prevail throughout the entire simulation run. This expectation is typically based on historical or forecasted data. The expected average link travel time database is utilized internally to assist in determining the routes that background (i.e. unguided) traffic should take through the network. This file is used to represent drivers with information about recurring congestion in the network, but not knowledge of non-recurring congestion, that is the average unguided vehicle familiar with the network.

3.2.7 Data File 7 (optional): Time Series of Anticipated Travel Times

This file is very similar to data file 6. The only difference is that file 6 only allows a single average travel time value to be specified for each link for the entire simulation, while file 7 allows the modeller to inform some of the drivers of an entire series of expected future travel times. The length of the time period that each series of travel times is in effect for may range from 1 to 60 minutes.

Such travel time data may represent additional knowledge of background traffic about typical variations in travel times during the simulation period. In this case, these vehicles would route themselves based on different information, depending upon the times these vehicles actually started their trip. This time variation is intended to represent the travel time experience drivers acquired on previous days during the same commute.

3.2.8 Data File 8 (optional): Static Path Tree for Type 5 Vehicles

This file is used to communicate a user specified path tree to the program for use by the type 5 vehicles. The use of this tile, as a means of specifying the routes for special facility users, is preferred over the use of real-time route calculations when the routing of special facility users is known or constrained a priori. However, when the routing of special facility users is to be a function of the relative attractiveness of routes that utilize special facilities and those that do not, internal route calculations are preferred and this file should not be used.

3.2.9 Data File 9 (optional): Background Traffic Routings

The purpose of file 9, with respect to vehicle type 1, is the same as the purpose of data file 8 with respect to vehicle type 5. That is, data file 9 specifies the routing trees that vehicle type 1 should use in order to arrive at their destination. During each time period a series of equilibrium multipath routings for vehicle type 1 may be specified. By changing these routes and/or the weights given to each route from one period to the next, a dynamic series of routing trees may be specified for type 1 vehicles.

3.3 MODEL OUTPUTS

The INTEGRATION model provides a wide range of output that may be used in the calibration of the model and to assess the results of any given simulation run. The four types of output provided by the model are;

- On-screen graphics/text (displayed on video monitor)
- Error and diagnostic output file
- Labeled output statistics
- Unlabelled output statistics.

These model outputs are described in this section. Again, the user is referred to the INTEGRATION user's manual [61] for further details.

3.3.1 On-Screen Graphics/Text

During the simulation run, the main graphics window at the center of the screen provides a graphical representation of the traffic network that is being modeled and the vehicles traveling through it. A screen capture from a sample INTEGRATION run is shown in Figure 3.1. This figure shows a portion of the network coded as part of this research. The Santa Monica Freeway runs east/west in the middle of the screen and Washington Boulevard runs east/west near the top of the screen. A typical diamond interchange is near the center of the screen and a typical expanded arterial intersection is seen above that.

While not discernible in Figure 3.1, the model displays each individual vehicle in one of four colors based on the traffic conditions that the vehicle is experiencing. The position on the speed-flow curve for any vehicle determines which color to display the vehicle. Table 3.2 lists the four colors and their associated traffic conditions. The traffic congestion increases from the top to the bottom of the table (from green to red). Upper and lower portion indicate the upper (uncongested) portion of the speed flow curve and the lower (congested) portion of the curve respectively.

This color arrangement is very helpful in determining traffic conditions at a glance. Excessive queues from traffic signals or freeway ramps are easily discernible allowing for the analysis of problems in a manner much simpler than examining the output tiles. The model also displays vehicles of type 4 and 5 in their own color so that the routing paths taken by these vehicles may be easily observed.

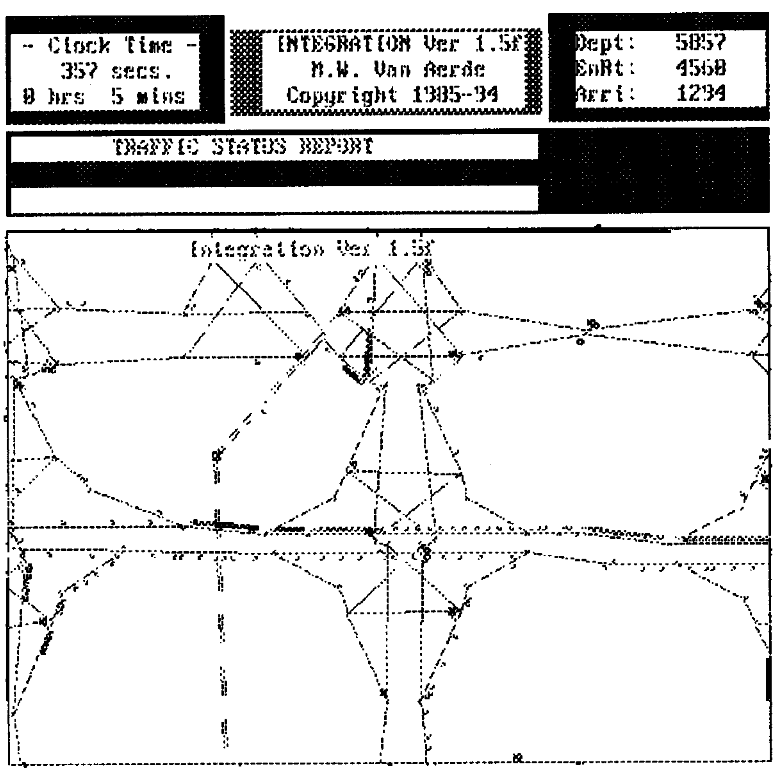

Figure 3.1: Screen Capture from INTEGRATION Program

Vehicle Color	Location on Speed-Flow Curve
Green	Upper portion - volume/capacity ratio less than 0.9
Blue	Upper portion - volume/capacity ratio greater than 0.9
Orange	Lower portion - volume/capacity ratio greater than 0.9
Red	Lower portion - volume/capacity ratio less than 0.9

Table 3.2: On-Screen Color Representation of Vehicles

Traffic signals are indicated as being either in an effective green mode or in a red mode using a solid red/green box. Any incidents are shown as solid yellow boxes, and the location and severity of the incident are noted in the traffic status report window at the center of the screen. At user-specified time intervals, the graphics window may also display the minimum path trees that are in effect at that time.

The simulation time and general vehicle statistics are shown in the upper left and right of the graphics screen respectively, as can be seen in Figure 3.1. The general vehicle statistics displayed consist of the number of vehicles that have left their origins, the number that are still en route, and the number that have arrived at their destinations.

During the course of the simulation, the zooming and panning features can be activated, allowing the user to confirm the network configuration or analyze the traffic conditions at any given location, and at any level of detail, in the network. The function keys allow the user to point to a specific link or vehicle and obtain current traffic statistics for that object. The function keys also allow the display of link, zone, and/or node numbers.

3.3.2 Error and Diagnostic Output File

Upon executing any simulation run the INTEGRATION model produces a file named "RUNERR.OUT". This file serves two main functions. First, the file lists the network size constraints that have been built into the particular version of the model.

Secondly, the file provides a listing of any errors that are detected by the model as having occurred during the course of initiating or running the model. These errors may either be detected during the course of the input data processing and/or during the actual execution of the simulation. Some of the errors are deemed fatal, which will cause the model logic to be terminated, while other errors are simply listed as warnings and allow the model to continue execution.

3.3.3 Labeled Output Statistics (output file 10)

The data in this file is labeled so that the user may easily see what the data represents. This tile contains the following types of information:

- *Initial data input file* echo - This echo provides an analysis of each of the input files for purposes of input data verification and diagnostics.
- *Periodic origin/destination travel time statistics* - An origin/destination travel time matrix which contains the current travel times is generated at a user-specified frequency.

- *System oriented link statistics -* This section provides sample statistics for each link in the network, namely the flows, total travel time, average travel time, volume/capacity ratio, number of stops and the number of vehicles that are queued on the link.
- *Signal timing plan summary -* If any signal optimization was requested during the simulation, the optimized signal timing settings for each signal in the network are indicated at the user-specified optimization interval.
- *Summary statistics of completed trips -* At the end of the simulation run, two further summary tables are generated. The first table summarizes the number of vehicles that completed their trip, the average trip time per arrival, and the accumulation of the total trip times for all vehicles. The second table is a summary of the vehicle demand on the network, the number of vehicles that entered the network, and the number of vehicles left on the network after the simulation is completed. Such statistics are generated not only for all vehicle types combined, but also for each vehicle type by itself
- *Incident summary* - A summary of any incidents that occurred in the network, listing the location, severity and duration of each incident, is provided.

3.3.4 Unlabelled Output Statistics (output files 11 to 20)

The final type of output statistics are in the form of unlabelled output files which are suitable as inputs into a spreadsheet, other computer programs, or even as one of the optional input data files for another INTEGRATION model run (input data files 6 to 9).

Output file 11 produces a summary file of the traffic conditions that were experienced during the entire simulation run. The format of this file is identical to that of input file 6. Output file 12 produces the same summary statistics that file 11 produces, except that the data is given for user-specified intervals and not for the entire simulation period. Output file 12 is similar to input file 7. Output files 11 and 12 produce more data than are needed for input tiles 6 and 7. They also provide link by link statistics of queue growth, volumes, speeds, number of stops, and travel times.

Output file 13 produces a listing of minimum path trees in a format similar to input files 8 and 9. Only one tree is produced per time period. Trees are output at user-specified intervals and are based on the real-time travel information that exists at that point in time.

Output file 14 is in the same format as input file 3, the signal timing plan file. This file contains a time series of signal timing plans, allowing the user to confirm fixed time plans or to check optimized plans.

Output file 15 produces a vehicle probe listing which lists numerous data associated with vehicles that are labeled as probes in input file 4, the origin/destination demand matrices.

There are also 5 new output files, numbers 16 through 20 that are produced by the model. The current version of the user's manual does not yet provide descriptions of these files. To operate the current version of the program, the user must specify the name of these output files or enter "none" in place of the output filename in the master file to not have the output files created.

3.4 MANIPULATION OF MODEL OUTPUTS

A number of programs were developed by the research team as part of this research to analyze the output data files from INTEGRATION in a more timely manner. This section discusses only programs that analyze output data from INTEGRATION or its supporting modules. The many programs that were developed to manipulate input data for use with INTEGRATION are discussed at later points in this report. The sheer volume of the output files when simulating a large network make it difficult to scan the files and understand the general traffic performance in the network. Efforts to develop programs similar to the ones discussed below have been underway at Queen's University, but none are currently released with the INTEGRATION model. All of the programs presented in this section are discussed in greater detail in the Appendices. The Appendices also provide the source code for all of these programs.

3.4.1 INTOMAN

The INTOMAN program, discussed in Appendix A, was created to analyze output data for the simulation of the freeway portion of the network. A detailed description of the model was also given in Bloomberg and May [9]. The model reads output files number 10 and 12 and allows the user to create speed or density contour maps, or provide summary data for a set of user-specified links in the network.

3.4.2 ODDATA

This program analyzes output from a supporting module of INTEGRATION named QUEENSOD (refer to Section 3.5.6.) Further discussion of the program is provided in Appendix B. The program reads an origin/destination matrix and produces summary statistics on the total number of vehicles departing and arriving at each of the origins and destinations in the network. A spreadsheet was developed that imported the output from this program. This spreadsheet then was used to create graphs that compared the simulated values to known flow values into and out of the zones in the network.

3.4.3 MPT

Output file 13 contains minimum path tree data that is referenced by links. For each link, the file lists the next link that should be taken to arrive at any destination in the network. To trace a path from a given origin to a given destination, the user would have to manually follow the path from link to link. The MPT program, discussed in Appendix C, does this automatically. The user simply enters the origin and the destination, and the program displays the links that would be used to complete the trip. This program proved quite effective because it allowed the INTEGRATION generated path trees to be checked quickly and accurately. This was very important in the calibration effort since the QUEENSOD program output was very sensitive to the minimum path trees that are provided as input to the program.

3.4.4 RED12

The RED12 program, discussed in Appendix D, simply reduces the data in output file 12 to a subset of links in the network and identifies any problem links. The user specifies a file containing a list of links in the network and the program copies the data from these links to another file. This file is then read into a spreadsheet for further analysis. The use of the same list of links for each simulation run allows the user to quickly create speed contour maps and compare simulated flows to field measurements. This proved valuable given the large number of simulation runs that were conducted and analyzed. The program also produces lists of links that match a certain set of characteristics. For example, the user may request a list of all the links where the delay on the link exceeded 100 seconds.

3.5 SUPPORTING MODULES OF INTEGRATION

This section discusses the supporting modules that may be used with the INTEGRATION model. In 1993, Gardes and May [20] discuss seven modules that were used with the INTEGRATION model. Of these, four were not used at all in this research; the MULTIPATH, Q-PROBE, ROUTCOMP, and REAL-TRAN supporting modules. The ASSIGN submodule has been incorporated into the INTEGRATION model itself, but is discussed in this section as a separate submodule. The remaining submodules, QUEENSOD and INTMAP are discussed in this section as well.

3.5.1 MULTIPATH

The QUEENSOD program requires a priori estimates of likely dynamic multipath routings through the network for each control period if a dynamic origin/destination matrix estimation is to be attempted. One way to estimate the dynamic multipath routings through the network is to use the MULTIPATH program. A detailed description of the mechanics and mathematics of this program can be found in a report by McKinnon and others [33].

3.5.2 Q-PROBE

QUEENSOD also requires an a priori knowledge of the travel times and traffic volumes on each link. The Q-PROBE module [59] can be used to generate a seed origin/destination matrix from traffic information that would be transmitted by vehicles equipped with the appropriate equipment, i.e. probe vehicles. The Q-PROBE model processes dynamic traffic information transmitted by in-vehicle route guidance systems prototypes to a central traffic management center.

3.5.3 ROUTCOMP

The ROUTCOMP module [60] can be utilized with either the ASSIGN or the MULTIPATH program, to compare their respective routings against each other or against a simple all-or-nothing assignment. Specifically, the program considers the routes and the route weights for the two routing strategies, on an origin/destination pair basis, to determine simultaneously the overlap in the routes that were selected and the similarity in the weights that were assigned to each route.

3.5.4 REAL-TRAN

INTEGRATION can generate real-time signal timing changes in response to the observed real-time link flow counts of the intersection approaches. A basic feature for performing on-line signal optimization of individual signal timing plans does exist within the model. The REAL-TRAN module [39, 40] permits a more comprehensive and complex optimization of a series of coordinated signals in a manner similar to the SCOOT system.

3.5.5 ASSIGN

As mentioned, the ASSIGN submodule [42] is now a feature of the INTEGRATION model that is activated when the user enters a negative value for the simulation time in the master file of the INTEGRATION program. The ASSIGN module produces estimated equilibrium routings for each of the time slices in the simulation run. The model produces three path trees for each time slice and a weighting factor for each of the path trees as output file number 17. These trees are then used as input for the QUEENSOD model.

The incorporation of this submodule into the INTEGRATION requires an iterative process to be run between the INTEGRATION and QUEENSOD programs. INTEGRATION is used to produce minimum path trees for use by the QUEENSOD program in estimating an origin/destination matrix. This origin/destination matrix is then put back into INTEGRATION and new minimum path trees are generated. The execution of this iterative process is discussed in Chapter 7. To eliminate the need for iteration between the two programs, the developer of the QUEENSOD model is currently developing a version of the program that incorporates the features of the ASSIGN model within the QUEENSOD model itself.

3.5.6 QUEENSOD

Arguably the most important supporting module of the INTEGRATION program is the QUEENSOD model. QUEENSOD is a model for estimating origin/destination traffic demands based only on observed link traffic flows, link travel times, and drivers' route choices. The estimation of an origin/destination matrix of traffic demands from link flow data is often preferred over other methods because of the high cost of collecting demand data.

The program is designed to work with the INTEGRATION model but can also be used as a stand alone model. Many of the input files required by the QUEENSOD model are of the same format as certain input and output files of INTEGRATION. This section provides a brief overview of the input and output files that are used and generated by the QUEENSOD model. The reader is referred to the user's manual [25, 26] for a more detailed description of the program. The use of QUEENSOD in the calibration of the Santa Monica Freeway network is discussed at a later point in this report.

Table 3.3 lists the input files that are used by the QUEENSOD model and the compatible INTEGRATION files.

File Name	Status	Description	INTEGRATION Compatible File
Input file 1	Required	Node descriptor	Input file 1
Input file 2	Required	Link descriptor	Input file 2
Input file 3	Optional	Actual origin/destination demands	Input file 4
Input file 4	Optional	Seed origin/destination demands	Input file 4
Input file 5	Required	Observed link traffic flows and travel times	Output file 11 or 12
Input file 6	Required	Routing paths utilized by traffic	Output file 13
Input file 7	Optional	Link flow reliability factors	none
Input file 15	Optional	Seed matrix accuracy factors	none

Table 3.3: QUEENSOD Input Files

The most significant input files for the QUEENSOD programs are input file numbers 5 and 6, the observed link flows and the minimum path trees respectively. The program works by starting with the given seed origin/destination demands and determining the link flows resulting from this demand. The demands are assumed to use the path trees that are given in input file number 6. The link flows resulting from the seed demand are then compared to the observed link flows to calculate the link flow error. The program then alters the seed demands in an effort to reduce the link flow error. This iterative process continues for a user-specified number of iterations.

Of the numerous output files produced by the QUEENSOD program, output files number 9 and 10 were used most extensively in this research. Output file 9 is the estimated origin/destination demand matrix that is in the same format as INTEGRATION input file number 4. Output file number 10 lists the observed link flows and those estimated by the model.

3.5.7 INTMAP

The INTMAP model was originally written at Queen's University. The model was rewritten as part of this research since the existing version would not allow for non-sequential link numbers and could not handle the size of the network coded. The program works by reading the link and node file from INTEGRATION and producing an AutoLISP macro that is read by the AutoCAD program. Within AutoCAD, the drawing of the network may be printed or exported to an HPGL[1] file for use in another graphics program. The program was useful in debugging the network during the initial coding process and creating large maps of the network for continued analysis. A detailed description of the program and the source code is included in Appendix E.

[1] HPGL stands for Hewlett-Packard Graphics Language.

3.6 INTEGRATION'S MODELING CAPABILITY FOR ITS INVESTIGATIONS

One of the potential unique capabilities of the INTEGRATION model is its ability to simulate various ITS strategies, alone and in combination with one another. In particular, the model simulates many Advanced Traveler Information Systems (ATIS) and Advanced Traffic Management Systems (ATMS) strategies. As mentioned in Chapter 2, the need to assess the potential benefits of these strategies is critical due to the high cost of actual implementation, operation and maintenance. This section discusses the capabilities of the INTEGRATION model to simulate ATIS and ATMS strategies.

3.6.1 Modeling Ramp Metering

INTEGRATION can simulate the presence of ramp metering using a traffic signal with only one phase. Figure 3.2 shows the coding of an on-ramp with a ramp meter. The procedure is to simply split the on-ramp into two separate links. The upstream link is coded with a traffic signal at its downstream end.

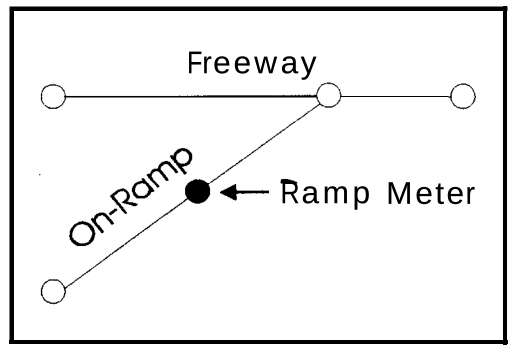

Figure 3.2: Coding of Ramp Metering

This signal is set with a two second green phase. This ensures that only one vehicle will discharge during each signal cycle. The ramp metering rate is determined by the cycle length. For example, a cycle length of 6 seconds will result in a metering rate of 600 vehicles per hour. At present, the model can not simulate cycle lengths that are fractions of seconds. This results in only certain metering rates being allowed. The developers of the model are working on a version of the model that allows non-integer cycle lengths.

The current version of the INTEGRATION model does not allow for ramp metering rates to be optimized based on the freeway flows. Again, this is proposed for a future version of the model. To develop an optimum ramp metering rate for the freeway under a given set of traffic conditions the FREQ model was used. The supply and demand aspects of the freeway were given to the FREQ model which produced an optimized set of ramp metering plans that were then used with the INTEGRATION model. However, since the FREQ and INTEGRATION models do not simulate the freeway in exactly the same manner, the use of the FREQ model ramp metering plans in an INTEGRATION run is quite limited.

3.6.2 Modeling Real-Time Traffic Signal Control

The INTEGRATION model is capable of performing limited optimization of individual traffic signals. The model optimizes green splits for single intersections by using the ratio of link volume to saturation flow rate for each signalized approach to the intersection using a modified version of Webster's method. First, the volume/saturation flow (v/s) ratio is calculated for all of the approaches to the given signal. Next, the approaches are grouped according to the phase of the signal timing plan in which they discharge. For each phase, the approach with the highest v/s ratio is determined and considered to be the critical approach for that phase. In order to assure a minimum amount of green time to each phase, if the critical v/s for any phase is less than 0.15 the program automatically assigns a v/s of 0.15 to that phase. Using these v/s ratios, the program then determines the optimum green splits for the intersection.

In optimizing any traffic signal, the user may specify how often the signals are optimized and the maximum and minimum cycle lengths that will be allowed. Thus, the user may preserve the original cycle length and the offset of the signal by specifying the minimum and maximum cycle lengths to be the same as the initial cycle length. Of course, if the cycle length is allowed to vary, any coordination of offsets will likely be lost.

3.6.3 Modeling Route Guidance Systems

In order to simulate the implementation of route guidance systems, the model uses the microscopic nature of the model to provide real-time information to certain vehicles in the network. The model uses a dynamic minimum path tree matrix to inform vehicles which path they should use at any given movement [45]. The discussion of INTEGRATION's capabilities to model ATIS should begin with a discussion of the minimum path tree matrix.

The minimum path tree matrix takes the form shown in Figure 3.3 for a network with x origins, y destinations, and z links.

Origin$_1$ - Link for Destination 1 . . Link for Destination y

Origin, - Link for Destination 1 Link for Destination y

Link$_1$ - Link for Destination 1 . . . Link for Destination y

Link, - Link for Destination 1 . . . Link for Destination y

Figure 3.3: Format of Minimum Path Matrix

The link that is listed with each destination in Figure 3.3 is the link that a vehicle should travel on after the current link or from the current origin, to arrive at this destination. With this matrix, the minimum path trees from any origin to any destination can be discerned by following the path from link to link. This method of providing information to guided vehicles is dynamic. The minimum path trees that exist as a vehicle leaves its origin may change before the vehicle has arrived at its destination. INTEGRATION continually updates these minimum path trees at a user-specified interval. This is the mechanism that allows for the provision of real-time traffic information.

The same format is also used to provide historical information to certain vehicles. However, in the case of historical information, the path trees are not updated during the simulation but are provided before the simulation begins. The user may specify a time series of historic information path trees. The historic information path trees may change at a user-specified interval. This interval and the interval in which real-time minimum path trees are updated do not need to be the same.

The five vehicle types of INTEGRATION are defined, for the most part, by their ability to access real-time and/or historical information. Table 3.4 lists the five vehicle types of INTEGRATION.

Vehicle Type	Description
1	Background vehicles - Route choice based on free flow speed unless specified trees are provided. The user may specify a set of path trees that the vehicles will follow. There may be more than one set of trees, each with its own probability of being selected, and the set of path trees may change over time. These path trees would likely represent historical information.
2	Guided vehicles - Have access to real-time information at every node in the network on which to base their route choice. The user specifies the frequency of information update and a distortion factor to reflect the amount of error in the information (see Section 3.6.4)
3	Vehicles which have anticipatory knowledge of expected future link travel times - Routing is based on a combination of real time and historical data. This historic data is used to determine the expected future link travel times.
4	TravTek vehicles - Have advanced route guidance systems within the vehicle itself.
5	Special facility users - Have exclusive access to selected links in the network (i.e. HOV vehicles). These vehicles can base their route choice on specified path trees or real-time information. In selecting their minimum path, they may use certain designated links, unlike the other vehicle types.

Table 3.4: The Five Vehicle Types of INTEGRATION

Type 1 vehicles are the general background, or unguided vehicles. To realistically represent an average commuter with historical information of the traffic patterns on the network, the average unguided vehicle should be provided with some type of path trees. Without any provision of path trees to this type of vehicle, they will follow the path trees based solely on free flow speed. This is not desirable as these vehicles will not divert under any circumstances, producing very unrealistic behavior. For this reason, it is suggested that historical information be provided to type 1 vehicles.

A series of path trees that are output from the calibrated base run of INTEGRATION are considered the historical information path trees. To establish the calibrated base run all of the vehicles should be guided vehicles of type 2. Using 100% type 2 vehicles, the assumption is that all vehicles in the network have knowledge of recurrent congestion and that the network has reached user travel time equilibrium. This assumption is necessary because determination of the exact paths taken by vehicles would undoubtedly be cost prohibitive. It should be noted that with this assumption there will be no benefit to using ATIS strategies under non-incident scenarios.

If an INTEGRATION run using 100% type 1 vehicles that follow the path trees that are output from the calibrated base run of INTEGRATION can not be established then the calibrated base run with 100% guided vehicles will have to be used for investigations. This restriction will significantly reduce the scope of the ATIS investigations to studies with a network loaded entirely with in-vehicle information systems. The results from these studies would only be relative since summary statistics from a network loaded entirely with unguided vehicles will not be available.

Type 2 vehicles represent the average guided vehicle. Type 3 vehicles represent guided vehicles with knowledge of future travel times. Type 4 vehicles are not recommended without knowledge of the specific characteristics of TravTek vehicles. Type 5 vehicles are special facility users that are also guided vehicles. If specific path trees are not provided to these vehicles via input file number 8, these vehicles will behave similar to type 2 vehicles with the exception that they may use all of the links in the network.

The percentage of each vehicle type in the network is specified in input file number 4, the origin/destination file. For each origin pair, five values are given that reflect the percentage of each vehicle type. Of course, these five values must add up to one hundred percent. While the percentages of vehicle types may vary for each origin/destination pair, collection of this type of data would be very costly.

A typical use of the model is to study the effect of market penetration of in-vehicle information systems on travel time savings for guided and unguided vehicles under non-recurring congestion [38]. For this type of analysis, type 1 and type 2 vehicles would be used. A sensitivity analysis would be done varying the percentage of type 2 vehicles in the network. A constant percentage of guided vehicles for each origin/destination pair is assumed. Gardes and others [19, 20, 22] give a good example of this type of study.

3.6.4 Modeling the Quality of Information Provided to Motorists

For each of the five vehicle types, the quality of information given to these vehicles must be specified. Two parameters, the frequency (F) and distortion (D) parameters control the quality of the information provided to the various vehicle types. Sensitivity analysis can be performed by varying either or both of these parameters. The F parameter specifies how often the path trees are updated and may be an integer value from 1 to 9,000 seconds.

The D parameter specifies the level of error that is introduced into the link travel times every time the minimum path trees are updated. The value of this parameter varies from -0.5 to 0.5. A positive value represents a normally distributed error while a negative value represents a log-normal distributed error. The magnitude of the introduced error is expressed as a fraction of the mean link travel time. The error term is intended to reflect the fact that drivers will not have perfect travel time information. This error term also provides INTEGRATION with the ability to incorporate a stochastic loading model, which prevents all vehicles from choosing the same route.

3.6.5 Modeling Changeable Message Signs

Changeable message signs (CMS) can be modeled with the model by specifying the nodes at which the CMS exist. Sensitivity analyses of CMS can be performed by varying the number and/or the location of the CMS. The model simulates CMS by providing information to type 1 vehicles as they pass a CMS designated node. For a limited period of time after passing these CMS nodes the type 1 vehicles perform similarly to type 2 vehicles. That is their routing is based on real-time path trees. CMS has no effect on other vehicle types.

3.6.6 Modeling High-Occupancy Vehicle Lanes

The INTEGRATION model is capable of simulating on-freeway HOV facilities. The model can simulate take-away or add-a-lane strategies. HOV lanes with complete barriers, partial barriers or no barriers can be simulated. Also, the HOV facility can be any number of lanes, and at any point or points along a freeway or an arterial. Priority bypass of ramp meters also can be coded with the model. A new feature of the model, turning restrictions, may to used to simulate an HOV facility where the status of a lane changes over time, i.e. a reversible or contraflow lane. The model can perform sensitivity analyses of the percentage of *passengers* in the network that are HOV passengers.

One problem discovered with the model was that it simulates using units of vehicles instead of passengers. When performing a sensitivity analysis of the percentage of passengers in the network that are HOV passengers, the number of vehicles must be changed to reflect the conversion between HOV and non-HOV vehicles. This must be done using a growth factor in the origin/destination matrix. A more complete discussion of INTEGRATION's capabilities regarding the simulation of HOV facilities can be found in reports by Bacon and others [4, 5].

3.6.7 Modeling Incidents

The model simulates incidents by restricting the capacity of a specific link for a specific period of time. The user specifies the link that is restricted, the start and end time of the incident and the number of lanes that are blocked. The INTEGRATION model has the capability to simulate a number of incidents.

An incident is modeled as a capacity reduction on the link's downstream end. The reduction in capacity is calculated from the specified number of lanes of traffic that are expected to be blocked by the incident. This number can be an integer, to reflect complete lane blockage, or a fraction, in order to indicate partial blockage of a lane.

The magnitude of diversion which occurs due to an incident is a function of the vehicle types, travel times on the effected routes, and the level of access that drivers have to travel time information updates. This was discussed in greater detail in Section 3.6.3.

3.6.8 Modeling Combinations of Strategies

One of the most promising aspects of the INTEGRATION model is its reported abilities of simultaneously simulating various combinations of ATMS and ATIS strategies. This creates a large number of experimental simulation runs that can be made with the model. This is very valuable as the optimal strategy for any implementation of ATIS and ATMS strategies likely involves a combination of the various strategies. The INTEGRATION model allows the analyst to study the options to a degree that would be cost prohibitive in reality.

3.7 PREVIOUS APPLICATIONS OF THE INTEGRATION MODEL

This section briefly describes experiences with the INTEGRATION model based on the review of the literature and information provided by the model developers. These experiences fall into three categories.

The developers have applied the INTEGRATION model to Highway 401 [23, 24] and the Burlington Skyway [58] in Ontario, Canada and as part of the TravTek Project [12, 43] in Orlando, Florida. These are the only currently published projects for which the INTEGRATION model has been calibrated and applied to a real world highway network.

The second category of experiences have been connected with the future application of the model in the FHWA Intelligent Vehicle Architecture Program [16, 65]. The third and last category of reported current and/or future experiences with the INTEGRATION model for which publications have not been located include the Central Artery Project in Boston, the QEW Freeway corridor network in Ontario, the I-80 Freeway corridor in the San Francisco Bay Area, the Sun Yat-Sen Freeway in Taiwan, and by the four contractors in the first phase of the FHWA Intelligent Vehicle Architecture Program.

Greater detail on these experiences are highlighted in the following paragraphs.

One of the first applications of the INTEGRATION model was a simulation of the Burlington Skyway near Hamilton, Ontario [58]. The model was used to evaluate and optimize the roadway under incident situations. The model was also used to study Highway 401 in Toronto, Ontario [23, 24] before and after the implementation of a freeway traffic management system. Both of these studies focused on the freeway portions of their respective corridor and did not involve a detailed calibration of the arterial streets.

The INTEGRATION model was also used to simulate the TravTek system in Orlando, Florida, an experimental route guidance system. Case and others [12] looked at the requirements for modeling the system and concluded that many of the supporting routines required to properly analyze the extensive input data were present in the INTEGRATION model. Rilett and others [43, 44] discuss the details of modeling the system using the INTEGRATION model and enhancements that were made to the model to improve its capability to simulate the system.

While the TravTek modeling effort involved a large real-life network, the calibration conducted for the arterial portion of the network was far less precise than that attempted in this research. The traffic flow data collected for the arterial streets was not quantitative in nature. The calibration of the network was based on freeway flow data. Also, the network used single node intersections which cannot accurately model complex intersections with large turning volumes.

As part of the FHWA IVHS Architecture Study [16], the INTEGRATION model is being used by four research teams to assess the potential of various IVHS strategies. Wunderlich and others [65] discuss the use of the INTEGRATION model in conducting the various tasks associated with this research. The research presented is largely conceptual in nature, presenting a possible research plan but not consisting of the calibration of an actual traffic network.

Chapter 4: DATA COLLECTION

This chapter discusses the data collection efforts that were undertaken as a part of this research. Section 4.1 provides an overview of the data collection efforts. Section 4.2 presents the collection of data from the freeway portion of the network, while Section 4.3 presents data collection efforts for the arterial streets. The results of these data collection efforts was one of the most complete data sets ever collected for the Santa Monica Freeway corridor.

Due to the large size of the network to be simulated and problems encountered during the data collection process, this portion of the research comprised a significant part of the research effort. This is probably the largest network to be coded for use with a microscopic simulation model. Data from the freeway portion of the network was not available and had to be collected manually from Caltrans' MODCOMP system. The installation of loop detectors on the arterial streets was delayed leading to a delay in the collection of arterial demand data.

4.1 INTRODUCTION

4.1.1 Supply, Demand and Control Data

In general, data for a transportation network can be viewed in terms of three facets: supply, demand, and control. For a traffic system, the *supply* components include the directly measurable aspects of the physical street and highway network (i.e. street lengths, number of lanes, and intersection geometries) and the theoretical characteristics of the network (i.e. the maximum capacities and densities of the roadways in the network). The *demand* in a transportation network consists of people desiring travel. This may be expressed in several ways, but two common means of describing demand are origin/destination pairs (the number of people who wish to travel between two points) and traffic performance data (the number of vehicles on the network at a specific point, in terms of vehicle flows, densities, or other statistics). Both of these measures are used in this study, and both are functions of time across the study period. The final piece of data collection involves ***control.*** For a traffic system, control elements might include signals, ramp meters, lane restrictions, or directional controls. High-occupancy vehicle (HOV) lane designation is also an example of traffic control.

4.1.2 Network Dimensions

As mentioned at the end of Chapter 2, it was decided that the efforts to simulate the Santa Monica Freeway corridor using the INTEGRATION model should be continued. For the present research, the network simulated by Gardes and May [20,22] was to be expanded with the following changes;

- The network was to be expanded from two parallel arterials to five. Olympic, Pico, and Venice were to be simulated in addition to Washington and Adams. Other significant arterial streets in the corridor would also be added.

- The length of the freeway corridor to be simulated was increased from approximately 6.0 to 9.3 miles. The new network will extend slightly beyond the San Diego Freeway (I-405) on the west, and to the Harbor Freeway on the east (SR-110).
- The time frame of the simulation would increase from four to fourteen hours, beginning at 6:00 a.m. and terminating at 8:00 p.m.
- New supply, demand and control data were to be collected. There had been a number of IVHS related projects proposed for the corridor and part of these involved the installation of loop detectors and other traffic data collection efforts. The City of Los Angeles was installing the ATSAC system on the corridor, a system of loop detectors to monitor arterial street traffic performance and an array of signal controllers to allow real-time signal optimization. The ATSAC system would likely result in a change in the signal timing plans used in the network.

Thus, a majority of the data that were used in the simulation by Gardes and May would have to be updated in order to simulate the proposed larger network. The supply data for the new arterials, and the portion of the Santa Monica Freeway not modeled by Gardes and May, would need to be collected. The existing demand data for the freeway portion of the network were from 1989 and needed to be updated. The arterial demand data were also quite dated. The ATSAC system offered the opportunity to update much of the arterial flow data. For both the freeway and arterial demand data, the new data would be more thorough, as well as more recent, than the existing data set. The control data for the network, including signal timing plans for hundreds of intersections and ramp-metering plans, would also need to be collected.

The network to be simulated was to represent the traffic conditions that existed in the summer of 1993. The freeway demand data was collected in May of 1993. The arterial demand data was collected in September of 1993. Two significant events since this time may have significantly affected the traffic conditions on the network. First, the Century Freeway was opened in October 1993. This freeway is parallel to the Santa Monica Freeway and about 10 miles to the south. The Century Freeway may offer an alternate route to many drivers who use the Santa Monica Freeway. Secondly, the Northridge earthquake in January of 1994 severely damaged the Santa Monica Freeway. For several months after this event, most of the users of the freeway were forced to use alternate routes. While the freeway has since been repaired, the permanent change in demand due to the earthquake is undetermined.

4.1.3 The Santa Monica Freeway Corridor

The area of the Santa Monica Freeway corridor to be modeled is illustrated in Figure 4.1, a map of the Los Angeles area. The Santa Monica Freeway itself runs east/west and is labeled as Interstate Route 10 in Figure 4.1. The portion of the freeway corridor that was simulated is highlighted in this figure, The limits of the simulated network are the San Diego Freeway (I-405) on the west, the Harbor Freeway (SRI 10) on the east, Olympic Avenue on the north, and Adams/Washington Boulevards on the south. Downtown Los Angeles is directly northeast of the network.

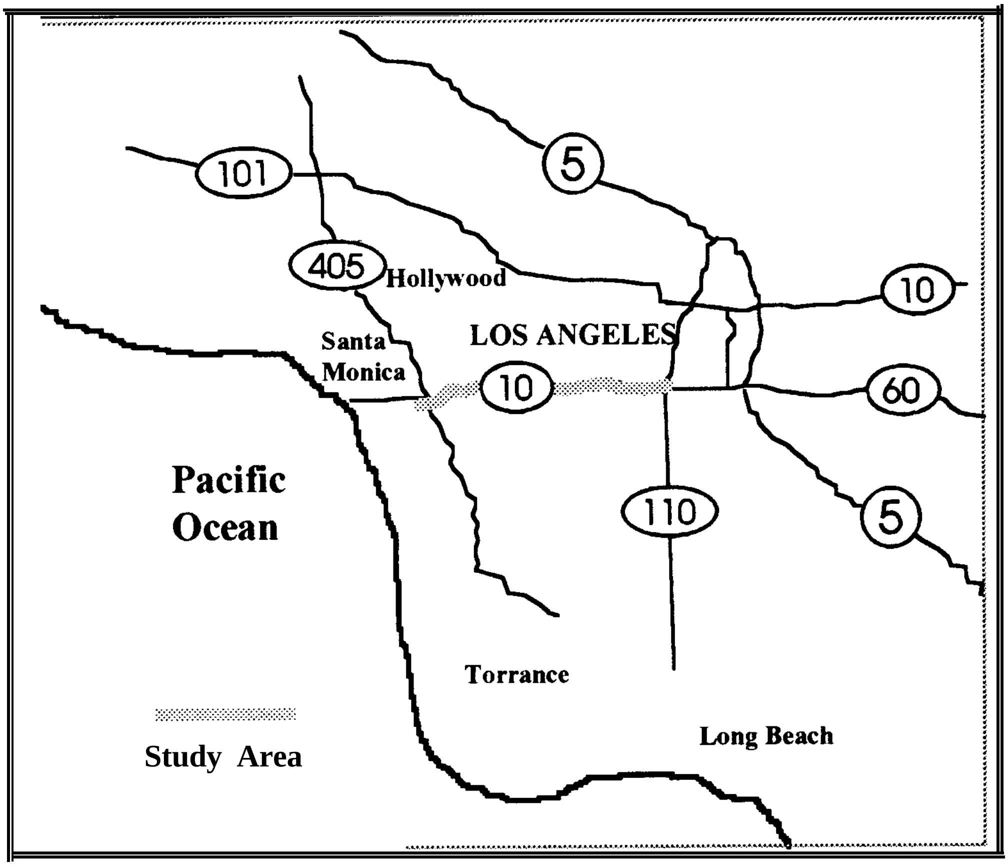

Figure 4.1: Map of Los Angeles and the Santa Monica Freeway

The Santa Monica Freeway is arguably one of the most traveled freeways in the country with an average daily traffic (ADT) of nearly 250,000 vehicles. The freeway corridor is also of interest because it has been the subject of various field test for ITS research. The Pathfinder project [35], an early test of in-vehicle information systems (IVIS), was conducted on the network in 1990. Currently, a number of public and private organizations in the Los Angeles area are working on the SMART Corridor project. This project is an attempt to install various ITS infrastructure on the freeway.

4.2 FREEWAY DATA COLLECTION

This section discusses the data collection efforts that were undertaken for the freeway portion of the Santa Monica Freeway corridor. The majority of the information in this section comes from the report by Bloomberg and May [9] that was conducted as part of this research project. The reader is referred to this report for further details. Collection and manipulation of the data for the freeway corridor was a very extensive process that resulted in a permanent record of the previous traffic count data in a systemic and retrievable manner.

This section begins by discussing the supply, demand and control data collections efforts for the freeway and concludes with a discussion of the spreadsheet that was created to store all of the data for the freeway portion of the network.

4.2.1 Freeway Supply Data Collection

The freeway portion of the corridor was defined as the portion of the network including only the mainline freeway and the on-ramps and off-ramps. The terminal ends of the ramps were located where the freeway ramp meets the arterial street. The intersection of freeway ramp and arterial street were later used as junction points when the arterial and freeway portions of the network were merged to form the entire freeway corridor (see Section 5.1.3).

Approximately eleven miles of linear freeway are included. This section of the Santa Monica Freeway includes 26 on-ramps and 26 off-ramps in each direction. Also, there are a total of four freeway connector on-ramps from the I-405 (northbound and southbound) to eastbound I-10, and from SR1 10 (northbound and southbound) to the westbound I-l 0. There are also four connector off-ramps, to the I-405 (northbound and southbound) from westbound I-10 and SR1 10 (northbound and southbound) from eastbound I- 10. Thus, a total of 30 on-ramps and 30 off-ramps are included in the freeway supply data. Figure 4.2 shows a schematic view of the freeway portion of the corridor by itself

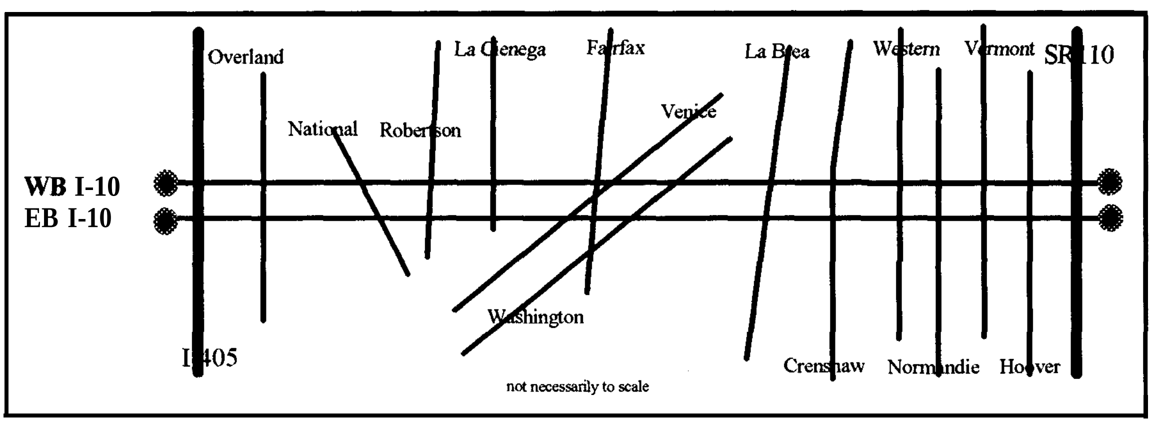

Figure 4.2: Schematic of Santa Monica Freeway

4.2.1.1 Geometry

The selection of which sections of roadway to include into the freeway section of the network was very straightforward. All of the freeway mainline and all of the on-ramps and off-ramps were included in the network. The first step in collecting the freeway supply data was to determine the x and y coordinates of the various nodes in the network, the lengths of the links in the network, and the location of the on-ramps and off-ramps. Initially, the x and y coordinates were coded to represent their exact physical location. These coordinates were later changed in order to view the network more clearly during simulation. The x and y coordinates of the nodes are for display purposes only and do not effect the simulation results.

A number of sources were available for this information. First, Caltrans District 07 provided aerial photographs of the freeway section under study. Each photo was taken to an exact scale (1 inch to 100 feet), so geometry data could be obtained simply by making measurements from the photos and converting to an appropriate metric. Next, several sets of maps of the freeway corridor from other studies were obtained from Caltrans and JHK and Associates. These maps described the networks in node/link format; attention was given to code the network in a similar fashion to allow comparison to others' work. Commercially available maps were also used to check certain details of the network. Finally, direct site visits were made to the freeway. The researchers drove the network a number of times to compare the tabulated data with the freeway itself It should be noted that actual site visits were considered to be very important in properly coding the network.

One limitation of these data sources was that each one only provided a relative reference of the components of the network; i.e. only the distance between two ramps was available, not the exact physical coordinates. This was an issue because each source of supply data had a different scale and varying detail. Therefore, in order to compare the information from two different maps, it was important to have absolute references from the freeway network. This was addressed by using coordinate values from the digitizing of the entire freeway corridor. Certain key points, namely the freeway/arterial interfaces, were obtained from the digitizing process and used as anchor coordinates. The other nodes in the freeway corridor were then determined by measuring distances relative to these anchor coordinates.

As a final check, the supply data were compared to the freeway geometries that were found in earlier research by Bloomberg and others [7, 8]. Freeway subsection lengths (i.e. the distances between ramps) and the number of lanes were compared and the values were found to be comparable. Also, the total freeway distances eastbound and westbound were equal. These tests confirmed the accuracy of the data collected for the freeway geometry. The next step was to estimate the capacity of the roadways in the network.

4.2.1.2 Capacity

For any traffic simulation, the issue of highway capacity is a critical one. In the real world, bottlenecks, queues, and congestion are caused because the traffic demand on a section of roadway is greater than some theoretical capacity value. When simulating such a system, it is necessary to specify this value for each section of roadway. However, there is no perfect method for determining the capacity of a given roadway. Some trial and error was needed to develop a reasonable set of capacity estimates for the Santa Monica Freeway.

As a start, a working value of 1,800 to 2,000 vehicles/hour/lane (vphpl) was assumed. The Highway Capacity Manual [53] suggests freeway capacities as high as 2,300 passenger cars/hour/lane (pcphpl) under ideal conditions. However, capacities this high on the Santa Monica Freeway may be unrealistic for several reasons. First, the on-ramps and off-ramps (some spaced at close distances) create weaving sections that will reduce effective capacity. Second, the presence of heavy vehicles (e.g. buses and trucks) should limit capacity. Finally, some freeway sections include numerous collector/distributor

(C/D) lanes that are included as part of the mainline freeway. Speeds and capacities are often lower on these facilities, so the average (per lane) capacity of the freeway is reduced in these sections.

The next step was to define a starting point for each freeway subsection. Each mainline was assumed to have a starting value of 2,000 vphpl. For those sections that began with an on-ramp, the capacity of the first lane (right lane) was reduced to 1,800 vehicles/hour. Also, those sections that included C/D lanes were estimated at 1,800 vehicles/hour for those lanes to recognize the fact that more weaving would be present in these sections so that capacity would be reduced. As an example, a four lane section of roadway after an on-ramp that had 2 parallel C/D lanes was assumed to have a capacity of 3(2,000) + 1,800 + 2(1,800) = 11,400 vehicles/hour.

To determine the impact of weaving sections on capacity, the FREQ simulation model was used. The FREQ model has a weaving analysis feature that will calculate the reduction in capacity for a given section of freeway based on the geometry of the roadway, namely the location of the on-ramps and off-ramps along the freeway. The reduction in capacity for the weaving sections along the freeway, as calculated by FREQ, were then made to the appropriate freeway links.

Figure 4.3 shows the final capacities that were used in the coding of the freeway portion of the network and the length of each of the links. Note that the capacities are, for the most part, between 1,900 and 2,000 vphpl. Nodes were placed at the location of off-ramps, on-ramps, loop detectors, and where the number of lanes in the freeway changes. Nodes were placed at the location of freeway loop detectors in order to permit comparison of field measured traffic performance with those predicted by the INTEGRATION model during later stages of the project.

For ramps, capacities varied depending on the geometry of the ramp merging section. Figure 4.4 shows the basic assumptions for capacities on ramps with different geometries. The capacity of a single lane ramp without an accompanying lane added on the freeway was estimated at 1,500 vehicles/hour. If the freeway section did have an additional lane adjacent to the ramp, the number of weaving maneuvers would be reduced and capacity increased. Depending on the length of the weaving section, capacity was increased to 1,800 or 2,000 vehicles/hour. For multi-lane ramps (used at the freeway interchanges), capacity was estimated at 1,500 vphpl.

	EASTBOUND (read down)				WESTBOUND (read down)			
link#	description (to end of link)	#lane	len (feet)	capac. (vphpl)	description (to end of link)	#lane	len feet	capac. (vphpl)
1	I-405 overpass	3	850	2000	SR1 10 overpass	3	600	2200
2	SB I-405 on	3	1900	2000	connector ramp	3	200	2200
3	SB I-405 off	4	50	1960	SB SR1 10 on	4	600	1900
4	National off	5	2900	1960	NB SRllO on	6	1450	1833
5	Overland on	4	1400	2100	20th Street off	7	700	1800
6	detector	5	1650	1880	20th Street on	7	1150	1943
7	Manning on	5	2150	1880	Vermont off	7	300	1800
8	Roberston off	5	1150	1980	Vermont on	7	1650	1943
9	detector	5	2100	1920	detector	7	600	1900
10	National on	5	300	1920	lane drop	7	100	1900
11	SB La Cienega off	5	1900	2000	Normandie off	6	100	1900
12	SB La Cienega on	5	1000	2000	lane add	6	400	1933
13	NB La Cienega	6	350	2000	lane drop	7	450	1943
14	Fairfax off	5	300	2000	Normandie on	6	700	1933
15	detector	5	1300	1960	Western off	6	550	1900
16	Venice on	5	400	1960	lane add	6	150	1933
17	Washington off	5	1550	1920	lane drop	7	400	1914
18	SB La Brea off	5	3800	2000	Western on	6	1550	1933
19	SB La Brea on	5	700	2100	Arlington off	6	500	1833
20	detector	6	200	1950	Arlington on	5	1750	2080
21	NB La Brea off	6	150	1950	Crenshaw off	6	2900	1950
22	NB La Brea on	5	1100	1920	detector	5	250	2000
23	Crenshaw off	5	2500	2040	Crenshaw on	5	1950	2000
24	detector	5	1450	1960	NB La Brea off	5	2650	2040
25	Crenshaw on	5	950	1960	detector	5	625	1960
26	Arlington off	6	2750	1967	NB La Brea on	5	125	1960
27	Arlington on	5	1650	2080	SB La Brea off	6	300	1900
28	Western off	6	450	1900	SB La Brea on	5	850	2000
29	lane add	6	1200	1933	Washington off	5	3550	1960
30	Western on	7	1150	1943	Venice off	5	1050	2000
31	Normandie off	7	850	1800	detector	4	750	2000
32	lane drop	7	400	1943	Fairfax on	4	1400	2000
33	Normandie on	6	1250	1933	La Cienega on	4	1800	2025
34	lane add	6	300	1900	Robertson off	5	1950	1820
35	Vermont off	7	350	1886	Robertson on	4	3350	2050
36	detector	7	1800	1943	National off	5	850	1880
37	Vermont on	7	150	1943	detector	4	1150	2100
38	Hoover off	7	400	1843	Overland off	4	3100	2100
39	Hoover on	6	1600	1933	Overland on	4	1250	1800
40	SB SRllO off	6	450	1967	SB 1405 off	5	2050	1800
41	NB SRllO off	6	1250	1833	NB 1405 off	4	1650	1925
42	lane add	5	200	1920	end	3	1525	2000
43	lend	6	675	1767				

Figure 4.3: Santa Monica Freeway Subsection Characteristics

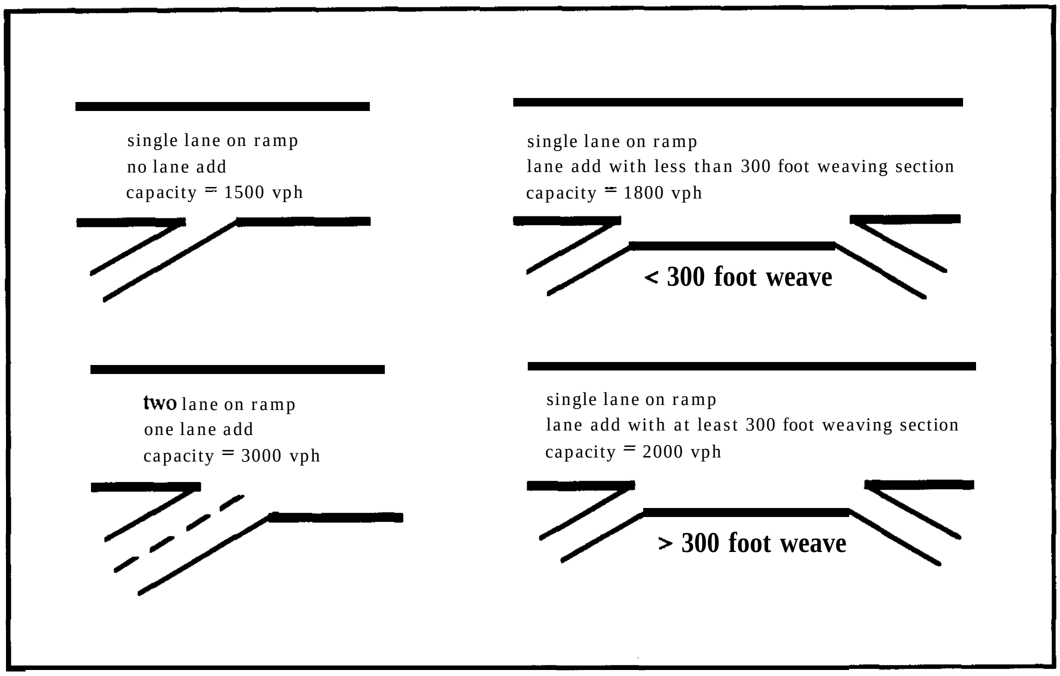

Figure 4.4: Capacity Assumptions for Different Ramp Geometries

4.2.1.3 Freeway Speed/Flow Relationships

For the INTEGRATION model, two different methods are available to describe the speed-flow relationship. The program developer suggests that the more robust method be used. This method is based on an approach integrating a car-following model and a generic speed-flow relationship described by Van Aerde [62]. Four parameters are required for this approach; the free flow speed, the speed at capacity, the jam density, and the capacity of the roadway.

The freeway portion of the network was coded and calibrated to match freeway flows in order to obtain traffic flows on all freeway links (Section 5.2.2) and to test the ability of the INTEGRATION model to simulate freeway performance (Section 6.3). Since the freeway portion of the network was calibrated using the FREQ model, the speed-flow relationship that was developed for INTEGRATION was designed to be similar to the one used by FREQ for the freeway calibration. It was found that by varying the above four parameters, a relatively close speed-flow curve could be developed. The resulting values for the mainline freeway were 99.8 km/h (62 mph) for the free flow speed, 70.8 km/h (40 mph) for the speed at capacity. Since the ramp speeds were lower, the corresponding values of 64 km/h (40 mph) and 45 km/h (28 mph) were used. The jam density was 130.5 veh./km (210 veh./mile) for both mainline freeway and ramps. The capacities varied as was discussed above. A plot of the speed-flow curve for a typical freeway mainline link is shown in Figure 4.5.

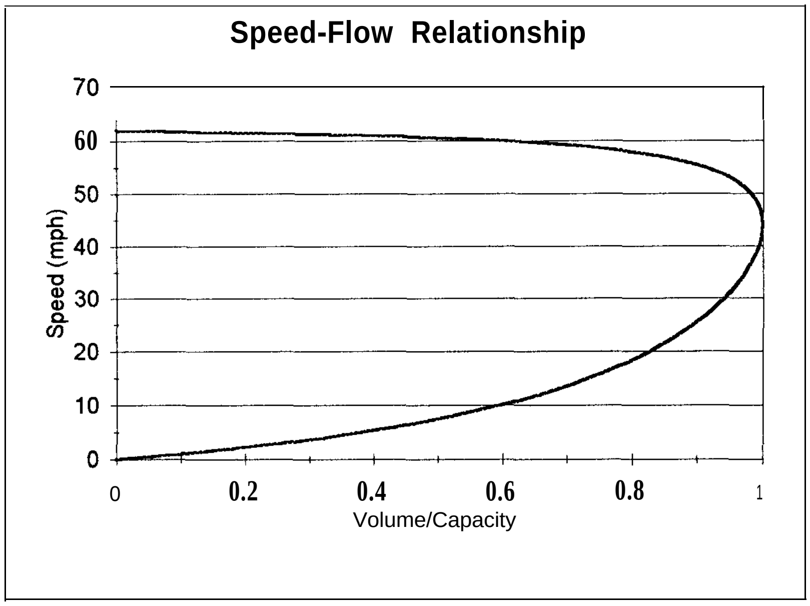

Figure 4.5: Speed-Flow Curve for Typical Freeway Segment

4.2.2 Freeway Demand Data Collection

While the collection of supply data is relatively straight-forward, the collection of demand data is often more complicated and laborious than the collection of supply data. Demand data are by their nature dynamic, so data must be collected temporally. It was decided that demand data would be aggregated into 30 minute periods.

In this research, traffic volumes were used to define the number of vehicles traveling between each pair of zones in the network during each time period, i.e. the demand. The traffic volumes collected for this project were collected from multiple sources in order to develop a timely, complete, and accurate demand data set. Alternative methods of data collection, e.g. license plate surveys, are usually far more costly. Even the method of estimating demand from traffic volumes proved to greatly tax the resources of the project.

4.2.2.1 Sources of Freeway Demand Data

The first source of demand data was Caltrans. With a system of loop detectors throughout the freeways of Los Angeles, Caltrans was able to provide traffic data through its MODCOMP computer system. Using this system, the research team collected traffic counts from 6:00 a.m. to 8:00 p.m., in 5 minute intervals, for three days from May 25 to May 27, 1993. These data included traffic flows at on-ramps and off-ramps, as well as on the mainline freeway. The collection of this data is explained in more detail in reports by Bloomberg and others [7, 8].

The first step in assessing the validity of this data was to determine where there was reasonable detector data. The definition of "reasonable" was somewhat arbitrary, but most detector malfunctions, at least on mainline freeway sections, were obvious enough to easily detect. Any detector determined to be unreasonable was removed from the traffic data obtained from the MODCOMP computer system. A detector would return one of three types of data; no data, all zeros, or some positive flow value. Obviously, the first two were discarded. For the freeway mainline sections, it was easy to determine if a given flow value was consistent with the rest of the mainline freeway. The ramp and other detectors (i.e. detectors on collector/distributor lanes and freeway connectors) were more difficult to assess. However, it was possible to subjectively assess which detectors were giving workable data: values within a specific range exhibiting some randomness and variation by time of day were considered to be returning good data.

Once this demand data were assembled, the next step was to determine what additional data were needed. The MODCOMP system did not provide detector data for every ramp in the network; data for off-ramps were especially limited. Obviously, it was critical to have traffic counts at all of the freeway locations, so other sources for these data were used. Of the other databases available, the most important was a record of Caltrans traffic counts for all of the freeway ramps and some mainline locations. These data included at least one day of hourly traffic volumes for each location from some point in time between 1989 and 1992. Also available were counts from the simulations performed in previous PATH research by Gardes and May [20]. These were half-hour volume counts for the morning peak period (6 a.m. to 10 a.m.) collected during 1988. Finally, data for a number of the ramps were also collected by the ATSAC system, since many ramps begin or end at signalized intersections monitored by the ATSAC system. The collection of data from the ATSAC system is discussed in the Section 4.3.

4.2.2.2 Manipulation of Freeway Demand Data

Once this demand data was collected, some of it required manipulation before it could be transferred into INTEGRATION input files (as discussed in Chapter 5). The manipulation of freeway demand data was done in three stages; demand triangulation, demand manipulation at the freeway ends, and demand adjustment using scale factors. Each of these three processes will be discussed in this section.

First, the raw demand data had to be aggregated into 30 minute volume counts. The data from Caltrans' MODCOMP system was aggregated into half hour data by simple addition of the appropriate five minute counts. Since the auxiliary ramp counts from Caltrans were in the form of hourly counts, a method had to be devised to convert these counts into half hour counts. To accomplish this, a demand triangulation spreadsheet originally created by JHK & Associates, and modified by Bloomberg and May [9], was used. This spreadsheet considers the demand in the previous and future time slices in dividing up the hourly flow into flow for each half hour time slice. An example of this process is shown in Figure 4.6.

Time	Input Volume	Output Volume
6:00 A.M.	221	168
6:30 A.M.		274
7:00 A.M.	436	388
7:30 A.M.		484
8:00 A.M.	604	582
8:30 A.M.		624
9:00 A.M.	593	608
9:30 A.M.		578
10:00 A.M.	486	494
10:30 A.M.		478
11:00 A.M.	525	526
11:30 A.M.		524

Figure 4.6: Demand Triangulation Example

Another limitation of the demand data was that there were no counts available for the freeway ends; the mainline freeway west of I-405 and east of SR110. To estimate these data, the available freeway mainline counts upstream or downstream of the freeway end were considered for each time slice. The ramp counts between the freeway end and the mainline detector were added and subtracted to obtain a time series of mainline flows at the four ends of the freeway. The volume counts that resulted were believed to be a reasonable approximation of freeway demand at these locations.

For many of the ramp counts, data were not available from the MODCOMP system and data from older counts had to be used. To adjust for the change in demand over time, a set of scaling factors was developed to convert these older counts into current demand data. For these scaling factors it was thought appropriate to classify ramps into low and high volume ramps. High volume ramps experience peak demand over 900 vehicles per hour. Then, for those ramps where data were available from both the MODCOMP system and from historical Caltrans counts, the average ratio was calculated and used as the scaling factor. The results of these calculations are shown below in Figure 4.7

Average low volume ramp flow data ratio (new/old) = 0.996
Average high volume ramp flow data ratio (new/old) = 1 .O 16
Average morning peak flow data ratio (new/old) = 0.97 1
Average midday flow data ratio (new/old) = 1.055
Average afternoon peak flow data ratio (new/old) = 0.986

Figure 4.7: Comparison Between MODCOMP and Historical Caltrans Counts

4.2.3 Freeway Control Data Collection

Freeway control data were the easiest element of the freeway data collection effort. Caltrans provided charts detailing the phase length for each on-ramp meter during a typical day. Ramp metering plans were in place throughout the peak periods at each on-ramp in the study area (except for the freeway interchanges). These data are shown in Appendix F.

4.2.4 Freeway Data Spreadsheet Summary

This section briefly discusses the format in which the data was kept after the initial stage of collection and manipulation. The supply and demand data were kept in spreadsheets while the control data was left in a text file. The data stored in these spreadsheets and text files were used to generate INTEGRATION input files as discussed in Chapter 5.

The supply spreadsheet created contains data for each of the nodes and links that will be coded into the INTEGRATION model. Each row of the spreadsheet represents that data for a single node or a single link. The node and link data are kept in separate portions of the same spreadsheet so that these data can be linked to one another by referencing the appropriate data in each portion of the spreadsheet. This allows for the creation of a flexible and dynamic spreadsheet, where a change in one portion of the spreadsheet may be automatically reflected in the link data. Figure 4.8 shows two portions of this spreadsheet which consists of the INTEGRATION node file and the INTEGRATION link file. Note that there is a shaded portion above certain columns of the node and link data. This shading represents the portion of the spreadsheet that will be exported to an ASCII file for use as an INTEGRATION input file. This allows other pertinent data not required in the INTEGRATION input file itself to be included in the spreadsheet.

					INTEGRATION Node File							
Node ID			Actual (in km)			Screen (in.)						
Ref	No	J H K	xcoord	ycoord	node	xcood	ycoord	o-d	Clstr		CM	Description
EFF	31	a66	20.637	4.941	31	30.31	13.47		3	-31	0	on-ramp (EB) Western
EFH	32	a67	21.547	4.941	32	31.63	13.45		3	-32	0	on-ramp (EB) Normndie
EFJ	33	a68	22.102	4.941	33	32.95	13.52		3	-33	0	on-ramp (EB) Vermont
BFB	34	<>	8.729	5.023	34	11.88	12.49		4	0	0	EB ML at I-405 overpass
BFC	35	r150	9.308	5.023	35	12.63	12.49		4	0	0	EB ML I-405 on-ramps
BFD	36	r151	10.261	5.023	36	14.01	12.39		4	0	0	EB ML Nat'l off-ramp
BFE	37	r1.52	10.521	5.023	37	14.45	12.39		4	0	0	EB ML Ovrlnd on-ramp
BFF	38	<>	10.875	5.023	38	15.10	12.36		4	0	0	EB ML Motor detector

INTEGRATION Link File												
	Start	End		F.F.	sat	#	Plat	Speed/Flow				
L i i	Node	Node	Length	Speed	Flow	Lane	Fact	A	B	{More Data...}		
1	17	34	0.259	100	2067	3	0	71	131		....	
2	34	35	0.594	100	2067	3	0	71	131	....	....	....
3	35	36	0.884	100	2000	5	0	71	131	....	....	....
4	36	37	0.427	100	2050	4	0	71	131	....	....	....
5	37	38	0.503	100	2000	5	0	71	131	....	....	....
6	38	39	0.655	100	2000	5	0	71	131	....	....	....
7	39	40	0.350	100	1980	5	0	71	131	....	....	....
8	40	41	0.640	100	1980	5	0	71	131	....	....	....

Figure 4.8: Portion of Supply Data Spreadsheet

All of the available demand data collected was coalesced into a single spreadsheet. A small portion of the complete demand data spreadsheet is shown in Figure 4.9. Since, the research could find no single source for demand data on the Santa Monica Freeway, the spreadsheet was used to select the best source of data for each of the locations. The two left columns indicate the location and direction of the data. The "source" and "date" columns describe where and when the data came from. The three right columns show the hourly flow rate for a given section for the first three half-hour time slices of the simulation. For each location, the most reliable source of data was used in the simulation. These lines are listed as "Simulation Data" in the source column. The appropriate growth factors discussed above were applied to each of the sources of data used in the simulation. For the purposes of this project and for future reference by others, this comprehensive finalized table of all available freeway traffic counts as of 1993 is included in Appendix G.

Location	Dir	Source	Date	6-6:30	6:30-7	7-7:30
Robertson on ramp	EB	(Simulation Data)		323	528	752
	EB	Caltrans hourly ct.	TUE 11/28/89	419	419	826
	EB	(converted)		318	520	912
	EB	previous UC study	1980s	447	529	864
Mainline 7.99 (National)	EB	MODCOMP data	TUE 05/25/93	3850	6920	8980
	EB	MODCOMP data	WED 05/26/93	4050	7010	8610
	EB	MODCOMP data	THU 05/27/93	3960	7080	8900
	EB	MODCOMP data	TUE 12/08/92	3890	6196	7894
National on-ramp	EB	(Simulation Data)		260	470	440
	EB	Caltrans hourly ct.	TUE 11/28/89	389	389	509
	EB	(converted)		360	420	496
	EB	MODCOMP data	TUE 05/25/93	260	470	440
	EB	MODCOMP data	WED 05/26/93	270	470	420
	EB	MODCOMP data	THU 05/27/93	280	470	400
	EB	previous UC study	1980s	351	374	332
	EB	ATSAC data	TUE 10/19/93	86	114	136

Figure 4.9: Excerpt from Final Demand Spreadsheet

4.3 ARTERIAL DATA COLLECTION

As with the freeway data, collection of the arterial data will be discussed in terms of supply, demand and control. This section will also discuss the spreadsheets containing the arterial data collected for the network. The data collection efforts for the arterial streets was also quite extensive and fraught with difficulties. The main delay to these efforts concerned the delay in getting the newly constructed loop detectors in the corridor on-line. The result of these data collection efforts is the creation of one of the largest comprehensive databases of arterial street data associated with a freeway corridor.

4.3.1 Arterial Supply Data Collection

In general, the supply data for the arterial portion of the corridor is the location and physical characteristics of the surface streets in the network. The first phase of the coding of supply data was to create the geometry of the network. This consisted of first determining which surface streets to include in the network, and then generating x and y coordinates for the nodes in the network. The next step was to connect the links to the appropriate nodes and provide them with the necessary data.

The coding of the arterial geometry actually proceeded in two separate phases. The first phase was the creation of a network with a single node representing every intersection in the network. The second phase consisted of expanding a number of the signalized intersections into an eight node/twelve link configuration that allows for a more detailed modeling of traffic movements at individual signalized intersections. The selection process of which intersections to expand and a discussion of the expansion process itself is given in Section 5.1.4.

4.3.1.1 Arterial Network Designation

The process of selecting which surface streets should be included into the network was done using US Geologic Survey (USGS) 7% minute quadrangle maps of the area. The decision was to include all of the major streets in the network and a number of the smaller arterials. Collector streets were not included. The USGS maps, commercially available street maps and visits to the site itself were used to decide which of the streets should be included in the network.

The resulting network consisted of the five major east/west roads parallel to the freeway; Olympic, Pico, Venice, Washington and Adams. Five other east/west roads; National, San Vicente, Cashio, 21st Street, and 1 lth Street, were also added to the network. These additional east/west roads do not run the entire length of the network. A total of 21 north/south roads were selected that run from the north end to the south end of the network. In addition, approximately 15 north/south roads were selected that do not run the entire length of the network. This arterial network is rather dense, which provides ample opportunity for alternate routes to be present for most origin/destination pairs. A plot of the freeway/arterial network coded for the INTEGRATION simulation runs is illustrated in Figure 5.4.

The research team also had to determine where origins and destinations (referred to as zones) would be within the network. The decision of how many zones there should be in the network turned out to be a critical one. On the one hand, the modeler would like to include any location on the network where a significant number of vehicles enter and/or exit the network as a zone. On the other hand, the time required to execute a number of the calculations performed by the INTEGRATION model is a function of the square of the number of zones. Thus, increasing the number of zones greatly increases the simulation run time. Additional zones also have significant memory requirements, which was a concern given the size of the network. The benefits of including a zone into the network when there is no flow data associated with that zone are questionable. Thus, the final decision was to code external zones (zones on the periphery of the network) only where the street connected to the zone was a major arterial or where there was flow data for vehicles exiting or entering the network on that street. This resulted in 56 external zones in the network. With two exceptions, all of the external zones are both origins and destinations, representing streets leading into and out of the network.

The selection of internal zones was done on the basis of the census tracts in the network [6]. Figure 4.10 shows the census tract boundaries as thick lines with the street network as thin lines in the background. The street network displayed in the background is shown in Figure 5.4 and discussed in more detail in Section 5.1.3. There are 56 census tracts in the network. Including all of the census tracts as internal zones provided a nearly even split between internal and external zones The use of census tracts as internal zones also allows for the use of census data if desired for any additional study. All of the internal zones are both origins and destinations.

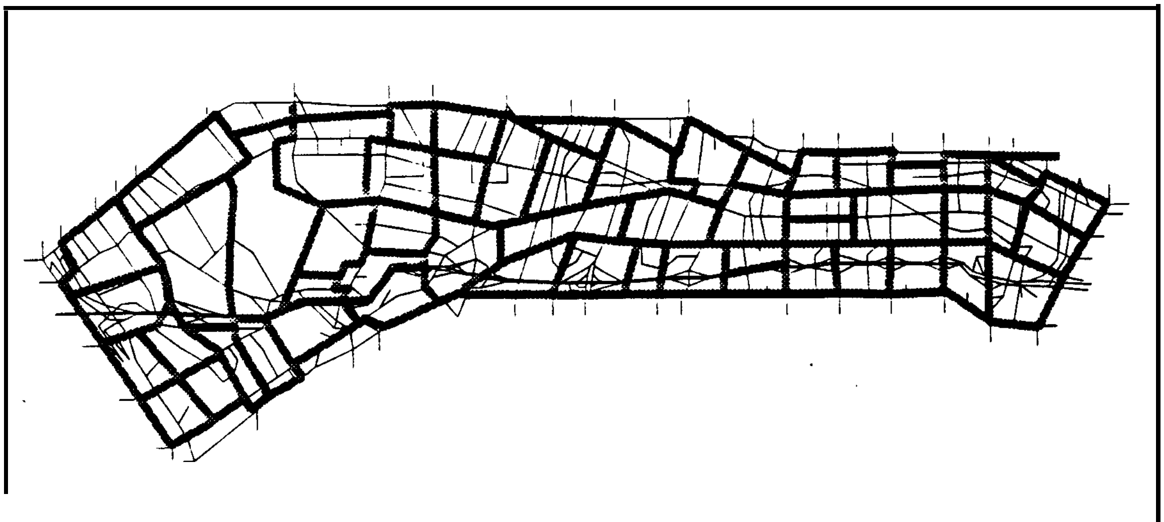

Figure 4.10: Census Tracts within the Network

4.3.1.2 Network Coding

The coding of the arterial network began by digitizing the major arterials and the freeway portion of the network using the Arc/INFO software. This resulted in a set of x and y coordinates for the nodes where any of the digitized links connected, and a set of links that are referenced as starting and ending at specific nodes. This node and link data was imported into the spreadsheet for the arterial node and link data.

Numerous alterations needed to be made to the spreadsheet before the arterial data could be used as input to the INTEGRATION model. First, all of the freeway links in the network were removed. Only the locations where arterials joined on-ramps and off-ramps were left in the arterial street database. The location of these arterial/freeway interfaces were later used when the freeway network was merged with the arterial network (see Section 5.1.3).

Secondly, all links in INTEGRATION are coded as unidirectional. With the exception of two one-way streets in the network, all of the link entries needed to be duplicated with the node reference numbers reversed to represent the same link with the opposite direction of travel. The portion of the spreadsheet containing the link data was then arranged in an orderly fashion, with consecutive link numbers proceeding along the direction of travel for a given arterial.

Since the digitizing was done only on the major streets there were numerous smaller streets that needed to be entered into the spreadsheet. The x and y coordinates of these smaller arterial streets were determined by interpolation. Again, the x and y coordinates of nodes are for display purposes only and do not affect the simulation results. There were a few instances where a major arterial would cross a minor street that was not included in the network, and there was a traffic signal at the intersection. A node was placed on the major arterial at this intersection to represent the signal. With this arrangement, the delay experienced on the major arterial due to the traffic signal would be properly represented. However, the assumption was made that the turning movements at these intersections were insignificant. Since the minor street is not modeled at these points, no turning movements can be modeled.

The internal zones were also added to the network using interpolation. The links connecting the internal zone to the rest of the network were then added. Since it is recommended that internal zones enter the network at mid-block locations as opposed to at intersections themselves, many links were split to create mid-block nodes. The internal zones were then connected to the network through these mid-block nodes. The number of mid-block nodes that connect to an internal zone ranges from one to three per internal zone.

4.3.1.3 Link and Intersection Data Collection

Supply data for the arterial streets was provided by JHK and Associates and the City of Los Angeles. JHK and Associates provided the number of lanes and link lengths for the majority of arterial links in the network. For the links where no link length was provided, the link length was manually measured using the USGS quadrangle maps. Direct site visits determined the number of lanes for an arterial link when this was not provided.

The City of Los Angeles' ATSAC system provided a number of intersection geometry templates that show the precise intersection configuration for the majority of intersections in the network. Figure 4.11 shows an example of one of these templates for the intersection of Pico Boulevard and Figueroa Street. This information on precise intersection geometries proved most useful in the process of expanding the single node intersections into eight node/twelve link configurations.

Finally, the coding of the arterial network was checked with site visits. The actual number of lanes, intersection geometry, and the location of signals were all checked against the coded data to ensure the accuracy of the arterial supply coding.

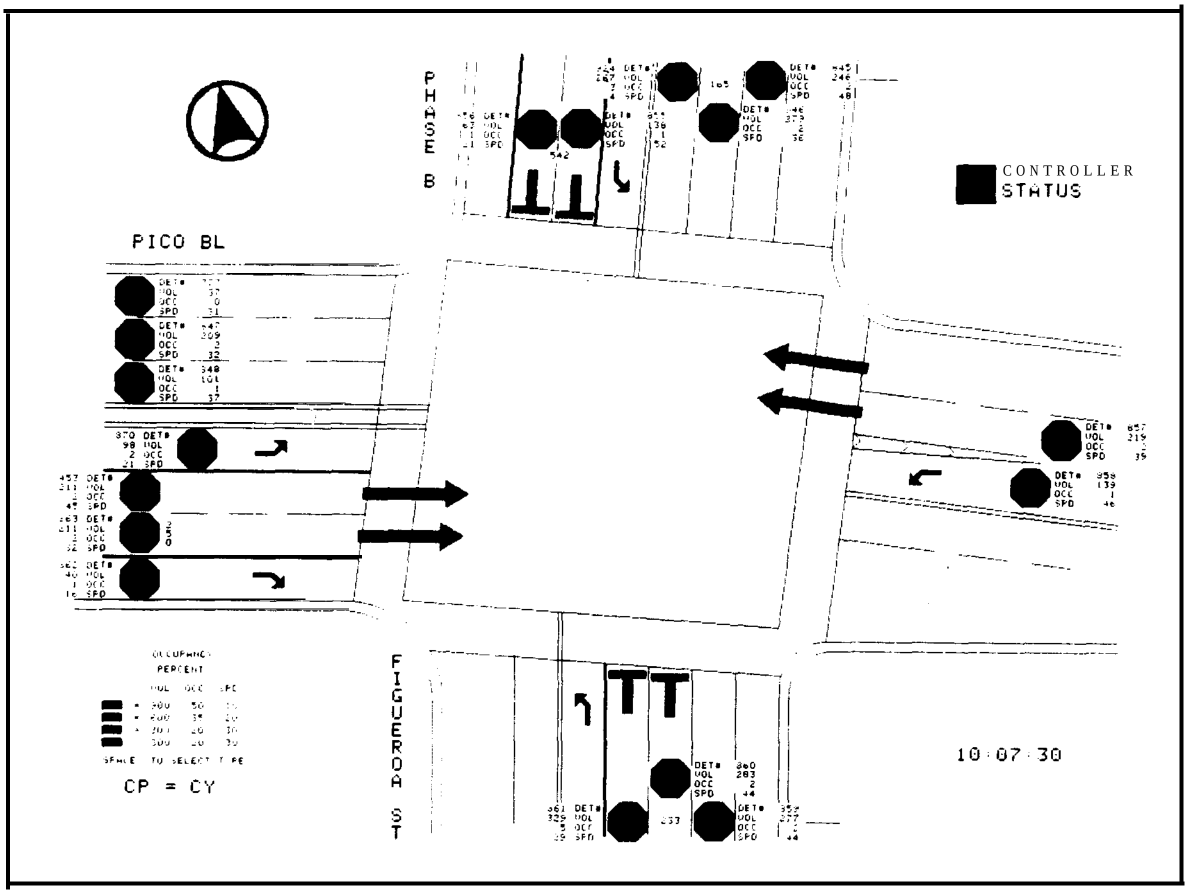

Figure 4.11: Sample Intersection Geometry Template - Pico and Figueroa

4.3.1.4 Determination of Arterial Street Capacities

Since there was no data collected on the capacity of the arterial streets in the network, the capacities for the arterial links had to be estimated based on the values in the Highway Capacity Manual [53]. First, all of the arterial streets in the network were classified into major and minor streets. The decision process was based on the USGS maps, commercially available maps, and visits to the site itself The capacity was set to 2,000 vehicles per hour per lane for the major streets and to 1,800 vehicles per hour per lane for the minor streets. Table 4.1 shows which arterial streets were classified as major and which were classified as minor. The capacity of links within the expanded intersection configuration is discussed in Section 7.2.1.

Major Arterials (capacity set at 2,000 vphpl)	Minor Arterials (capacity set at 1,800 vphpl)
4th	6th
Adams	11th
Apple	20th
Arlington	21st
Avenue of the Stars	Airdrome
Cadillac	Ashby
Century Park East	Bagley
Crenshaw	Beverly Glen
Fairfax	Beverwil
Figueroa	Cashio
Hoover	Clarington
La Brea	Cochran
La Cienega	Cotner
National	Crescent Heights
Normandie	Curson
Olympic	Doheny
Overland	Exposition
Palms	Gennessee
Pico	Harvard
Robertson	Hauser
San Vicente	Hillsboro
Sawtelle	Hughes
Sepulveda	Manning
Venice	Motor
Vermont	Prosser
Washington	Redondo
Western	Rimpau
	Roxburv
	Union
	Veteran
	Vineyard
	West
	Westmoreland
	Westwood
	Wilton

Table 4.1: Major and Minor Arterial Streets in the Network

4.3.1.5 Determination of Arterial Street Speed-Flow Relationships

Recall that the four parameters required to designate a speed-flow relationship with INTEGRATION are; the free flow speed, the speed at capacity, the jam density, and the capacity of the roadway. The free flow speed was initially set at 70 kilometers per hour (44 mph) for the major arterials and 55 km/h (34 mph) for the minor arterials. One exception was a portion of Venice Boulevard where the roadway is divided and designed to handle high speeds. The free flow speed for this section was set at 80 km/h (50 mph). During the calibration efforts, significant diversion from the freeway to the arterial streets was observed which led to a lowering of the free flow speeds for many of the major arterials. Table 4.2 shows the final speeds that were used for the major arterials in the network. Any arterials not listed in this table have a free flow speed of 70 km/h (44 mph) if they were designated as a major street and 55 km/h (34 mph) for minor streets. The speed at capacity was set at 0.6 times the free flow speed for all arterial links. This value was based on speed-flow curves that were defined within the Highway Capacity Manual. The capacity of the arterial links was described in the previous section. The jam density was set at 13 1 vehicles/kilometer (210 veh./mile) for all arterial links, the same value as was used for the freeway links.

Arterial	Free Flow Speed (km/h)
Adams	55
Crenshaw	70
Figureroa	65
Hoover	65
La Brea	75
La Cienega	65
National	65
Normandie	65

Arterial	Free Flow Speed (km/h)
Olympic	65
Overland	65
Pico	65
Robertson	65
Venice	65
Vermont	65
Washington	65
Western	70

Table 4.2: Free Flow Speeds for Major Arterials in the Network

With these four parameters defined the relationship between speed and flow, also known as the speed-flow curve can be defined. This relationship is equivalent to the speed-flow relationships used for the freeway links. A representative speed-flow curve for Olympic Boulevard is shown in Figure 4.12. Other arterial speed-flow curves have an identical form to this curve. The only difference will be the upper bound of the curve, which will always be the free flow speed. The maximum volume to capacity ratio will always be reached at a speed 0.6 times the free flow speed. Since the flow is measured in terms of volume/capacity ratio, the x-axis is the same for all of the curves. The curves could also be expressed with capacity on the x-axis. Again, the form of the curves would still be identical, but the upper bound on the capacity would vary between 1,800 and 2,000 vehicles per hour per lane.

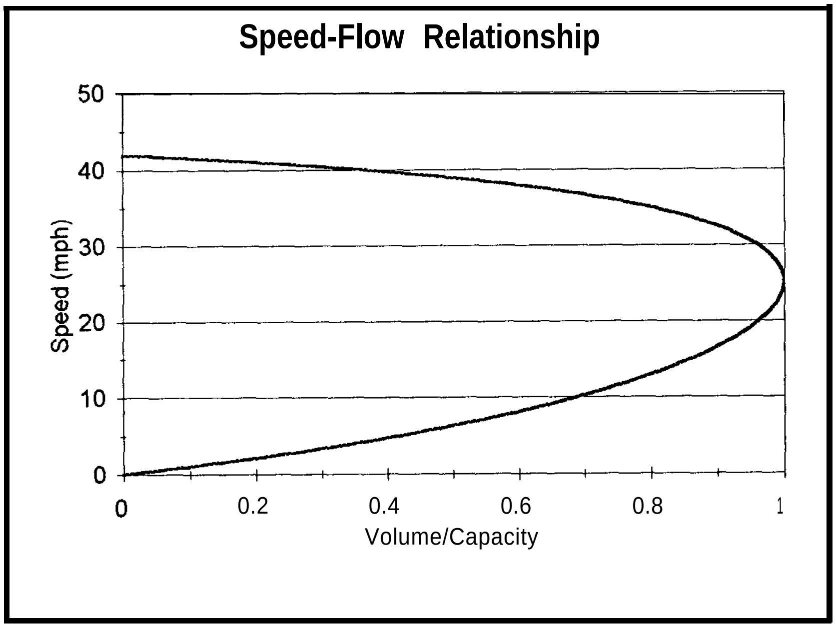

Figure 4.12: Speed-Flow Curve for Olympic Boulevard

4.3.2 Arterial Demand Data Collection

The demand data for the arterial streets came from two separate data collection efforts, both undertaken by the City of Los Angeles. The first set of data comes from a series of manual traffic counts that were done at selected intersections in the network. The second and most important arterial demand data collection effort was the collection of loop detector data from numerous detectors recently installed at most of the major intersections within the network.

As with the freeway demand data collection efforts, the observed traffic counts throughout the arterial network will be used to derive origin/destination demand data in the form of O/D matrices. This process is discussed in Section 5.2.5. Also, 30 minute time slices were used to capture the dynamic nature of demand data. The collection and manipulation of arterial street demand data proved to be a huge tax on the resources of the project.

4.3.2.1 ATSAC Data Tape Collection

In February of 1993, a binary data tape was received from the City of Los Angeles' ATSAC system. This data tape contained measured arterial flows from nine intersections along Olympic Avenue in the central business district, from Figueroa Street to Los Angeles Street. These observed flows were from 6:00 a.m. to 8:00 p.m. and disaggregated into flows for each individual signal cycle. At this point in time, the installation of loop detectors at all of the major intersections in the Santa Monica Freeway corridor had not been completed. The purpose of analyzing this data tape was to ascertain the format of the tape, develop methods to expedite further, more extensive, data collection efforts when the data became available, and to assess the validity of the detector data. It was found that the detector data on the tape were quite reliable, with only one detector in 43 giving questionable results. A report detailing the analysis of this arterial street loop detector data from the ATSAC system was written by Bacon and May [3].

In October of 1993, another data tape was received from the City of Los Angeles' ATSAC system. This tape contained both control data (signal timing plans) and demand data (loop detector counts). The tape contains data for 383 signalized intersections in the network and over 1,000 individual loop detectors. The data manipulation process to convert the binary tape data into 30 minute link flows and signal timing plans is illustrated in Figure 4.13.

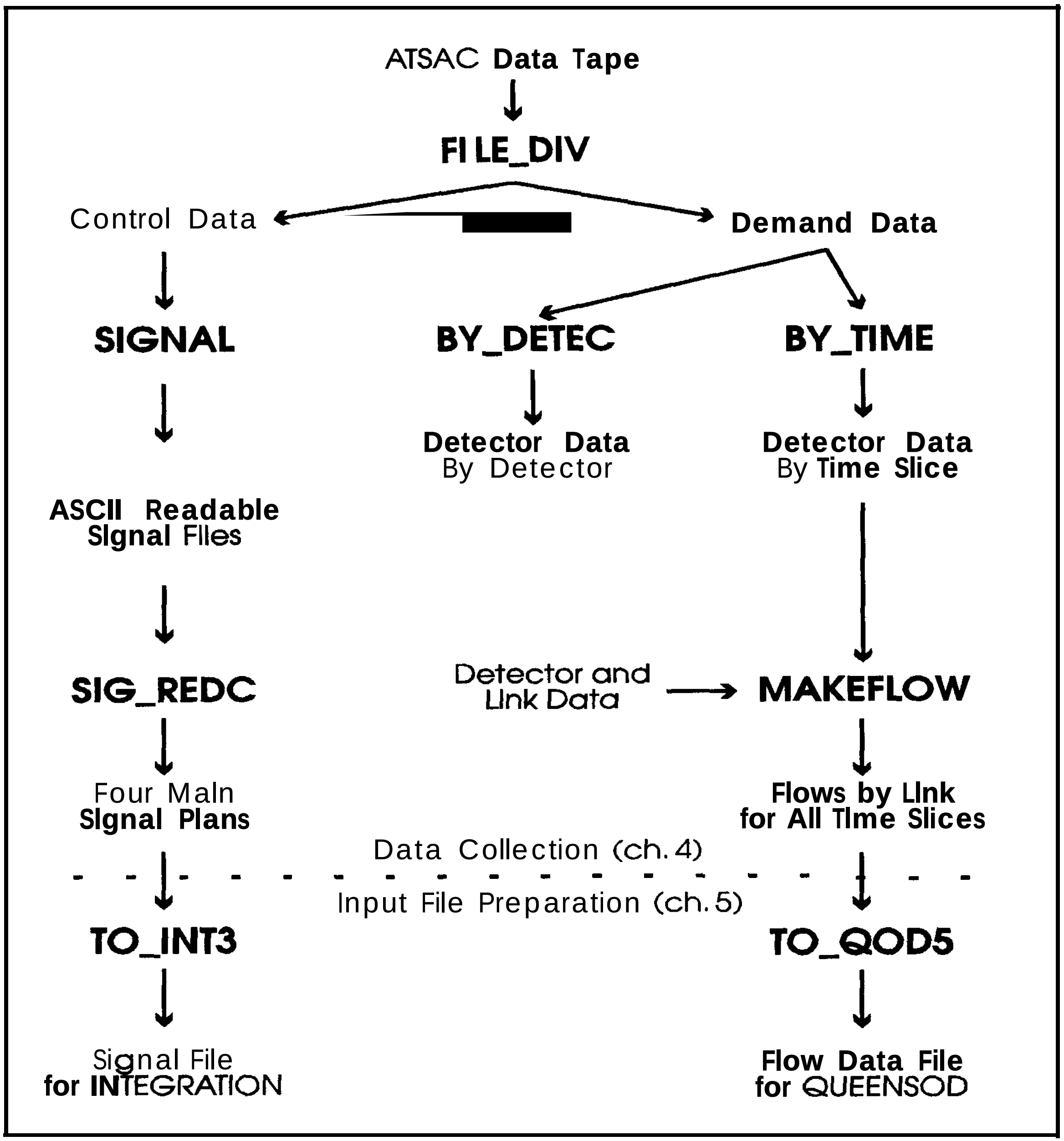

Figure 4.13: Manipulation Process for the ATSAC Data Tape

The first step in the manipulation of the ATSAC data tape was to separate the data into demand and control data. This was done using a simple computer program, FILE-DIV. The source code for this program and an explanation of how it works is given in Appendix H. The processing of the control data is shown in the left half of Figure 4.13 and is discussed in Section 4.3.3.1. The processing of the demand data is shown in the right half of the figure and is discussed in the Section 4.3.2.2. It should be noted that the bottom two manipulation processes in Figure 4.13, TO_INT3 and TO_QOD5, are considered input data preparation processes and are discussed in Chapter 5. The dashed line in the figure represents the somewhat arbitrary division of tasks into data collection and input file preparation. Of course, the ultimate goal of the data manipulation process is to create input files for INTEGRATION or one of its supporting modules.

4.3.2.2 Manipulation of Arterial Demand Data

The demand data (loop detector traffic counts) was broken down into individual data files in two ways, by individual detector and by individual time slice. This was accomplished by two computer programs, BY-DETEC and BY-TIME. Detailed discussion of these programs and their source code is given in Appendices I and J, respectively. Both of the programs work by reading the raw demand data file, stripping away any extraneous data, and creating individual files for each detector (BY-DETEC) or for each time slice (BY-TIME). A sample portion of the output files from each of these programs is given in Figures 4.14 and 4.15. The output from the BY-DETEC program is the flows and occupancies, in fifteen minute periods, for each detector on the tape. Each BY-TIME output file represents the average detector counts for a 30 minute period. Each output file is named according to the detector or time slice that the data represents.

```
06:00   440  6   512  5   688  6   696  6
07:00   780  8   848 15   784 14   856 10
08:00   728 18   772 10   696  7   584 10
09:00   488  5   500  4   568  5   468  6
1o:oo   520  7   540  8   568  7   576  7
           ...
```

For all lines;
1st column - Hour
2nd column - Flow for first 15 minutes of hour
3rd column - Occupancy for first 15 minutes of hour
Remaining columns - Alternating flow and occupancy for the remaining 15 minute periods of the hour.

Figure 4.14: Output from BY-DETEC

The data for individual detectors was used mainly in assessing the integrity of the data from each loop detector. There was concern that there may be inaccuracies in the detector data given that the loop detectors went on-line only in the latter part of July 1993, only two months before the data tape was collected. Any data that did not appear reasonable was discarded from any further data manipulation or input coding processes.

```
0007      216    2
0008      182    1
0009      124    1
0010      122    1
0011       42    0
           ...
```

For all lines;
1st column - Detector
2nd column - Flow for detector for given half hour
3rd column - Occupancy for detector for given half hour

Figure 4.15: Output from BY-TIME

The first test of the detector data was the analysis of the flow data for all of the detectors at a given approach. Figure 4.16 shows an example of this type of analysis.

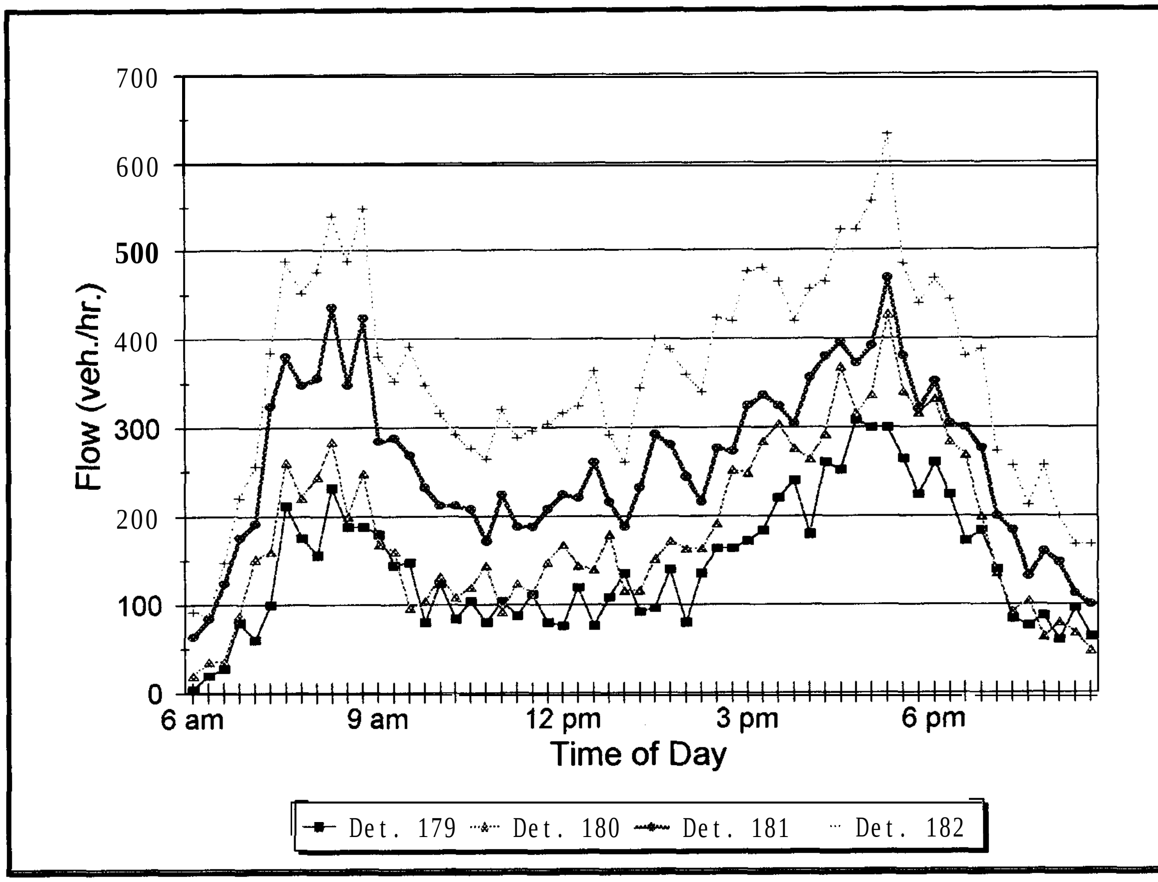

Figure 4.16: Detector Flow Data from San Vicente, Southbound Approach at Pico

Figure 4.16 shows the individual flow data for each of the four detectors at the Southbound approach of San Vicente at Pico. Detector number 179 is located in the right turn lane, detector numbers 180 and 181 are in the two through lanes, and detector number 182 is in the left turn lane. The left turn volume at this intersection is the heaviest of all the flows. This is not surprising as the left turn movement has its own signal phase. The general test from this type of analysis was to simply insure that the general pattern of the detector data for each detector at a certain approach was in general agreement with the other detectors at that approach. It is clear from the above figure that this is the case for this intersection. Similar analysis was performed on a number of intersections in the network.

After the analysis of individual detector data, the detectors were grouped by approach. The flow data for all of the detectors at an approach was consolidated into link flow data using the MAKEFLOW program. A complete description of this program along with the source code is given in Appendix K. This program requires an input file which contains data pertaining to which detectors are associated with which link. Each of the links in the network is associated with one to four detectors. Figure 4.17 shows a portion of this input file. The other set of data files required by the MAKEFLOW program is the files containing detector data grouped by time slice.

1	3	0892	0893	0894
3	3	0900	0901	0902
5	2	0908	0909	
7	2	0915	0916	
9	2	0921	0922	

...

For all lines;
1st column - Link number
2nd column - Number of detectors associated with link
Remaining columns - Detector numbers for the detectors associated with the link

Figure 4.17: Portion of Link and Detector for MAKEFLOW

For each link, the MAKEFLOW program accesses the flow data for each detector associated with that link, adds the detector flows together, and outputs the link number and its total flow to an output file. For each time slice one output file is created. Each output file from this program simply consists of the link numbers and the total link flows for the given time slice.

With the data in the form of link flows by time slices, flow along an arterial could be plotted and checked for reasonableness. Figures 4.18 and 4.19 show the link flow along Pico westbound and Venice eastbound respectively for the time period from 7:30 a.m. to 9:00 a.m.

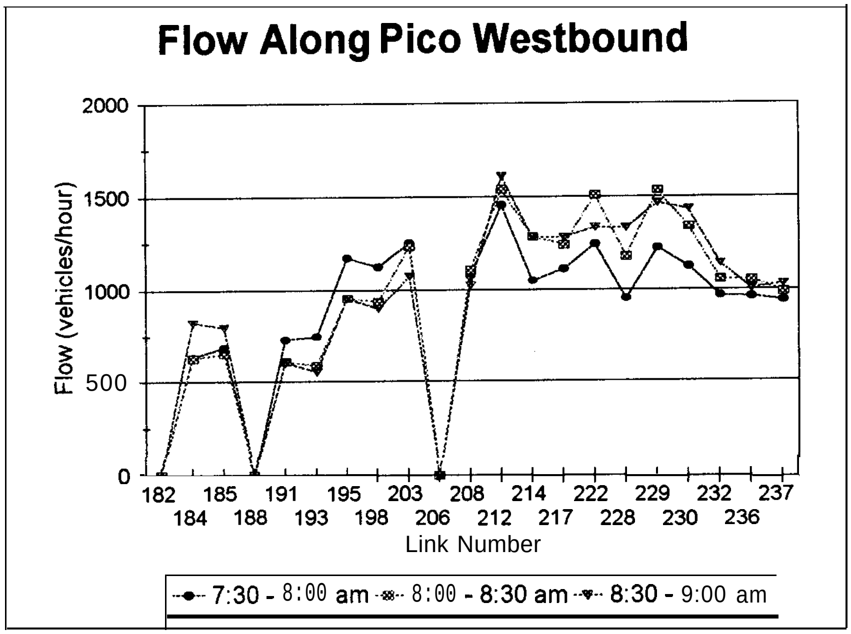

Figure 4.18: Link Flow Data for Pico Westbound

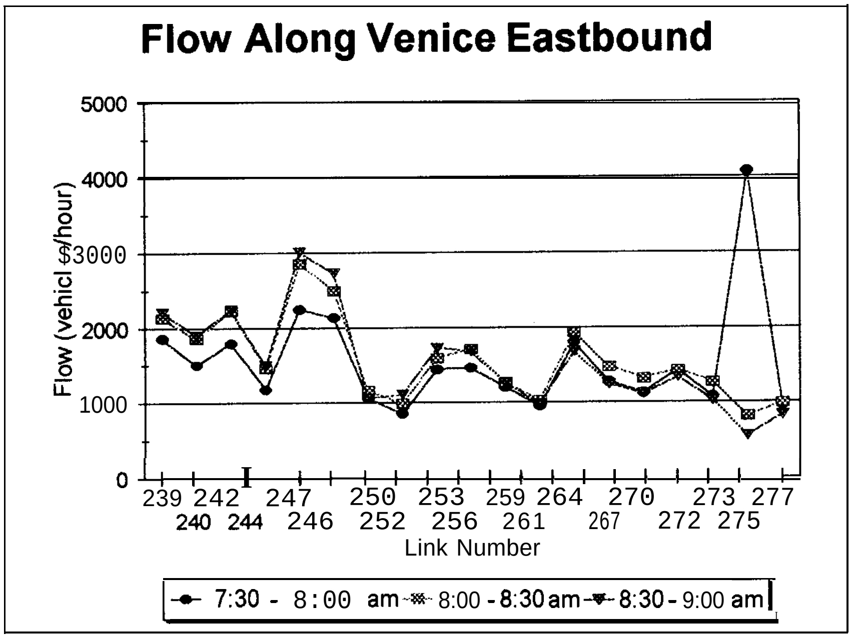

Figure 4.19: Link Flow Data for Venice Eastbound

It is easy to see from these figures which link flow values are unreasonable. Clearly, links 182, 188, and 206 in Figure 4.18 are faulty as the flows are zero. The remaining links along Pico westbound show a reasonable progression along the arterial. For Venice eastbound, only link 275 for the 7:30 a.m. to 8:00 a.m. time slice appears to be giving erroneous data. The increase in flow at links 247 and 248 can be attributed to vehicles using Venice only for a short distance before entering the freeway.

Similar graphs were created for all of the major east/west and north/south arterials in the network. Link flow data was removed from the data set for three reasons; the flow data was zero, the data was aberrant, or there was already existing data for this link from the freeway demand data set. The latter was true for a number of on and off-ramps in the network. These links with duplicate demand data were analyzed as part of the freeway demand data analysis (see Section 4.2.4). Which demand data to use for each link was decided as part of the freeway demand analysis. Table 4.3 summarizes the results of the validation process.

Total number of arterial links with detector data	373
Minus those removed because of zero or aberrant data	48
Minus those removed because of duplication with the freeway demand data set	20
Total number of arterial links with valid detector data	305

Table 4.3: Summary of Link Flow Data Validation Process

4.3.2.3 City of Los Angeles Turning Counts

A number of manual turning count studies were conducted at various intersections in the Santa Monica Freeway corridor from 1989-1990 by the City of Los Angeles. There were a number of locations in the corridor where there were no adequate demand data. It was hoped that these counts could be used to supplement the demand data from the ATSAC data tape. However, there were a number of concerns about using the data from these counts. First, the data were from various points in time over a few years. The proper growth factors to apply to each intersection were uncertain. Secondly, the data was in the form of hourly counts, whereas the existing demand data was separated into half hour time slices. Also, this data were only collected for the peak periods; 7:00 a.m. to 10:00 a.m. and 3:00 p.m. to 6:00 p.m.

For these reasons this turning count data set was used sparingly. Only when there was a significant gap in the loop detector data discussed above or when the loop detector data was suspect, was the data from these manual counts used. Twenty links were provided with link flow data from these counts. In order to extrapolate the turning count data for the peak periods to the time slices were no data was provided (non-peak periods) and to divide the data into half hour, not hourly, counts a spreadsheet was developed.

The result of the arterial demand data collection and manipulation process was a set of files, one for each half-hour time slice, containing link flow data for approximately 325 of the arterial links in the network.

4.3.3 Arterial Control Data Collection

The arterial control data consists of the traffic control in place at each of the intersections in the network. The control may be in the form of stop signs, yield signs, or traffic signals. The designation of streets as one-way only and the presence of turning restrictions are also elements of arterial traffic control. While high-occupancy vehicle (HOV) facilities are also elements of control, none are present in the arterial network.

4.3.3.1 Sources of Arterial Control Data

Most of the data concerning the location of stop signs, yield signs, and traffic signals came from previous efforts by JHK and Associates. The location of one-way streets and the presence of turning restrictions was provided by the City of Los Angeles. There were a few instances where this information was not available from either of these two sources. In these instances, the necessary data was collected by direct visits to the site itself Direct visits to the site also ensured the accuracy of the arterial control data.

The majority of the signal timing plans in the network were provided by the City of Los Angeles' ATSAC system. The extraction and manipulation of these data are discussed in the next section. There were a few signal timing plans in the corridor that were not included on the ATSAC tape. These signals are located in the cities of Beverly Hills and Culver City. The City of Los Angeles was able to provide the timing plans for those located in Culver City and the City of Beverly Hills provided the signal timing plans for the signals within its jurisdiction. The data for this additional group of signals was manually added to the final list of timing plans obtained from the ATSAC data.

4.3.3.2 Manipulation of Arterial Control Data

Referring back to Figure 4.13, the manipulation of the control data from the binary data tape received from the City of Los Angeles' ATSAC system is shown on the left side of the figure. The first step of the data manipulation process was to convert the null characters into carriage returns so that the control data set from the ATSAC data tape could be viewed in a traditional text editor. The program SIGNAL was written to accomplish this. The source code and a description of the program are included in Appendix L. In retrospect, this was not a necessary step in the processing of the data.

The next step was to convert the raw control data into a format that did not contain any extraneous data and was easily ready for conversion into an input file for the INTEGRATION program. The program SIG-REDC converts the raw control data into this format. The source code and a description of the program are included in Appendix M. The program works by reading the ASCII readable signal data file and removing the extraneous data and eliminating 35 of the 39 signal timing plans for each signalized intersection. The original data tape contains a total of 39 signal timing plans. Most of these timing plans are rarely used, only the first four signal timing plans are used on a regular basis and were considered in this study. The operating times of these four timing plans throughout the day are displayed in Table 4.4. At this point, the data has been converted into a format that contains only the necessary data. This data, the output from the SIG-REDC program, is shown in Figure 4.20. At this point there is no data entered for lost time at the intersection. The output from this program is used as input to the TO-INT3 program, which is discussed in Chapter 5.

Plan Number	Time of Operation
1	12:00 a.m.- 6:00 a.m.
2	6:00 a.m.- 10:00 a.m.
3	10:00 a.m.- 3:00 p.m.
4	3:00 p.m. - 7:00 p.m.
1	7:00 p.m. - 12:00 a.m.

Table 4.4: ATSAC Signal Timing Plan Implementation

```
Signal 2
Plan 1  2  60  48 1 33 22 2 27 27
Plan 2  2  90  41 1 60 22 2 30 27
Plan 3  2  90  39 1 60 22 2 30 27
Plan 4  2  90  52 1 60 22 2 30 27
Signal 3
Plan 1  2  60  48 1 30 27 2 30 29
Plan 2  2  90  51 1 40 27 2 50 29
Plan 3  2  90  51 1 40 27 2 50 29
Plan 4  2  90  51 1 40 27 2 50 29
   ...
```

For lines beginning with "Signal"
1st column - Signal number
For lines beginning with "Plan"
1st column - Signal timing plan number
2nd column - Number of phases in the plan
3rd column - Cycle length
4th column - Offset
5th column - Phase number
6th column - Green time for this phase
7th column - Minimum time for this phase
Repeats 5th through 7th column for each phase

Figure 4.20: Sample Output from SIG-REDC Program

4.3.4 Arterial Data Spreadsheet Summary

This section discusses the final format of the arterial data after the initial stage of manipulation. The supply data is stored in a spreadsheet while the demand and control data are stored in ASCII text files. The data stored in the spreadsheet and text files were used to generate the INTEGRATION files as discussed in Chapter 5.

The link and node data for the network was stored in a spreadsheet similar to the one created for the freeway supply data. The connection between the node and link portions of the spreadsheet is identical to that within the freeway supply data spreadsheet. The link file entries for starting and ending node are referenced to the corresponding nodes. This allows for the link and node numbers to be updated so that an organized numbering system may be preserved. This was necessary as many arterial links were split and fused together throughout the process of coding the arterial network.

The one additional reference that was made to the arterial link and node spreadsheet was the inclusion of data on which traffic signal controls which link and the signal phases in which the link has green time. While. the actual signal timing plans are still contained in a separate file, in INTEGRATION the location of traffic signals in the network is defined in the link file. A link controlled by a traffic signal is given a signal number that refers to the signal at the downstream end of the link that restricts vehicles from exiting the link. The signal number of each traffic signal was entered in the node data portion of the spreadsheet but is not exported as part of the actual INTEGRATION node file. By referencing the appropriate portion of the node data, a link could be assigned a signal number based on the node at the end of the link. Again, any changes to the link file could be made while ensuring that the appropriate signal was assigned to the appropriate link. The phases of the signal had to be added manually to the spreadsheet.

The end result of the demand data collection is a set of text files, one for each half hour time slice. Each file simply contains a list of the links for which there is detector data and the flow, in vehicles per hour, for that link for the given time slice. This text file is the output of the MAISEFLOW which is modified to include data from the City of Los Angeles turning count data.

The arterial control data actually ends up in two separate locations. The location of stop signs, yield signs, and one-way streets is coded with the link data in the supply data spreadsheet. The designation of which signal controls which link, and what phase(s) each link may discharge during, is also contained in a spreadsheet. The actual signal timing plans are contained in a text file, the format of which was shown in Figure 4.20. This file is output from the SIG-REDC program and modified to include signal timing plans that were not included in the City of Los Angeles ATSAC data tape. The complete arterial control data text file is contained in Appendix N.

Chapter 5: PREPARATION OF MODEL INPUT

Chapter 4 discussed the collection and manipulation of the various network data. Chapter 5 discusses the process of converting the manipulated data into input files for the INTEGRATION model, and one of its supporting models, QUEENSOD. As mentioned, the dividing line between data manipulation and preparation of model input is somewhat arbitrary. The coding of the network discussed below actually began with the entry of the data into the various databases for the network, as discussed in Chapter 4.

As in Chapter 4, the preparation of model input is divided into supply, demand, and control data. Section 5.1 discusses the coding of supply data for INTEGRATION. Section 5.2 discusses the coding of demand data and Section 5.3 discusses the coding of control data.

5.1 INTEGRATION SUPPLY CODING

Of the five input files of INTEGRATION, two are of specific concern to coding the supply aspects of the network, the node file (input file number one) and the link file (input file number two). All of the input files for INTEGRATION are ASCII files that may be edited using a standard text editor. The node and link files were created by simply exporting specific portions of the supply data spreadsheet described in Chapter 4 to an ASCII file. The modifications required in these exported ASCII files to form the INTEGRATION node and link files is presented in this section.

The process of creating the final node and link files was long and involved. Since the INTEGRATION demand coding (refer to Section 5.2) and testing stage (refer to Section 6.3) required coding and calibrating the freeway only section of the network, this portion of the network was coded independently from the arterial portion. Once both the arterial and freeway portions were accurately coded, they were merged to form a network of the complete freeway corridor.

There were actually three stages of development in coding the complete freeway corridor. These stages were identified as the small, medium, and large network. The small network was created from all of the links that were digitized using the U.S.G.S. maps. No serious coding or simulation was performed using this network. The medium network was created by adding all of the smaller arterials that were desired in the coding of the network but not digitized. The supply data spreadsheet created in the data collection stage of the project contained the supply data for the medium network. Finally, the creation of the large network involved expanding many of the signalized intersections in the network. This section discusses the processes that led to the final node and link files that were used to begin the modal testing and calibration stages of the project.

5.1.1 Coding of the Freeway Section of the Network

The coding of the freeway section of the network began with the entry of data into the freeway supply spreadsheet, as discussed in Chapter 4. The data in this spreadsheet determine the placement of nodes and links and many of their characteristics. The exported ASCII file from this spreadsheet with minor modifications are the node and link files for the freeway portion of the freeway corridor. Only the modifications to the freeway data required to form the node and link files are presented here.

In coding an INTEGRATION network, the user must specify which nodes are origins and/or destinations. A data field in the node file specifies the node type for each node. Also, the destination nodes may be macro clustered to reduce the minimum path tree calculation time. A macro cluster represents a group of destination zones which have been aggregated. With destinations macro clustered, minimum path trees are built for only one zone within the macro cluster, and all vehicles which are destined for any zone within the macro cluster will be routed along the same routes to the same destination node. Macro clustering was not done for the freeway network because of its smaller size.

Ramps are coded as links connected to the mainline freeway. The point where an on-ramp link intersects a freeway link is a merge point and the point where an off-ramp link intersects a freeway link is a diverge point. Note that in most cases, ramps for the Santa Monica Freeway do not contain a constant number of lanes. For most of the on-ramps, there is a two lane section (which includes an HOV bypass lane) which merges to a single lane (after the ramp meter) before the freeway. Similarly, most of the off-ramps permit a single lane of traffic to exit the freeway, which widens to two or three lanes before the off-ramp reaches the surface street intersection (where separate turn lanes are usually provided at the traffic signal). The model was coded using a separate link for each section. of an on-ramp or off-ramp containing a given number of lanes. Nodes on the ramps were used as locations where there are changes in the number of lanes. The first freeway link in each direction is the mainline entry to the freeway, and the last freeway link in each direction is the mainline exit from the freeway. The collector/distributor lanes of the freeway were coded as additional freeway lanes with a reduced capacity.

Figure 5.1 shows the final number of links and nodes required to code the freeway portion of the Santa Monica Freeway corridor. Simulation runs were performed using this network. A discussion of the results of these simulation runs is discussed in Section 6.3 and contained in the report by Bloomberg and May [9].

5.1.2 Coding of the Arterial Section of the Network

As with the freeway portion of the network, the coding process began with the entry of data into the supply spreadsheet, as discussed in Chapter 4. The exported ASCII files from this spreadsheet were ready for use with the INTEGRATION model with only minor modifications. The modified exported ASCII files are the node and link files for the arterial portion of the freeway corridor.

The nodes that serve as origins and/or destinations are coded as the terminals of certain arterial links. For the most part, an arterial link that serves as a gateway into or out of the network has a single terminal node to represent the *external* origin and the destination point. Vehicles entering the network from these locations proceed directly to the nearest

Feature	EB	WB	Total
Mainline links	42	43	85
On-ramp links	33	28	61
Total on-ramps	16	14	30
Off-ramp links	24	29	53
Total off-ramps	15	15	30
Total liis	99	100	199
Origin nodes	17	15	32
Destination nodes	16	16	32
Total nodes	100	101	201

Figure 5.1: Summary Statistics for Freeway Portion of the Freeway Corridor

intersection on that arterial. *Internal* origins and destinations are also coded using a single node to represent the origin and the destination. The links connecting the internal zones to the rest of the network connect to mid-block locations as opposed to directly at an intersection itself

At this stage in the coding process, the freeway ramps are not included in the network. Only the nodes where they will join with the arterial streets are present. There are no origins and/or destinations coded at these nodes where vehicles will enter and exit to the freeway portion of the corridor.

For the arterial network, the destination nodes in the network were macro clustered. As noted, macro clustering reduces the time required for minimum path tree calculations. However, computer memory constraints limited the number of destination macro clusters that could be used. The version of INTEGRATION used for this project was capable of handling up to 45 destination macro clusters, while there were over 100 destinations in the network. Macro clustering involved combining a group of up to four destination zones into a single macro cluster.

The decision of which destination nodes to include in a given macro cluster was based on the potential for improper routing. First, one selects the destination node which will be considered the macro destination. All minimum path tree calculations further than a given distance from the macro cluster zone will be calculated for all of the destinations within that macro cluster using the macro destination as the destination node. One must insure that all of the logical paths for the zones in the cluster are the same for links in close proximity to the macro cluster. One example of improper macro clustering is where given destinations in the cluster would involve vehicles exiting from different freeway ramps. If the off-ramp location is a significant distance from the macro cluster, all vehicles destined for a destination within this cluster will use the same ramp which is not desired.

Figure 5.2 shows the final number of links, nodes, and zones required to code the arterial portion of the medium network for the Santa Monica Freeway corridor.

Feature	Total
Arterial links	1,060
Arterial nodes	550
External Origin Zones	48
Internal Origin Zones	57
Total Origin Zones	105
External Destination Zones	48
Internal Destination Zones	57
Total Destination Zones	105

Figure 5.2: Summary Statistics for Arterial Portion of the Freeway Corridor

5.1.3 Combining the Freeway and Arterial Supply Data

The next step in the process of creating the corridor network was to combine the freeway and arterial networks into a single network representing the entire freeway corridor. The process involved combining the arterial node and link data and the freeway node and link data into single node and link files.

As mentioned, common x and y coordinates were developed for the nodes where the arterial and freeway networks meet, namely the nodes that connect freeway ramps and arterials. These nodes exist in both networks and have the same x and y coordinates in each network. In the freeway network, these nodes are origins and/or destinations while they are neither in the arterial network. Once the node files were joined, one set of these common nodes, the freeway network set, was eliminated.

All of the freeway links that referenced one of these common nodes as its upstream or downstream node, had to be modified so that it referenced the corresponding node from the arterial network. The remaining nodes from the freeway network are unique to the freeway network and were not eliminated.

Once the links referenced all of the correct nodes, the links and nodes had to be renumbered to avoid duplicate identification numbers and to establish an orderly numbering system. The additional freeway links were simply numbered sequentially starting from the highest link number in the arterial network. Since the link identification numbers are not referenced in any way, this proved adequate and also allowed for the same orderly numbering scheme from the freeway network to be preserved (i.e. all of the eastbound mainline freeway links proceed sequentially from link number 1176 to 1219).

The numbering of the nodes required additional consideration. The maximum zone number allowed with the version of the INTEGRATION-program used for this project was substantially less than the maximum node number allowed. Thus, it is recommended that the nodes be numbered with the zones first and the other nodes last. Most of the zones from the freeway-only network were eliminated as zones, since off-ramp and on-ramp links do not enter or exit the freeway corridor network. However, the four mainline freeway ends and the ends of the eight freeway connector links will serve as zones in both networks. These freeway nodes, remaining as zones in the arterial network, had to given node numbers lower than the allowable number of zones. The remaining freeway nodes were simply assigned new identification numbers that were not used by the arterial nodes. The use of links referencing node numbers in the supply spreadsheet allowed for the simple reassignment of node identification numbers to accommodate any necessary changes. The links automatically referenced the correct node.

With the freeway and arterial network properly combined, the network was plotted using the INTMAP program to check for accuracy. Often gross errors in the supply coding, such as incorrect x and y coordinates or improperly connected links, are easily evident with a plot of the network, but are difficult to discern from the ASCII files themselves. Using the AutoCAD drawing created from the INTMAP program, the freeway/arterial interface was plotted in great detail.

Magnified printouts of the network from the INTMAP program were also used to make numerous changes to the x and y coordinates of many nodes for aesthetic reasons. Using the actual x and y node coordinates, most of the nodes at a given freeway interchange would be too close to one another to be discernible when viewing the entire network. For this reason, the nodes at the freeway interchanges were displayed in such a manner that the interchange was evident when viewing the entire network.

Figure 5.3 provides the summary statistics for the medium network. A plot of the medium network created using the INTMAP program is shown in Figure 5.4. The dashed lines in this figure represent links that are connected to a node that is an origin and/or a destination. The Santa Monica Freeway proceeds from left to right near the bottom of the figure. The diamond interchanges in the eastern half of the network are easily recognized.

Feature	Total
Mainline freeway links	85
Arterial links	1,060
Zone connector links	314
Total links	1629
Origin nodes	111
Destination nodes	111
Total zones	118
Total nodes	691

Figure 5.3: Summary Statistics for the Medium Network

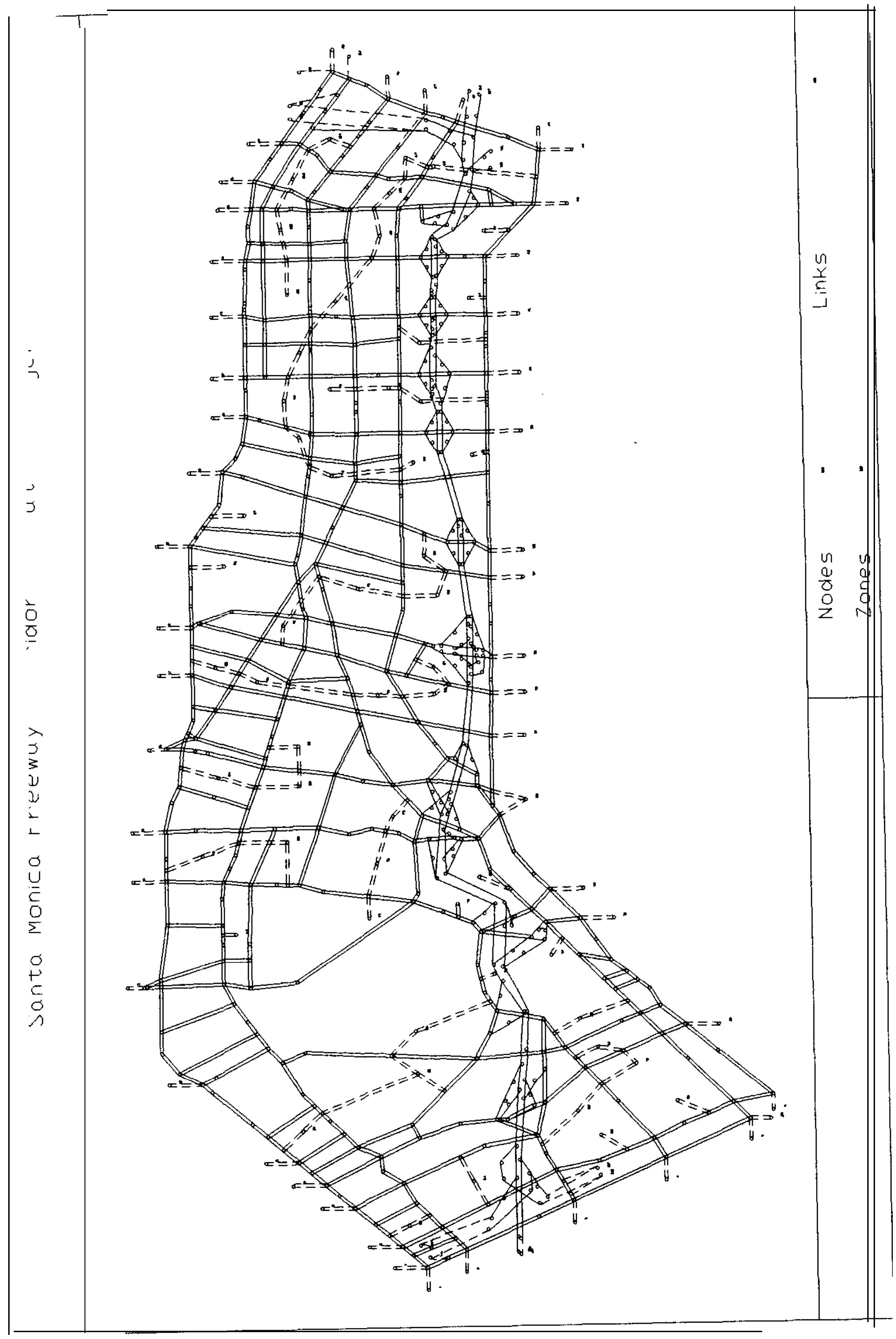

Figure 5.4: Plot of the Medium Network from the INTMAP Program

5.1.4 Expansion from the Medium to the Large Network

A few simulation runs were conducted using the medium network. However, it was quickly discovered that the one node signalized intersection configuration resulted in the creation of unrealistically long queues developing at many of the intersections in the network. The reason for this is that with this one node signalized intersection configuration, all of the movements at an intersection (left, through and right) exit the intersection from the same link. INTEGRATION does not distinguish between the individual lanes of an intersection approach. If there is delay to one of these movements in exiting the intersection, all of the movements will be delayed since the vehicles are queued in a FIFO stack.

In reality, the delay to the through and right turning movements at a signalized intersection will usually be smaller than the delay for the left-turning vehicles. However, the one node signalized intersection configuration did not allow for the coding of a left-turning movement which is delayed by an opposing vehicle flow. Thus, the left-turning movements not being penalized in the medium network resulted in unrealistic vehicle routing. The single node intersection configuration also requires that all movements of the intersection discharge the intersection during the same signal timing phase.

The solution to this problem was presented in the Ph.D. dissertation of Michel Van Aerde [57] at the time of the development of one of the earliest versions of the INTEGRATION model. It involves expanding an intersection from a single node configuration into an eight node/twelve link configuration, Figure 5.5 presents the two types of intersection configuration

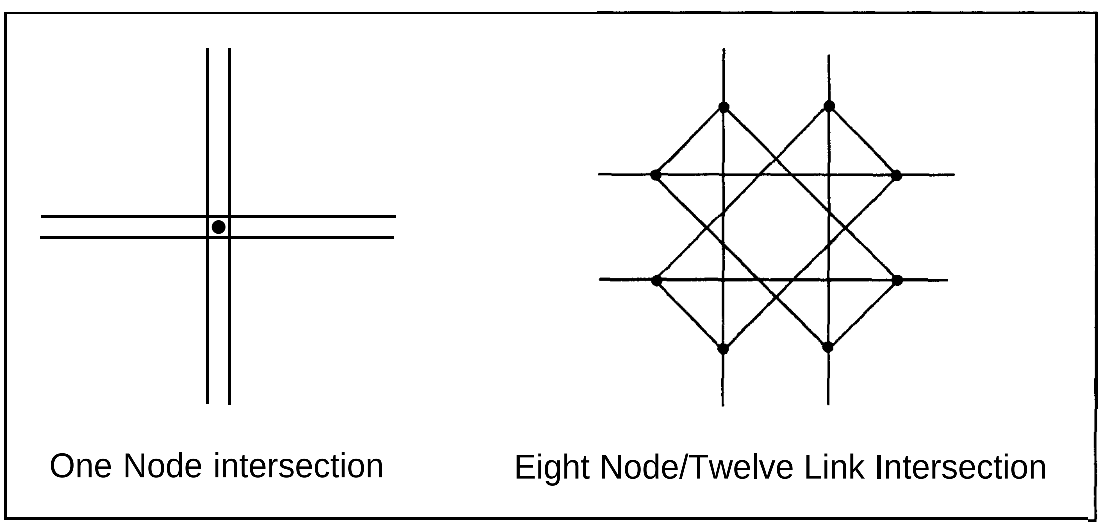

Figure 5.5: Unexpanded and Expanded Intersection Configurations

There are advantages and disadvantages to expanding an intersection to the eight node\twelve link configuration shown above. The advantages are, of course, a more realistic simulation of a signalized intersection. The major disadvantages resulting from the increased number of nodes and links are increased memory requirements, increased coding time, and increased simulation time. Thus, a decision had to be made as to exactly which intersections should be expanded. It was decided that only intersections where the major north-south arterials intersected the major north-south arterials and all intersections that were close to the freeway (including all intersections which connect the arterial portion of the network to the freeway portion of the network) would be expanded. Of course, signalized intersections where one of the cross streets is not present in the network could not be expanded. A total of 167 of the 3 12 signalized intersections in the network were expanded.

The expansion process was accomplished by the development of a program entitled INTGEN. This program reads the medium network (unexpanded) link and node files and files that contain parameters for the intersection expansion process (APPROACH.DAT and EXPLODE.DAT). The program then creates new link and node files that contain the network with the expanded intersections, referred to as the large network. A complete description of this program and the source code are included in Appendix 0.

The program eliminates the one node signalized intersection configuration and creates the eight nodes of a four-legged expanded intersection. The x and y coordinates for these nodes are assigned so as to create the geometry shown in Figure 5.5. Twelve new links are added to the network for each intersection expanded to represent the turning movements within the intersection, three links per approach. Only six nodes and six links are created for a three-legged intersection. The program reassigns the existing links to the appropriate new nodes and assigns the new links to the appropriate nodes.

The tile APPROACH.DAT contains the information necessary to complete the coding of the link characteristics within the expanded intersection. Figure 5.6 shows a portion of this file along with an explanation of the data in the file.

The APPROACH.DAT is very robust, in that it is designed to allow the coding of any type of intersection approach. The user may add additional approach types, or alter the default values as desired. The file EXPLODE.DAT contains the information required for exploding a given network. A portion of this file and an explanation of the data within it are given in Figure 5.7. The complete APPROACH.DAT and EXPLODE.DAT files, as well as a more thorough explanation of the various data fields, are provided in Appendix 0.

The program produces three files, the expanded node file, the expanded link file, and a file entitled RESULTS.OUT. The RESULTS.OUT warns the user if any errors have been detected in running the program.

```
67
1  4   0.1 0.1 0.1 3.0 1.0 1.0 1600 1200 1400 80 60 70 0 *comment
2  2   0.1 0.1 0.1 2.0 0.0 1.0 1600    0 1400 80  0 70 0 *comment
3  4   0.1 0.1 0.1 3.0 1.0 0.0 1600 1200    0 80 60  0 0 *comment
4  2   0.1 0.1 0.1 1.0 1.0 1.0 1600 1200 1400 80 60 70 0 *comment
5  3   0.1 0.1 0.1 2.0 1.0 1.0 1600 1200 1400 80 60 70 0 *comment
```

```
First line;
  The number of approach types described in the file.
Remaining lines;
  1st column - Approach type number (sequential)
  2nd column - Number of lanes at the approach
  3rd, 4th and 5th columns - Lengths of the through,
    left and right links respectively (kilometers)
  6th, 7th, and 8th columns - Number of effective
    lanes for the through, left and right links
  9th, 10th, and 11th columns - capacities of the
    through, left and right links (vphpl)
  12th, 13th and 14th columns - free flow speed for
    the through, left and right links (km/hr)
  15th column - flag to indicate if a protected left
    is present at this approach
  Remaining columns - a star followed by a comment
    to describe the approach type i.e. "*3 thru,
    1Xleft, 1 shared right"
```

Figure 5.6: Portion of the APPROACH.DAT Input File

```
167
538    5  5  5 16  587  685  595  693  596  694  588  686
424   55 16 55 11  285  744  359  748  360 1137  286  745
347   15    15       0    0    0    0    0    0    0    0
323    1  4  1  5    0    0    0    0    0    0    0    0
516    5  5  5  5    0    0    0    0    0    0    0    0
```

```
First line;
  The number of intersections to be expanded
Remaining lines;
  1st column - node number for the intersection
  2nd through 5th column - approach types for the
    four approaches at the intersection
  Remaining columns - (optional) specification of
    the eight links that connect with the intersection
```

Figure 5.7: Portion of the EXPLODE.DAT Input File

5.1.5 Final Preparation of the Node and Link Files

Significant modifications were made to the node and link files that were prepared by the INTGEN program. First, there were several turning restrictions that needed to be coded into the network. In instances where the left-turning movement is not allowed at all, the left-turn link was removed from the network. Turning restrictions are technically an element of control data and are discussed more thoroughly in Section 5.3.2.

During the calibration stage of the network, it was discovered that the expansion of the intersections allowed for certain illogical routing trees to be created. In particular, if the delay to a through movement at an intersection were large enough, the minimum path tree for through moving vehicles might consist of a right-turn, a U-turn, and another right-turn. Normally, the INTEGRATION model does not allow for U-turns to be made. However, with the intersection expanded, there are two nodes for each intersection approach. Thus, the model does not recognize this movement as a U-turn. To eliminate this problem, all of the potential U-turn movements were made illegal using the turning restriction feature.

Figure 5.8 displays the summary statistics for the large network with 167 expanded intersections.

Feature	Total
Mainline freeway links	85
Arterial links	1,060
Expanded intersection links	1,672
Zone connector links	314
Total links	3,286
Origin nodes	111
Destination nodes	111
Total zones	118
Total nodes	1,747

Figure 5.8: Summary Statistics for the Large Network

5.1.6 INTEGRATION Node and Link Files

The end result of the supply coding process was the complete node and link files that were used to begin the calibration stage of the project. Figure 5.9 lists a portion of the complete node file along with a description of the various data fields. The complete node tile is listed in Appendix P.

Figure 5.10 contains a portion of the final link file along with a description of the various data fields. The complete link file is listed in Appendix Q.

The large network was plotted using the INTMAP program. Figure 5.11 shows the network as plotted by this program.

```
INTEGRATION Node File For Santa Monica Freeway Corridor
1747  1.0 1.0
    1   9.774  13.896  1    -4  0  comment
    2  10.160  13.310  1     4  0  comment
    3  11.226  11.680  1     2  0  comment
    4  12.130  10.299  1     2  0  comment
    5  12.997   9.036  1     1  0  comment
    6  13.650   8.700  1     1  0  comment
```

First line;
- Comment

Second line;
- Number of nodes
- X coordinate scaling factor
- Y coordinate scaling factor

All subsequent lines;
- Node identification number
- X coordinate
- Y coordinate
- Node type (origin, destination, neither or both)
- Cluster (used in minimum path tree calculations)
- Changeable message sign flag
- Comment (optional)

Figure 5.9: Portion of INTEGRATION Node File

```
New INTEGRATION Link File -created by INTGEN.C
3286 1.0 1.0 1.0 1.0 1.0
 1    1 1401 0.400 -1 2000 5 0.5  8  131  0  0     0   00   0 0 0 0 1
 2 1407 301 0.215 70 2000 3 0.5 42 131 133  0 50400 0 0 273 1 0 0 1
 3 301 1891 0.076 70 2000 3 0.5 42 131 0 0     0   00   0 0 0 0 1
 4 1897  158 0.186 70 2000 3 0.5 42 131 131  0 50400 0 0   0 0 0 0 1
 5 158 303 0.186 70 2000 3 0.5 42 131 0 0     0   0 0 2 7 5 1 0 0 1
 6 303 1309 0.221 70 2000 3 0.5 42 131   0 0     0   00   0 0 0 0 1
```

First line:
Title line
Second Line:
Number of links
Five various scaling factors
All subsequent lines:
Link identification number
Start node
End node
Link length (km.)
Free flow speed (km./hr.)
Capacity (veh./hr./lane)
Number of lanes
Platoon dispersion factor
Speed at capacity (km./hr.)
Jam Density (veh./hr./lane)
Link with turning restrictions
Start time of turn restriction
End time of turning restriction
Opposing link number 1
Opposing link number 2
Signal number controlling this link
Signal phase number 1 for this link
Signal phase number 2 for this link
High occupancy vehicle link flag
Surveillance flag
Comment (optional)

Figure 5.10: Portion of INTEGRATION Link File

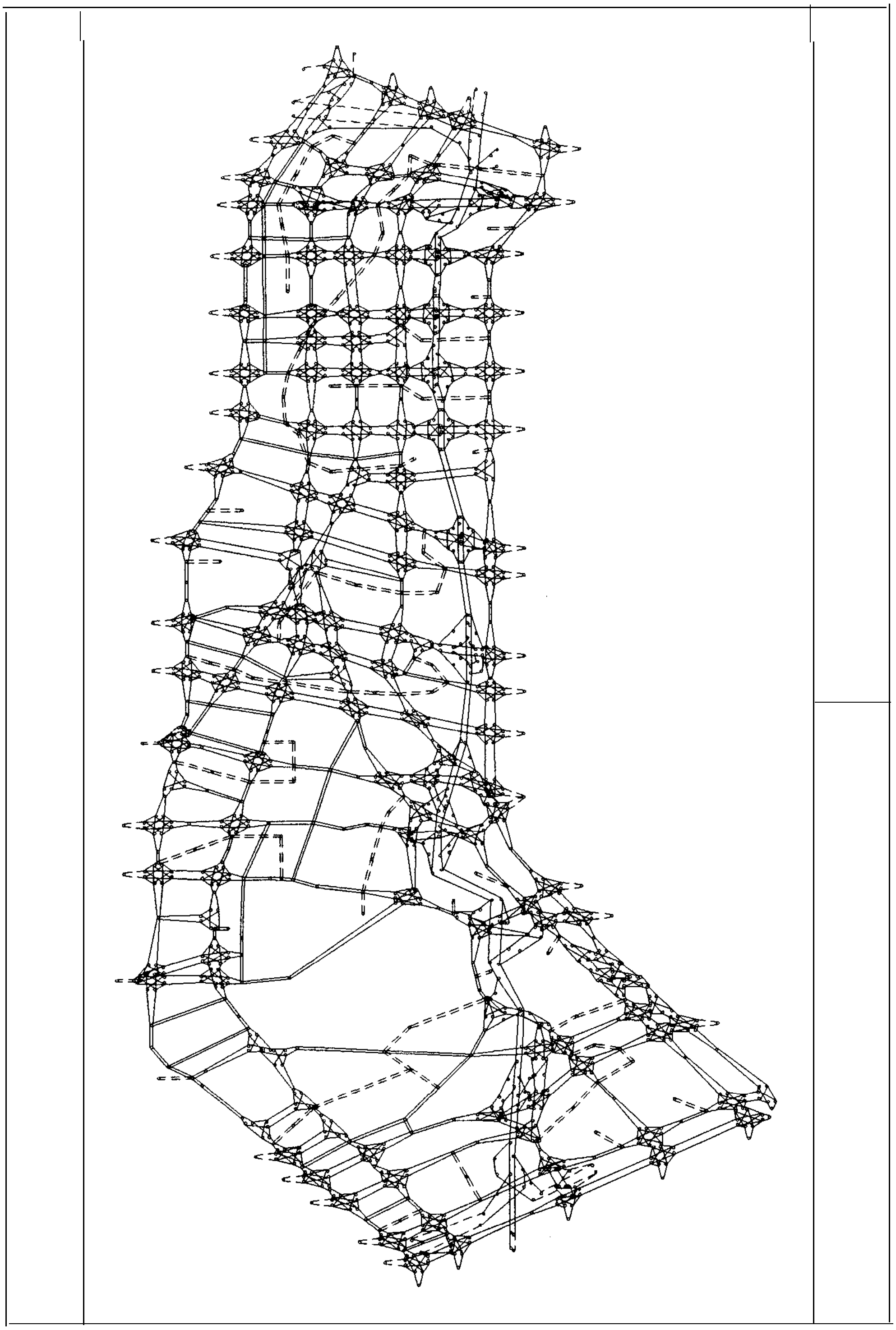

Figure 5.11: Plot of the Large Network Using the INTMAP Program

5.2 INTEGRATION DEMAND CODING

The final product of the demand coding process for INTEGRATION was the creation of input tile number four, the origin/destination file. The origin/destination file describes the traffic demand from each origin to each destination in the network for each time slice. This process involved using the QUEENSOD program and the ASSIGN feature of the INTEGRATION model. This section begins by discussing the creation of the input files used with the QUEENSOD program. Then, the iterative process, referred to as the demand estimation process, of using the QUEENSOD program and the ASSIGN submodule together to generate an origin/destination table for each time slice is presented.

The QUEENSOD program estimates an origin/destination matrix for a given time slice based on the observed link flows and driver's route choices (as described by the minimum path routing trees) corresponding to the specific time slice. The QUEENSOD program estimation methodology begins with a seed origin/destination matrix, which may be user specified, and successively modifies it such that the mean of the absolute squared difference between the observed link flows and estimated link flows is minimized. The QUEENSOD program estimates the link flows by instantaneously propagating the origin/destination demand estimated for the current iteration along all links as defined by the user supplied minimum path routing trees. Refer to the QUEENSOD User's Guide [25, 26] for a more detailed discussion of the origin/destination estimation methodology. Recall that the ASSIGN feature of INTEGRATION generates minimum path trees from each origin to each destination based on an origin/destination table for each time slice.

The input files used by the QUEENSOD program are the node description file (QUEENSOD input file one), link description file (QUEENSOD input file two), seed origin/destination demands (QUEENSOD input file four), observed link traffic flows (QUEENSOD input file five), minimum path routing trees (QUEENSOD input file six), and the link flow reliability factors (QUEENSOD input file seven). The node and link file for the QUEENSOD program, are compatible with the corresponding files for the INTEGRATION program. Thus, the final node and link files prepared for use with the INTEGRATION model (discussed in the previous section) could be used without modification. The creation of the seed origin/destination demands is discussed in Section 5.2.1. The procedure used to generate the observed link flow values is discussed in Section 5.2.2. The creation of QUEENSOD input file number six, the minimum path tree file, is presented in Section 5.2.3. The use of the link flow reliability factors file is discussed in Section 5.2.4. Finally, section 5.2.5 presents the demand estimation process followed to derive an origin/destination table for each time slice.

5.2.1 Coding the Seed Origin/Destination File of QUEENSOD

QUEENSOD input file number four contains the seed origin/destination matrix file, the initial origin/destination demand matrix that the QUEENSOD program successively modifies in order to minimize the mean squared link flow error (the mean of the squared difference between the observed link flows and estimated link flows). Each iteration of the program produces a new origin/destination matrix. Modifications to this matrix are made at every iteration in an effort to reduce the mean squared link flow error. This input file is optional. The information contained within the seed origin/destination matrix can vary from no knowledge of the trip interactions to perfect knowledge of the actual demands. If the user does not specify a file, the program creates a seed origin/destination matrix where all the possible origin/destination pairs are assigned a demand of 1,000 vehicles per hour.

It was discovered that using the default seed origin/destination values for this network was not appropriate, The observed link flows on the arterial portion of the network were fairly sparse (only 323 of 1,060 arterial links had observed link flows). With sparse observed link flows on the arterial portion of the network the seed origin/destination demand between an origin and a destination pair may only be slightly modified if the true link flows between this origin/destination pair are unknown. Sparse observed link flows resulting in little modification to the seed origin/destination demand matrix mean the final origin/destination table estimated by the QUEENSOD program is heavily dependent on the seed origin/destination demands. Therefore, allowing the QUEENSOD program to specify a demand of 1,000 vehicles per hour between all origin pairs as a seed origin/destination demand matrix will result in the generation of an unrealistic final origin/destination table. Thus, this seed origin/destination file had to be created as part of the simulation effort.

A program was developed, TOSEED, that creates a seed origin/destination file automatically. A complete description of this program and the source code is included in Appendix R. The inputs to the TOSEED program are the INTEGRATION node file and a file that contains the link flows into and out of each zone. In some instances, the link flows in to and out of the zones had to be estimated.

The availability of the link flows into and out of each zone is illustrated in Figure 5.12. For the external zones only 22 of the 48 zones had links connecting the zone to the network with observed flow. There were no observed link flows for links connecting the network to the external zones. For the external zones when the link flow out of the zone (26 links) and the link flow into the zone (48 links) was not known, the value was estimated by considering other arterial link flows in proximity to the zone and flows at other points on the same arterial, For the internal zones, there were no instances were flows in and out of the zone were known. Since the internal zones are all associated with a given census tract, origin and destination demand data was obtained from the 1990 US Census and the Southern California Association of Governments (SCAG) concerning the number of residents and employees in each census tract. Equations from the National Cooperative Highway Research Program (NCHRP) Report Number 187 [48] were used to estimate the traffic flow into and out of each internal zone based on this data.

Link Type	Origin	Destination
Links connecting an external zone		
Observed flow	22	0
Estimated flow	26	48
Link connecting an internal zone		
Observed flow	0	0
Estimated flow	56	56

Figure 5.12: Availability of Link Flows Out of and Into Zones

The TOSEED program uses the value of the out flow from a specific origin and the in flow to a specific destination, either observed or estimated, to estimate the demand from each origin to each destination for all origin/destination pairs in the network. The user may set certain parameters of the program that determine which equations will be used in calculating the final seed demand for each origin/destination pair. Refer to Appendix R for more information on these parameters.

The output from the TOSEED program was modified to reflect the scope of the network that was coded. The interchanges of the Santa Monica Freeway (I-10) with I-405 and SR110 were not coded in their entirety. Traffic that travels through the network on I-405 or SR1 10 was not considered. Traffic that travels from northbound I-405 to westbound I-10, or from southbound I-405 to westbound I-10, was also not considered. Similarly, traffic traveling from northbound SRI 10 to eastbound I-10, or from southbound SR1 10 to westbound I- 10, was not considered. Thus, any origin/destination pairs involving only one of these interchanges, and no other part of the network, were removed from the seed origin/destination file. Eliminating an origin/destination pair from the seed file ensures that no vehicles are assigned to this pair.

Trial runs with this seed origin/destination table indicated that the values used in this file had little effect on the final origin/destination table of the QUEENSOD program for some of the origin-destination pairs. However, when there were no known link, flows between an origin and a destination, the program only slightly altered the demand between the origin and destination. For some origin/destination pairs, the final demand was very similar to the initial demand value specified in the seed origin/destination file. Thus, due to the magnitude of the approximations used to develop the seed origin/destination file, the validity of the final origin/destination table may be suspect.

5.2.2 Coding the Link Flow File of QUEENSOD

QUEENSOD input file number five contains a time series of observed link flows. Observed link travel times could also be entered into the QUEENSOD program through this input file. The observed link travel times would be used by QUEENSOD to determine when demands arrive at links. The observed link travel times for the Santa Monica freeway corridor were not available, thus they were not entered into the

QUEENSOD program. The free flow travel times for each link (as calculated from the link length and free flow velocity) were entered in the place of the observed link travel times in QUEENSOD input file number five. Entering the free flow travel times instead of the observed link travel times results in QUEENSOD assuming that all the estimated link flows propagate instantaneously along their paths.

The creation of the link flow file basically required combining the link flows that were collected from the freeway network with those collected from the arterial network. However, the link flow data collected for the freeway network was fairly limited along the mainline (especially the eastern and western freeway ends) [8]. To overcome these gaps in the data the FREQ model was calibrated to the empirical data available and the link flows predicted by the FREQ model, for all freeway links in both directions, were used as the observed link flows by the QUEENSOD program in estimating the origin/destination demand table. Refer to Bloomberg and May [9] for a full discussion of the coding and calibration of the freeway only portion of the network with the FREQ model.

FREQ is perhaps the most recognized model for the simulation of freeways. The FREQ family of models were originally developed in the 1960s and have been continually refined at the Institute of Transportation Studies at UC Berkeley. FREQ is a macroscopic deterministic model that is ideally suited for freeway studies along a linear directional freeway corridor [36].

The freeway only portion of the network was coded in the FREQ model. The number of freeway links and the characteristics of these links were coded in FREQ to be identical to the freeway only portion of the Santa Monica freeway corridor coded for the INTEGRATION model, which is described in Section 5.1.1. The generation of the origin/destination demand table in FREQ was done internally based on the on-ramp and off-ramp traffic flows. Unlike the INTEGRATION model, FREQ does not consider the link flows along the freeway, just the flow entering and exiting the freeway. The on-ramp counts entered in FREQ can be accurately described as demand, but the off-ramp data definitely can not. The volume data at the off-ramps reflect the number of vehicles that passed by the ramp count station during a given time slice, not necessarily the volume that wanted to (i.e., demanded that ramp). Whenever there was congestion on the freeway system, cars were delayed and the off-ramp volumes were not a true reflection of demand. FREQ overcomes this problem in calculating an origin/destination table of demands by adjusting the off-ramp volumes using scaling factors when there is vehicle queuing on the freeway. Thus, FREQ can accurately synthesis an origin/destination table for the freeway network.

Once the freeway only portion of the Santa Monica freeway network was coded in FREQ the next step was to calibrate the FREQ model to match the available empirical data. The FREQ calibration effort compared predicted freeway performance with general knowledge of the system. Summary statistics considered were the start and end time of heavily congested periods, and the average freeway speed and maximum demand/capacity ratio for each time slice for both directions of the freeway. These general freeway summary statistics matched the performance expected based on congestion monitoring studies conducted on the Santa Monica freeway.

More specific performance data were also used for comparison in the calibration effort. The first detailed analysis was a comparison of predicted mainline flows to detector volumes. Figure 5.13 lists the mainline detectors where reasonable data were available (for more information refer to [8]). All of these detectors are referenced to a specific link (FREQ subsection) so that the output from the FREQ simulations could be compared against expected volumes from the detectors.

Eastbound Location	Mile Marker	Westbound Location	Mile Marker
Overland	6.50	Budlong	13.53
Motor	6.73	Normandie	13.21
Manning	7.27	Western 2	12.95
National	7.99	Gramercy	12.58
Venice	9.01	Arlington	12.23
Fairfax	9.21	Crenshaw	11.53
Washington	9.51	West	11.06
La Brea 1	10.23	La Brea 2	10.53
La Brea 2	10.53	Fairfax	9.21
Harcourt	10.70	Manning	7.22
West	11.00	Motor	6.73
Crenshaw	11.53		
Arlington 1	12.23		
Arlington 2	12.45		
Gramercy	12.58		
Western 2	12.95		

Figure 5.13: Mainline Detectors Used for FREQ Calibration

A sample of the results from this comparison is shown in Figure 5.14, where FREQ predicted flows are compared against observed flows for the mainline detectors at the Crenshaw (eastbound) station. In this case the comparison is very good: there is never a difference of more than 200 vehicles/hour during any half-hour time slice. However, for other detector stations, the fit was not nearly as good. Overall, it was felt that the FREQ flows matched the observed detector flows fairly accurately.

The second detailed analysis was a comparison of predicted traffic densities against detector densities. Figure 5.15 shows an excerpt from the FREQ contour diagram of traffic density for *part* of the eastbound freeway. The horizontal axis is the subsection number, scaled by length of the section (i.e., the first section is 3 characters wide and the second is 8 characters, so the length of subsection 2 is approximately 8/3 that of subsection 1). The time slices (24 half-hour periods from 6:30 A.M. to 6:30 P.M.) are on the vertical axis. The data in the chart are traffic density values, in units of 10 vehicles/mile. For example, the series of "1"s in time slice one indicate that the density of traffic on subsections 1 and 2 was between 10 and 20 vehicles/mile from 6:30 to 7:00 A.M.

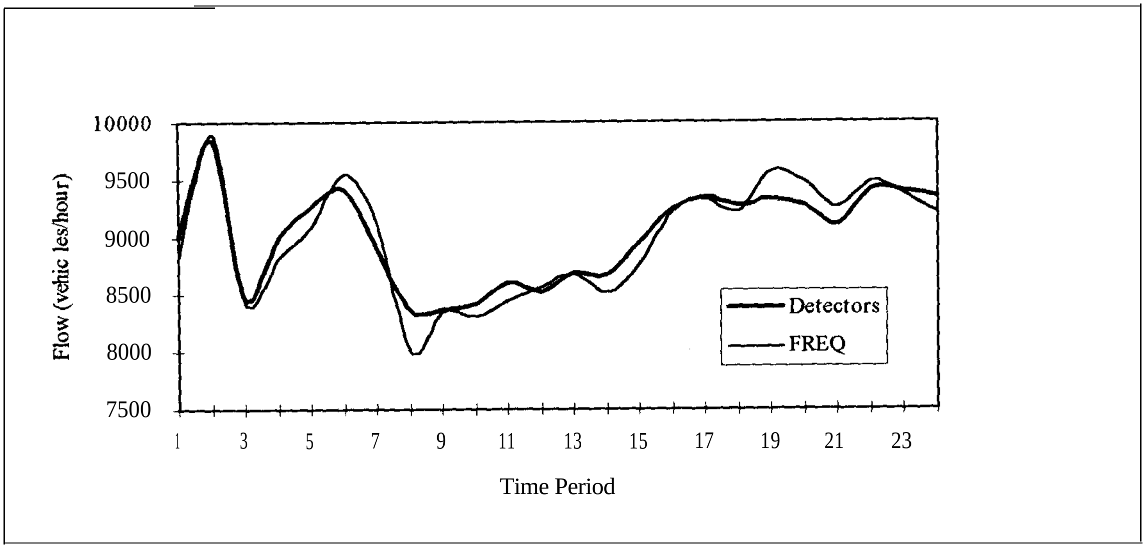

Figure 5.14: Comparison of FREQ Flows with Detector Data

These predicted densities were compared to the empirical densities, as shown in Figure 5.16. This chart was created using the available detector occupancy data (generally from the median lane, or the lane with the best available data) for any subsection with a working detector. The occupancy data were converted to density using an assumption of a constant detector length (equivalent to average vehicle length plus average detector length) of 25 feet.

Where densities of at least 50 vehicles/mile are found, the value is shown in bold and strikethrough on the figures. These figures give a rough idea of where and when congestion is present along the freeway. Note that there is a reasonable correlation between the two charts.

It was concluded from the analysis of the FREQ predicted general freeway summary statistics and the detailed analysis of the FREQ calibration that the FREQ model was accurately modeling traffic on the freeway network. The FREQ predicted link flows for all subsections could now be considered as accurate as the empirical link flows. These link flows predicted by FREQ for all the freeway links for all time slices were then entered into the freeway demand spreadsheet in the place of the empirical link flows. Thus, this enhanced freeway demand spreadsheet has an 'observed link flow' listed for every freeway link for all time slices.

The link flow file could now be generated by combining the enhanced freeway demand spreadsheet and the arterial demand spreadsheet, with minor modifications. The enhanced freeway demand spreadsheet was modified so that the link numbers for the freeway links refer to their new identification numbers in the combined freeway/arterial network. Then the link flow data from the enhanced freeway demand spreadsheet was exported to ASCII files. For each time slice one ASCII file was created. These ASCII files were created so as to be identical in format as those created for the arterial links using the MAKEFLOW program (see Section 4.3.2.2 and Figure 4.17). These ASCII files containing the enhanced freeway link flows for each time slice and the ASCII files containing the arterial link flows for each time slice (as outputted from the MAKEFLOW program) were merged

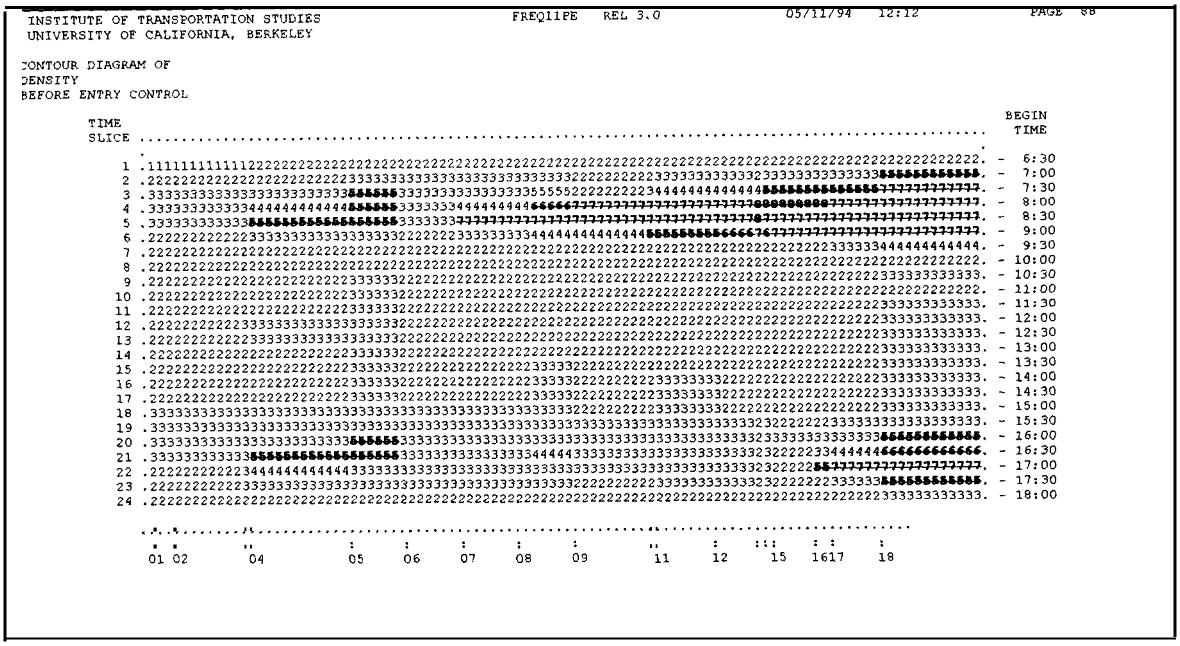

Figure 5.15: FREQ Density Contour (West Side of Eastbound Freeway)

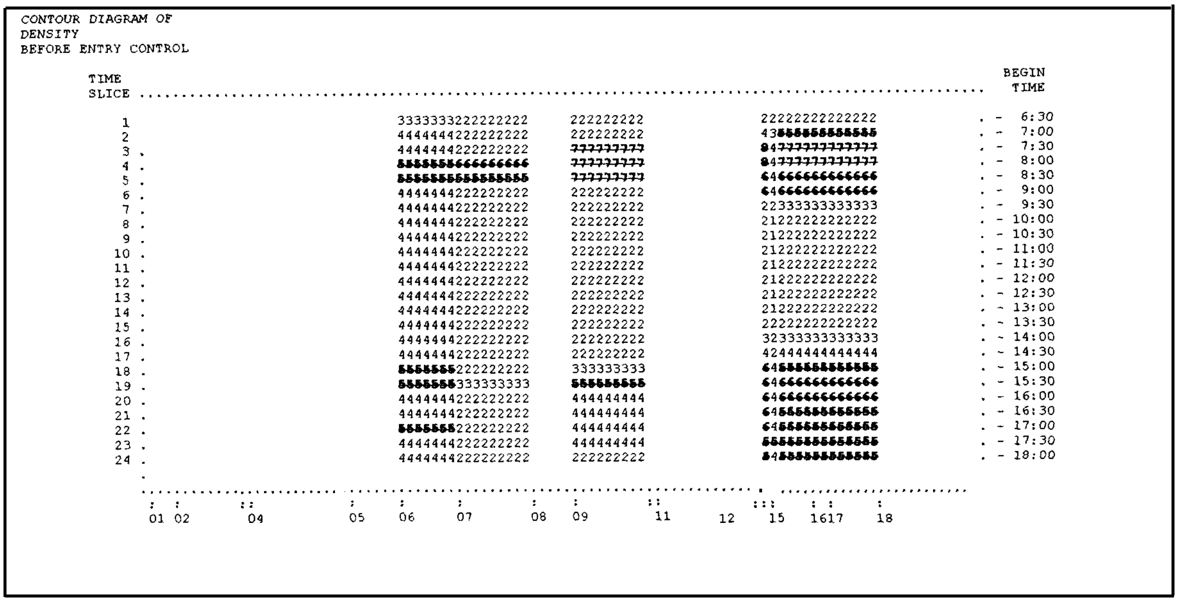

Figure 5.16: Available Empirical Densities (West Side of Eastbound Freeway)

into one file for each time slice using the DOS COPY command. Thus, there are 28 files, one for each half-hour time slice, that contain both the freeway and arterial link flows. For purposes of this project and for future reference by others, the link flows contained in these 28 files are listed in one comprehensive table which is contained in Appendix S. These files and the network link file are then used by the TO_QOD5 program to create the final QUEENSOD input file number five. The source code for this program and a complete description are contained in Appendix T.

The functioning of the TO_QOD5 program is fairly simple. All of the information required to create the QUEENSOD input file five is contained in the network link file and these observed link flow files. The TO_QOD5 program just extracts the relevant information from these two files and restructures it to create the QUEENSOD input file five. For each time slice, the program reads each link from the link file and determines if there is an associated link flow. If a link flow exists for the link that value is used. If no link flow exists in the link flow file a value of negative one is used. The program accesses other link data from the link file and produces a file that is ready for use as input file number five of the QUEENSOD program.

5.2.3 Creating the Minimum Path Tree File of QUEENSOD

The minimum path tree file, input file number six for the QUEENSOD program, is created using the ASSIGN submodule of INTEGRATION. To initiate this submodule of INTEGRATION, the user runs the INTEGRATION model with the simulation time coded as a negative number. With this feature engaged, the program will output a single set of minimum path trees as output file number **13.** These path trees represent the minimum path trees for all origin/destination pairs at the moment in which the output file was created. The time at which the set of minimum path trees is created is user specified.

The minimum path tree file can contain one single set of path trees (which represents an all-or-nothing traffic assignment) or some number of multiple path with each path being utilized by some portion of the demand. When two or more sets of trees are provided the user must specify the portion of demand (ranging from 0.0 to 1 .0) that use each path tree. The sum of the portions of demand following each set of path trees must be 1 .O (i.e., all of the demand must be allocated to the various path trees). Each time slice can have a unique set of path trees to account for changing driver routing behavior over time.

The vehicle routing trees in this file are represented by link and node trees. A tree is a matrix of values which indicate the link that a vehicle should utilize next given the current vehicle location and desired destination. Current vehicle position can be determined by node (for vehicles departing an origin) or by the link currently being traversed (for vehicles enroute to their destination).

5.2.4 Coding the Link Flow Reliability Factor File of QUEENSOD

QUEENSOD input file number seven contains information on the reliability of the observed link flows. The QUEENSOD program allows the user to specify that certain observed link flow values be treated with a higher reliability than other observed link flows in the origin/destination matrix estimation process. This feature works by modifying the correction factor that is to be applied to a specific link in minimizing the mean squared link flow error such that an observed link flow on a link with a high reliability factor will be given more weight then an observed flow on a link with a low reliability factor. The observed link flows on both the freeway and arterial portions of the Santa Monica freeway corridor were considered to be equally reliable, thus a uniform reliability factor of 1 .0 was applied to all links with observed flow. The calibration stage of the project did investigate the impact of modifying these reliability factors. Refer to Chapter 7 for a discussion of the impact of varying the link flow reliability factors.

5.2.5 The Demand Estimation Process

The QUEENSOD program estimates an origin/destination matrix for a given time slice based on the observed link flows and the minimum path trees. The development of the observed link flows was detailed in Section 5.2.2. The minimum path trees are generated by the ASSIGN submodule of INTEGRATION as discussed in Section 5.2.3. However, the ASSIGN submodule requires an origin/destination demand matrix to assign traffic to the network in order to estimate a minimum path tree. Thus, an iterative process between QUEENSOD and INTEGRATION was required. This iterative process is referred to as the demand estimation process and is outlined in Figure 5.17. It should be noted that this demand estimation process was developed and refined by this research effort. The numbers, (n), in this section correspond to those shown in Figure 5.17. This demand estimation process was repeated for every half-hour time slice.

In the first stage of the demand estimation process, the INTEGRATION model was run with no traffic on the network to produce a set of *static* minimum path trees, 1. One set of minimum path trees was created 30 minutes into the simulation, which is the end of the first time slice. It should be noted that the network was not completely empty for this INTEGRATION run, there were actually a 'few' vehicles traveling through the network. If an origin/destination pair had no demand specified between it, then INTEGRATION would not generate a routing tree between-this origin/destination pair. Thus, this first run with INTEGRATION, 1, had an origin/destination matrix that specified a demand of one vehicle per hour between each origin/destination pair. It was felt that the path trees generated with a network with a 'few' vehicles would be virtually identical to those generated with a network with absolutely no traffic on it.

This first set of routing trees was based on free flow travel times because there was essentially no traffic in the network. The first set of routing trees is referred to as static because link free flow travel times are the same for each time slice, thus this set of trees is the same for all time slices. This set of routing trees was only generated for the first time slice and used as the starting point in the demand estimation process for each time slice.

Since the QUEENSOD program assigns traffic on an all-or-nothing basis when only one set of minimum path trees is entered, this first set of path trees had to be inspected closely to determine that the routing was realistic. The MPT program was used to analyze the minimum path tree file generated by the ASSIGN module of INTEGRATION. The source code and complete description of the MPT program is contained in Appendix C. The MPT program reads the minimum path tree file (INTEGRATION output file 13) and generates the links a vehicle would follow between any user specified origin/destination pair. The path tree generated for the empty network were extensively analyzed. Some changes were made to the network (mainly link free flow velocities) to alter the path trees in order to produce realistic routing behavior.

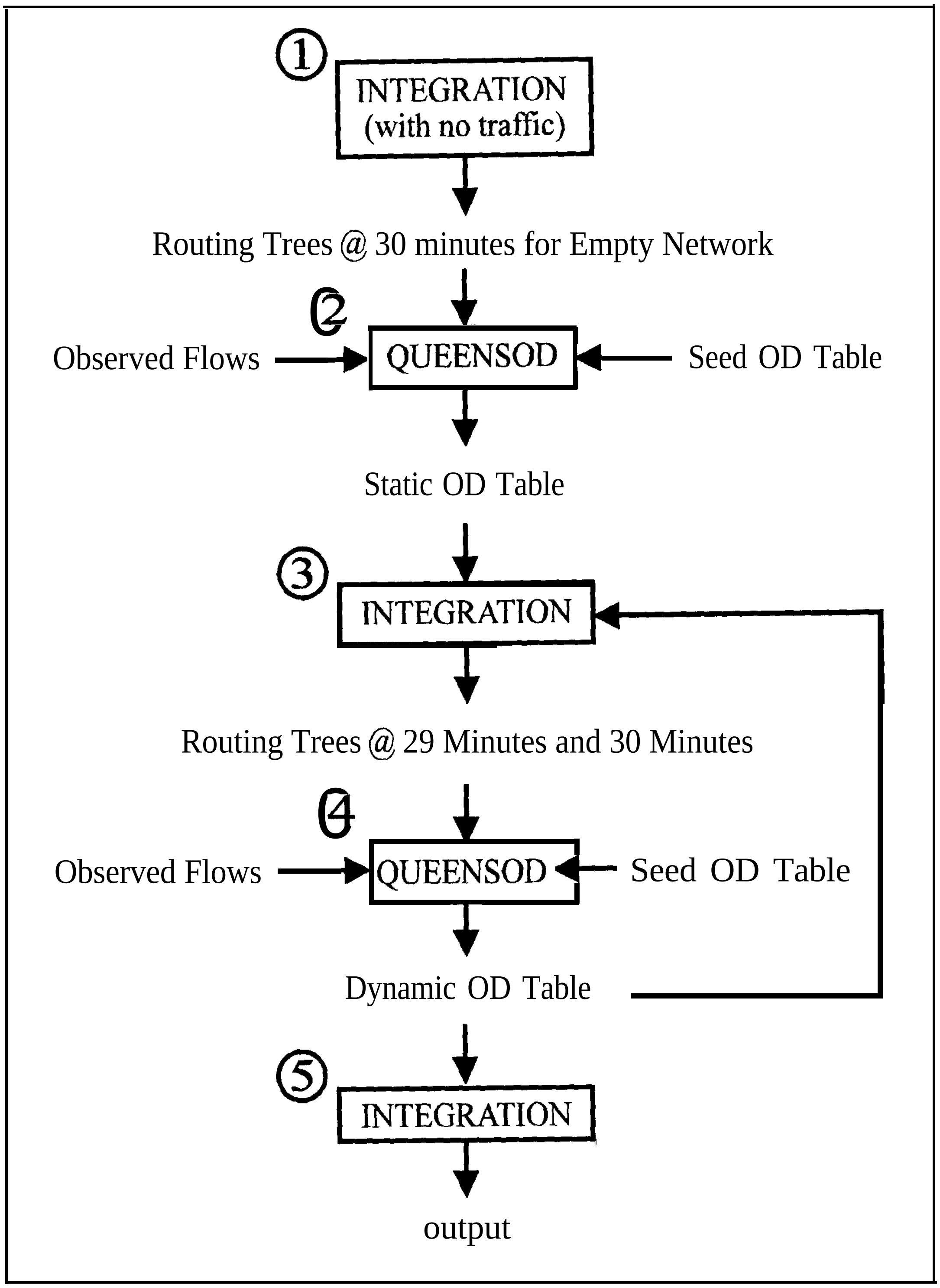

Figure 5.17: The Demand Estimation Process

With an acceptable first set of routing trees established for an empty network, the QUEENSOD program was run to produce a static origin/destination matrix, 2. This matrix can be considered the first estimate of the origin/destination matrix for each time slice. Each time slice required the generation of a static origin/destination demand table because this table was based on observed link flows, which varied every time slice. The matrix generated from this run of the QUEENSOD program is referred to as a static origin/destination table because the routing trees used in the origin/destination estimation process of QUEENSOD were generated from an empty network.

The next stage of the demand estimation process involved running the INTEGRATION model with this static origin/destination demand matrix to produce a set of *dynamic* path trees for each time slice, 3. This next set of routing trees is referred to as dynamic because they were generated from a network with traffic loaded onto it.

With traffic on the network, the minimum path trees will tend to oscillate between various routes. Again, since assignment in QUEENSOD is all or nothing, the use of a single set of dynamic path trees was deemed inappropriate because the demand between an origin/destination pair utilize numerous routes, not just one route. Initial runs with one routing tree were problematic. Incorporating two routing trees into the origin/destination estimation process relieved many of the problems.

To produce multiple path trees, INTEGRATION was run with the static origin/destination matrix for a simulation time of 29 minutes and then with a simulation time of 30 minutes. The minimum path trees produced by these simulation runs represent the travel times at the end of each simulation. The path trees generated at the end of the time slice were used to allow the network to become fully loaded, as these INTEGRATION runs, 3, were done on a time slice by time slice basis. The dynamic routing trees for each time slice were produced from the corresponding one time slice simulation run. For example, the time slice two static origin/destination table was used for a one time slice run in order to generate the dynamic routing trees for time slice two. It would of been more desirable to run the static origin/destination table for time slice one followed by the static origin/destination table for time slice two all in one run. The dynamic routing trees would then be generated at the end of the second time slice. However, this was not possible due to the huge memory requirements and slow computer run times of INTEGRATION with the ASSIGN feature activated for more than one time slice. It was felt that the traffic conditions from just simulating the time slice of interest would be accurate enough, at the end of the time slice, to generate dynamic routing trees for the time slice.

These two dynamic path trees, which were generated at 29 and 30 minutes of the INTEGRATION run for that time slice, were then manually merged into one file. Each routing tree was then assigned a weighting factor of 0.5. This file containing two sets of path trees was then used in the next run of QUEENSOD, 4. The origin/destination demand table generated by this QUEENSOD run is referred to as a dynamic origin/destination table because it is based on two sets of dynamic routing trees. Every time slice required the generation of this dynamic origin/destination table.

This dynamic origin/destination table was then used as input to the INTEGRATION model, 5. Time slice with heavy demand patterns usually required a few iterations of the demand estimation process. Each iteration required another INTEGRATION run, 3, and a QUEENSOD run, 4. After a few iterations, the difference between the dynamic origin/destination table generated in each iteration had become small enough that the demand estimation process could be considered to have converged to a final origin/destination table. The decision of when a final origin/destination demand table had been achieved was somewhat arbitrary.

The final origin/destination table resulting from the demand estimation process may not result in reasonable traffic performance when used in the INTEGRATION simulation. The problems in the demand estimation process and efforts to produce a more reasonable final origin/destination demand table are discussed in Chapter 7.

5.2.6 INTEGRATION Origin/Destination File

The final output from the demand estimation process was input file number four for the INTEGRATION model, the origin/destination demand file. Figure 5.18 shows a portion of this file along with an explanation of the various data fields.

```
O/D File for Santa Monica Freeway Simulation
3633   0   0   1.0
   1    13   22   1   0   1800   0.00   1.00 0.00 0.00 0.00   0.0 1.0
   2    14   6   1   0   1800   0.00   1.00 0.00 0.00 0.00   0.0 1.0
   3    15   211   0   1800   0.00   1.00 0.00 0.00 0.00   0.0 1.0
```

First line:
  Title line
Second Line:
  Number of origin/destination pairs
  First o/d pair to be used
  Last o/d pair to be used (0 0 uses all pairs)
  Growth factor for all o/d pairs
All subsequent lines:
  Origin/destination pair number
  Origin node
  Destination node
  Departure rate for this o/d pair (veh./hr.)
  Randomness factor (0.0 - 1 .O)
  Start time of this o/d pair (seconds)
  End time of this o/d pair (seconds)
  Percentage of type 1 vehicles (0.0 -1.0)
  Percentage of type 2 vehicles (0.0 -1 .0)
  Percentage of type 3 vehicles (0.0 -1 .O)
  Percentage of type 4 vehicles (0.0 - 1.0)
  Percentage of type 5 vehicles (0.0 -1 .O)
  Fraction of probe vehicles
  Passenger car equivalency factor

Figure 5.18: Portion of INTEGRATION Origin/Destination File

5.3 INTEGRATION CONTROL CODING

The final output from the preparation of the network control data was INTEGRATION input file number three, the signal timing plan file. This file contains a time series of signal timings that are to be in effect during the simulation period. The signal timing plans are specified in terms of the cycle length, offset of the start of phase one, the number and sequence of phases, and the duration of the green intervals and lost times for each phase. Freeway ramp metering plans are modeled in the INTEGRATION model as traffic signals. The freeway control data, namely the ramp meter timing plans, were manually added to the arterial data ASCII file containing the arterial signal timing plans. This modified ASCII file was then used to create the INTEGRATION input file number three. It should be noted that the ramp meter timing plans were coded into the INTEGRATION input file number three, but were not referenced in the link file. Thus, the ramp meter timing plans were not engaged during any of the INTEGRATION simulation runs conducted as part of this research project. The ramp meter timing plans were added to the arterial data ASCII file for completeness. The process of integrating the arterial and freeway control components was rather straightforward since there is only a small amount of control data for the freeway. This section discusses the creation of the signal timing plan file (INTEGRATION input file number three) and modifications to the link file (INTEGRATION input file number two) that are related to control data coding.

53.1 Coding the Signal Timing Plan File

Recall that the output from the SIG-REDC program was the four main signal timing plans for each of the signalized intersections in the network (see Section 4.3.3.2 and Figure 4.20). The operating time of these four timing plans throughout the day are contained in Table 4.4. The freeway ramp metering plans were manually added to the ASCII file created by the SIG-REDC program. A ramp metering rate was coded as a signal timing plan by calculating the cycle length, number of phases, and green time that would result in a signal timing plan that yielded a maximum allowable ramp flow equivalent to that possible with the corresponding ramp metering plan. Refer to Bloomberg and May [9] for a more detailed discussion of how the freeway ramp metering plans were coded in INTEGRATION as signal timing plans. Recall that the ramp meter timing plans were added to the ASCII file created by the SIG-REDC program for completeness and were not implemented during any INTEGRATION simulation runs.

This modified ASCII file, which contains the four signal timing plans for each signalized intersection and freeway on-ramp, was entered into the TO_INT3 program to generate the INTEGRATION input file number three. The source code and a complete description of the TO_INT3 program are found in Appendix U.

The functioning of the TO_INT3 program is rather straightforward. The TO_INT3 program performs two main tasks in generating the INTEGRATION input file number three from the modified ASCII file. The raw signal timing data contained in the modified ASCII file does not contain any information regarding yellow time in each phase. The first task of the TO-INT3 program was to divide the green time for each phase into three seconds of yellow time and the remainder into green time. This adjustment was not made for signal phases that overlapped. For example, an intersection approach operating on two consecutive phases, such as a protected left-turn phase followed by a permitted left-turning phase, was not coded with a three second yellow time between these phases.

The second task of the TO_INT3 program was to restructure the modified ASCII file to the format required in the INTEGRATION input file number three. This input file requires that all the first signal timing plans for all the intersections and on-ramps be listed first, and then all the second signal timing plans be listed, and so on. One problem encountered was that while INTEGRATION allows for separate signal timing plans to be used, the duration of each plan is required to be the same. This was easy to rectify by listing all the timing plans in one hour intervals. The final INTEGRATION input file number three listed 14 signal timing plans (one for each hour) in consecutive order. If a plan were actually in effect for more than one hour, the plan would simply be repeated as necessary.

Unfortunately, the INTEGRATION program does not simulate actuated signals and there were two actuated signals in the network. Fortunately, these two intersections were on minor streets and this limitation of the program was not considered serious. To code the actuated signals, the flow at each intersection approach was analyzed and a fixed signal timing plan was developed based on these flows.

A portion of the final signal timing plan file developed.and a description of the various data fields is shown in Figure 5.19. Recall that the four signal timing plans for each signalized intersection in the arterial network are contained in Appendix N. The freeway ramp metering plans are contained in Appendix F. The INTEGRATION input file number three is not contained in the appendixes as the information in this input file is contained in Appendixes F and N.

5.3.2 Coding the Arterial Component Control Data

Much of the control data is actually coded in the link file (INTEGRATION input file number two). This subsection discusses the changes that were made to the link file in the coding of the control data. The coding of the one way streets in the network was done by simply eliminating the link representing the direction of traffic that is not permitted. There were only two-one way links in the network: Figueroa Street between Olympic Boulevard and destination zone number 28, and 1 lth Street between Figueroa street and origin zone number 112.

Turning restrictions that existed for the entire simulation time were coded by removing the link which permitted the turning movement. For turning restrictions that existed for a portion of the simulation time, the turning restriction feature of INTEGRATION was employed. To code turning restrictions, the link number which can not be accessed from the current link is specified on the current link along with the start time and end time of the turning restriction. Vehicles may not travel from the current link to the specified link between the starting and ending time of the specified turning restriction duration.

```
Signal timing plans for Santa Monica Freeway
338  14   3600
1
    2   90   49  120  41   2  57   3   27   30
    3   90   56  120  51   2  37   3   47   30
    4   90   54  120  87   2  57   3   27   30
    5   90   33  120  85   2  59   3   25   30
    7   90   60  120  36   3  27   3   37   3   17  3  0
```

First line:
  Title line
Second Line:
  Number of traffic signals in the network
  Number of signal timing plans
  Duration of each of the signal timing plans (seconds)
Third Line
  Signal timing plan number
Subsequent lines:
  Signal number
  Initial cycle length (seconds)
  Minimum cycle length allowed (seconds)
  Maximum cycle length allowed (seconds)
  Offset (seconds)
  Number of phases
  For each phase:
    Phase duration (seconds)
    Lost time (seconds)
  Optimizer interval (seconds)
Note that the file repeats from the third line on
  for each of the signal timing plans.

Figure 5.19: Portion of INTEGRATION Signal Control File

The link file also contains the data fields that determine which signal number the link is associated with, and which phase(s) the link operates on. The signal number associated with each link was specified in the arterial supply data spreadsheet. First, the nodes were referenced to a specific signal number. Then in the arterial supply data spreadsheet each link with a downstream node associated with a signal was assigned that signal number. To assign the appropriate signal phase(s) to each link, no automated process was developed. Thus, the signal phases for each link were entered manually. Since the ramp meter timing plans were not implemented for any of the INTEGRATION simulation runs, the ramp links were not modified.

There were only a few instances (thirteen) in the network where a link was controlled by a STOP sign or a YIELD sign. To code these within INTEGRATION, the user codes the signal number as 10002 for a STOP sign and 1000 1 for a YIELD sign. No phase number was assigned. Yield sign controlled links are assigned a constant travel time penalty of 10 seconds by the INTEGRATION model. Similarly, stop sign controlled links are assigned a constant travel time penalty of 20 seconds.

The process of expanding the intersections from a single node to an eight node/twelve link configuration using the INTGEN program allowed for many of the control aspects of an intersection to be coded more accurately. The INTGEN program transferred the control aspects from the original approach link to the three new links created for each traffic movement (left-turning, through moving, and right-turning traffic). This transfer of control data was accomplished by changing the signal number and phase numbers of the original approach link to zero and then assigning the appropriate signal numbers and phasing to each of the links in the expanded intersection.

The left-turning links within the expanded intersection were also appropriately coded to represent protected and permitted turning movements. This was done by using the opposing link feature of INTEGRATION. The JNTGEN program assigned the link number of the opposing through movement to each of the left-turning links. For permitted left-turns, the INTEGRATION program will reduce the capacity of the left-turning link as a function of the flow on the opposing through movement. In executing the opposing link feature, the program checks to see if the opposing through movement discharges during the same phase as the left-turn movement. If it does not, the movement is protected and the capacity of the link is unchanged. Refer to Chapter 7 for a discussion of the tests completed to determine the performance of signalized intersections with the opposing link feature activated.

The INTEGRATION model does not currently simulate right-turn on red (RTOR), so the right turn links were coded with no control but with reduced free-flow speed. With this coding, right-turning vehicles could proceed through the intersection on red but would be delayed a certain amount of time. The fixed delay was set to approximate the delay a vehicle would experience in having to make the turning movement. This delay would increase as the traffic flow increased and/or if the through or left movements backed up into the preceding approach link that is common to all three movements.

Chapter 6: TESTING THE INTEGRATION MODEL

The purpose of this chapter is to describe the testing of the INTEGRATION model that was conducted as part of this research effort. Model testing was conducted continuously throughout the entire project, however the majority of the model testing was conducted during the initial stages of the calibration phase of the project, which concentrated on calibrating a single expanded signalized intersection. There were two objectives in performing the INTEGRATION model testing activities. First, to identity problems within the INTEGRATION program and where possible, rectify them. Second, to determine when the calibration process could commence and what limitations would be imposed upon the calibration and investigation stages of the project.

6.1 INTRODUCTION

The INTEGRATION model was continuously modified throughout most of this project as problems with the model were encountered and as the model developer released enhanced versions of the program. The INTEGRATION model version 1.4 was tested in early 1992 [20], but the version to be used for calibration was significantly diierent from the version tested in early 1992. Thus, thorough testing of the INTEGRATION version to be used for the calibration phase of the project was deemed necessary due to the limited number of real-world applications of the model and due to the continuous updating of the model.

The ability of the INTEGRATION model to realistically model freeway and arterial links was tested in Section 6.2. The INTEGRATION model's ability to simulate various simple freeway networks was also tested in Section 6.2. The ability of the INTEGRATION model to simulate the freeway-only portion of the corridor was determined in Section 6.3. Thus, the testing of the INTEGRATION model considered both freeway and arterial links, however the freeway portion of the corridor was emphasized due to its importance in the corridor simulation.

In Section 6.2, the INTEGRATION model was tested on a series of hypothetical freeway and arterial networks. The purpose of this testing was to identify any problems remaining in the latest version of INTEGRATION available before the calibration of the Santa Monica Freeway corridor began.

The next INTEGRATION model test, Section 6.3, was conducted on the freeway-only section of the Santa Monica Freeway corridor. This part of the testing developed two simulations of the freeway, one using the FREQ model and the other using the INTEGRATION model. The FREQ model was calibrated to the empirical freeway data and then the INTEGRATION model was calibrated to match the FREQ model's output. The purpose of this test was to determine the capability of the INTEGRATION model to realistically simulate a freeway-only network. Recall that the link flows predicted by the calibrated FREQ model were used as the observed freeway link flows in the demand estimation process, as discussed in Section 5.2.2.

Finally, Section 6.4 discusses research that was performed concerning the simulation of High Occupancy Vehicle (HOV) facilities using the INTEGRATION model. The research on HOV facilities consisted of simulating a simple straight-pipe bottleneck and the smaller Santa Monica Freeway corridor that was coded by Gardes and May [20, 22] and discussed in Section 2.2.6 (refer to Figure 2.4).

A plateau in the testing of the INTEGRATION model was reached once the above tests were complete. These tests were completed by July of 1994. This plateau was the ability of the current INTEGRATION version to satisfactorily model simple hypothetical freeway and arterial networks and the freeway-only portion of the Santa Monica freeway corridor. The next step from this plateau was to expand the calibration effort from the calibration of a single intersection to the calibration of the entire Santa Monica Freeway corridor, which is discussed in Chapter 7.

6.2 INTEGRATION MODEL TESTS ON SIMPLE NETWORKS

This section of the report details the INTEGRATION tests that were conducted on a series of hypothetical freeway and arterial networks. These tests were conducted using both INTEGRATION version 1.5d and version 1.5e. The primarily difference between version 1.5d and 1.5e is the methodology used to calculate link travel times. Accurate calculation of link travel times by the INTEGRATION model is crucial, as the minimum path trees are based on link travel times. Thus, proper vehicle routing behavior is dependent on accurate link travel time calculations. INTEGRATION version 1.5d calculated link travel times as the sum of the original free link travel time plus any increases that were strictly due to the increase in the magnitude of the traffic volume on the link. The increase in travel time due to traffic volume was based on the travel time vs. link volume/capacity ratio relationship, which was user-specified for each link. INTEGRATION version 1.5e employed car-following theories to model vehicle flows. The capacity and traffic flow at the entrance to a downstream lii determined the maximum allowable discharge rate at the exit of an upstream link. Thus, version 1.5e calculated traffic progression on a link using car-following theories and the maximum vehicle discharge rate at the exit of the lii. The lii travel time was then determined by averaging the travel time of each individual vehicle to traverse the link. Both INTEGRATION versions 1.5d and 1.5e were tested on the hypothetical freeway and arterial networks because these versions were the most recent versions of the program available before the calibration stage of the project was to begin (March of 1994).

The purpose of this testing on hypothetical networks was to identify any problems remaining in the latest version of the INTEGRATION program available before the calibration of the entire Santa Monica Freeway corridor began. These tests also served to identify the version of INTEGRATION, version 1.5d or 1.5e, which was to be used for the calibration stage of the project.

The test networks coded in Sections 6.2.1 and 6.2.2 were identical to the networks used by Gardes and May [20] for testing the modeling performance of INTEGRATION version 1.4. These same networks were used to facilitate the comparison of predicted traffic performance by the INTEGRATION model, FREQ model, and analytical calculations. The work performed for this section of the report was conducted by Stephane Gastarriet.

6.2.1. Freeway Straight-Pipe Bottleneck

Figure 6.1 illustrates the hypothetical freeway straight-pipe bottleneck coded to test the INTEGRATION model's ability to simulate traffic conditions at a freeway bottleneck. This is the same test example described in the earlier Section 2.2.3. The purpose of reusing this test example was to insure that the newer versions of the INTEGRATION model (versions 1.5d and 1.5e) gave similar results to the FREQ model and analytical calculations. The freeway segment is divided into five subsections with traffic flowing from left to right. The first four subsections are composed of three lanes and the fifth subsection is a bottleneck composed of only two lanes. All subsections are 610 meters (2000 feet) long. Each lane has a capacity of 2000 vehicles per hour and a free flow speed of 97 km/h (60 mph). The A and B parameters to specify the speed-flow curve in INTEGRATION were set at 0.7 and 5.0 respectively.

The demand pattern used for this test is contained in Table 6.1. The traffic hourly rates were chosen so that congested conditions occur on the network. There is one origin along the directional freeway: the freeway mainline origin. All the traffic is destine to the downstream end to the freeway segment.

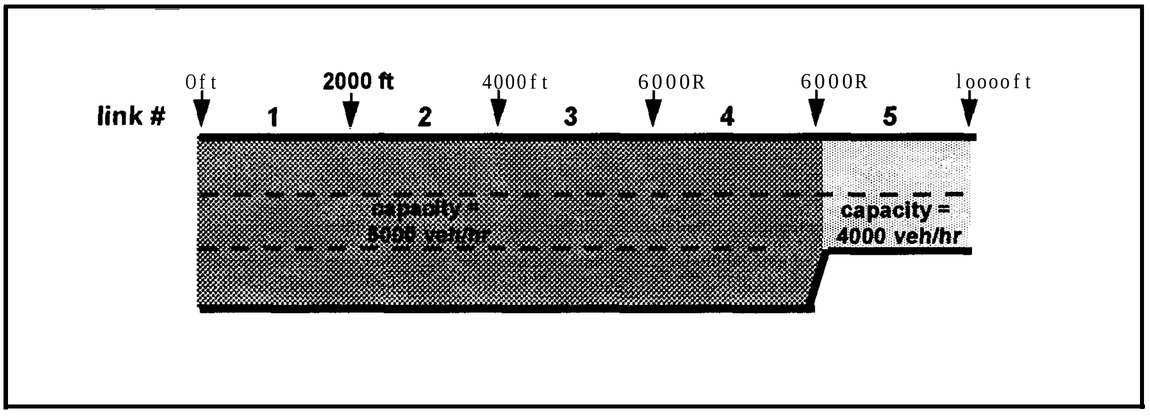

Figure 6.1: Freeway Straight-Pipe Bottleneck Network

Time slice	Time period	Input origin
1	8:00 - 8: 10	3000
2	8:10 - 8:20	4000
3	8:20 - 8:30	5000
4	8:30 - 8:40	3000
5	8:40 - 8:50	3000

Table 6.1: Freeway Straight-Pipe Bottleneck Network - Demand Pattern

The results of simulation runs using INTEGRATION version 1.5d and 1.5e are compared with the results from a simulation run using the FREQ model and an analytical solution use shock-wave theory. The results from the FREQ model and analytical solution were taken from Gardes and May [20]. FREQ is a macroscopic deterministic simulation model that is widely accepted as an accurate simulation model for predicting freeway traffic performance. The speed-flow curve coded in INTEGRATION and FREQ, and employed for the analytical solution were all approximately equivalent in order to compare results.

The summary of the queuing analysis comparison of queue lengths, queue starting and ending times, and time of maximum queue length are contained in Table 6.2. The INTEGRATION version 1.5d and 1.5e queuing analysis results were identical. The INTEGRATION model's predicted queuing patterns were very close to the theoretical values. The INTEGRATION model's predictions were also close to the FREQ predictions. The discrepancies between the INTEGRATION predictions and the FREQ predictions is probably due to the fact that FREQ uses a time slice approach in which freeway congestion can only begin and end at boundaries between time slices, whereas the INTEGRATION model uses a continuously updated simulation approach. Therefore, the queuing analysis indicates that the predicted traffic performance from INTEGRATION versions 1.5d and 1.5e was approximately equivalent to the traffic performance predicted by FREQ and shock-wave analysis.

Model	Queue Start Time	Maximum Queue Time	Maximum Queue length	Queue End Time
Shock-wave	8:22	8:31	0.87 mile	8:42
FREQ	8:20	8:30	1.1 miles	8:44
INTEGRATION 1.5d	8:23	8:31	0.9 mile	8:42
INTEGRATION 1.5e	8:23	8:31	0.9 mile	8:42

Table 6.2: Freeway Straight-Pipe Bottleneck Network - Queuing Analysis

The travel times, volume to capacity ratios and densities predicted by the INTEGRATION versions 1.5d and 1.5e and FREQ in each time slice for each link were also compared. All three models predicted that the bottleneck creates a queue which spreads upstream of the bottleneck. All three simulation models predicted similar travel times and volume to capacity ratios. The two INTEGRATION versions predicted similar traffic densities, however these density predictions were noticeably higher than the FREQ predicted traffic densities. Thus, the INTEGRATION versions were predicting heavier traffic than FREQ. Therefore, the traffic performances predicted by FREQ, INTEGRATION version 1.5d and INTEGRATION version 1.5e are all relatively close, except for the traffic density predictions.

6.2.2. Freeway On-Ramp Bottleneck

The hypothetical freeway on-ramp bottleneck coded to test the INTEGRATION model's ability to simulate traffic conditions at a more complicated freeway bottleneck than the straight-pipe bottleneck is illustrated in Figure 6.2. This test example was also described in the earlier Section 2.2.3. The directional freeway segment is divided into three subsections. The first subsection is three lanes wide and extends 4.0 km (2.5 miles) from the upstream end of the freeway segment to the first on-ramp. The second subsection is three lanes wide and extends 1.6 km (1 mile) from the first on-ramp to the second on-ramp. The third and last subsection is four lanes wide and extends 1.6 km (1 mile) from the second on-ramp to the downstream end of the freeway segment. Each lane has a capacity of 1800 vehicles per hour and a free flow speed of 97 km/h (60 mph). There is no control at the ramps. The A and B parameters to specify the speed-flow curve in INTEGRATION were set at 0.7 and 5.0 respectively.

The demand pattern used for this test is contained in Table 6.3. During the afternoon peak period, the traffic demand hourly rates are given for each origin in each 15-minute period. The traffic hourly rates were chosen so that congested conditions occur on the freeway. Since the ramp demands are relatively low in comparison with the ramp capacities and freeway shoulder lane, any congestion will effect freeway vehicles only. There are three origins along the directional freeway: the freeway mainline origin (O1), the first on-ramp (O2), and the second on-ramp (03). All of the traffic is destined to the downstream end of the freeway segment.

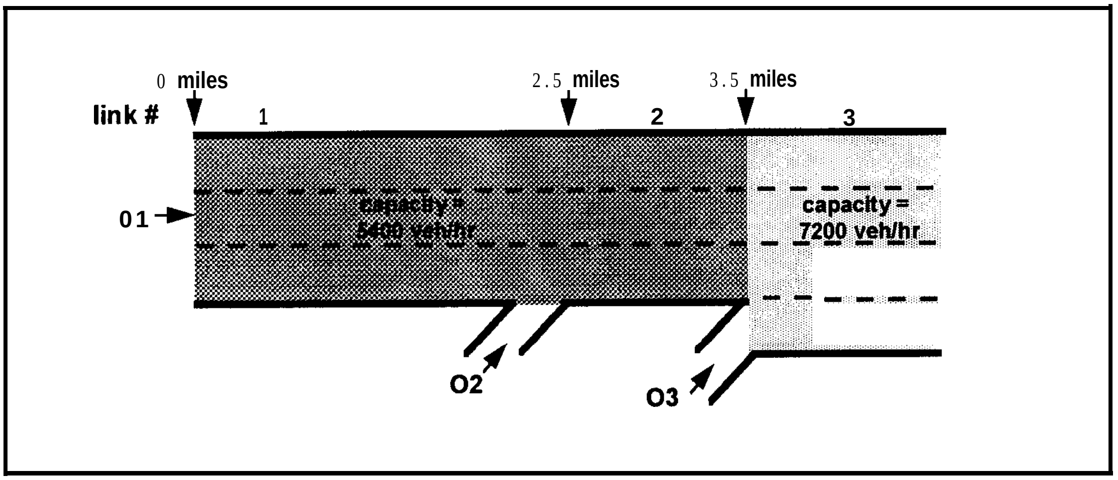

Figure 6.2: Freeway On-Ramp Bottleneck

Time Slice	Time Period	Origin O1	Origin O2	Origin O3
1	4:45 - 5:00	4800	400	800
2	5:00 - 5:15	4000	800	800
3	5:15 - 5:30	5000	800	800
4	5:30 - 5:45	3800	800	800
5	5:45 - 6:00	3200	200	600

Table 6.3: Freeway On-Ramp Bottleneck - Demand Pattern

The queue patterns predicted by the INTEGRATION versions 1.5d and 1.5e were compared with those from the FREQ model and the analytical shock-wave solution. The results from the FREQ model and analytical solution were taken from Grades and May [20]. The speed-flow curve employed in the analytical solution and used in both the INTEGRATION and FREQ models were all approximately equivalent in order to compare the predicted traffic performance from the various analysis.

The queuing patterns (queue lengths, queue starting and ending times, and time of maximum queue length) predicted by the various analysis are contained in Table 6.4. The INTEGRATION versions 1.5d and 1.5e predicted approximately the same queuing patterns. All analysis predicted approximately the same queue starting time and maximum queue time. However, the INTEGRATION versions predicted a slightly longer maximum queue length than both the shock-wave analysis and FREQ model. The predicted queue ending time by the INTEGRATION versions were a few minutes after that of the shock-wave analysis and FREQ model. Therefore, the queuing patterns predicted by INTEGRATION versions 1.5d and 1.5e were approximately equivalent and these predictions were similar to the FREQ and analytical estimated queuing patterns.

Model	Queue Start Time	Maximum Queue Time	Maximum Queue length	Queue End Time
Shock-wave	5:03	5:31	1.75 miles	5:51
FREQ	5:oo	5:30	1.7 miles	5:45
INTEGRATION 1.5d	5:01	5:31	1.9 miles	5:48
INTEGRATION 1.5e	5:01	5:31	1.95 miles	5:48

Table 6.4: Freeway On-Ramp Bottleneck - Queuing Analysis

The predictions of link travel times, volume to capacity ratios and densities by the INTEGRATION versions 1.5d and 1.5e, and the FREQ model in each time slice were also compared. The INTEGRATION version 1.5d predicted link statistics were nearly identical to those of INTEGRATION version 1.5e. The INTEGRATION versions predicted approximately the same link travel times and slightly lower volume to capacity ratios as the FREQ model. However, the link density values predicted by the INTEGRATION versions were significantly higher than the FREQ predicted densities. Therefore, the comparison of the traffic performance predicted by the INTEGRATION versions and the FREQ model for the freeway on-ramp bottleneck is comparable with the comparison for the freeway straight-pipe bottleneck.

6.2.3. Arterial Network

The purpose of this current investigation was to be assured that the newer versions of the INTEGRATION model (versions 1.5d and 1.5e) provided reasonable results for a hypothetical arterial network for which results could be anticipated. The hypothetical arterial network coded to test the INTEGRATION model's ability to simulate traffic conditions on an arterial network is illustrated in Figure 6.3. The arterial network coded represents two one-way arterial routes that intersect each other. Each arterial route has three signals and the crossing point of the two routes is the middle signal. Vehicles on the horizontal arterial route travel from 01 to D1 and vehicles on the vertical arterial route travel from 02 to D2. The horizontal route is composed of the following subsections: the first subsection extends 0.5 km (0.3 miles) from origin 1 to the first signal; the second subsection extends 0.5 km (0.3 miles) from the first signal to the second signal where the two arterial routes cross; the third subsection extends 0.5 km (0.3 miles) from the second signal to the third signal; and the fourth subsection extends 0.5 km (0.3 miles) from the third signal to destination 1. All subsections are composed of 2 lanes with each lane having a capacity of 2000 vehicles per hour and have a free flow speed of 70 km/h (43 mph). The vertical arterial is symmetrical to the horizontal arterial. The A and B parameters to specify the speed-flow curve in INTEGRATION were set at 0.7 and 5.0 respectively.

The demand pattern used for this test is contained in Table 6.5. There are two origins in the network: the horizontal route origin (01) and the vertical route origin (02). There are two destinations in the network: the horizontal route destination (D1) and the vertical route destination (D2). The demand pattern was set so there would be no turning movements at the node where the two arterial links intersect (signalized node number two).

The five signals in the network all had a 60 second cycle length and a green time of 27 seconds in each direction and a lost time of 3 seconds per phase. The offset for each signal are contained in Table 6.6. The signal offsets were set to allow efficient vehicle propagation along both the arterial routes.

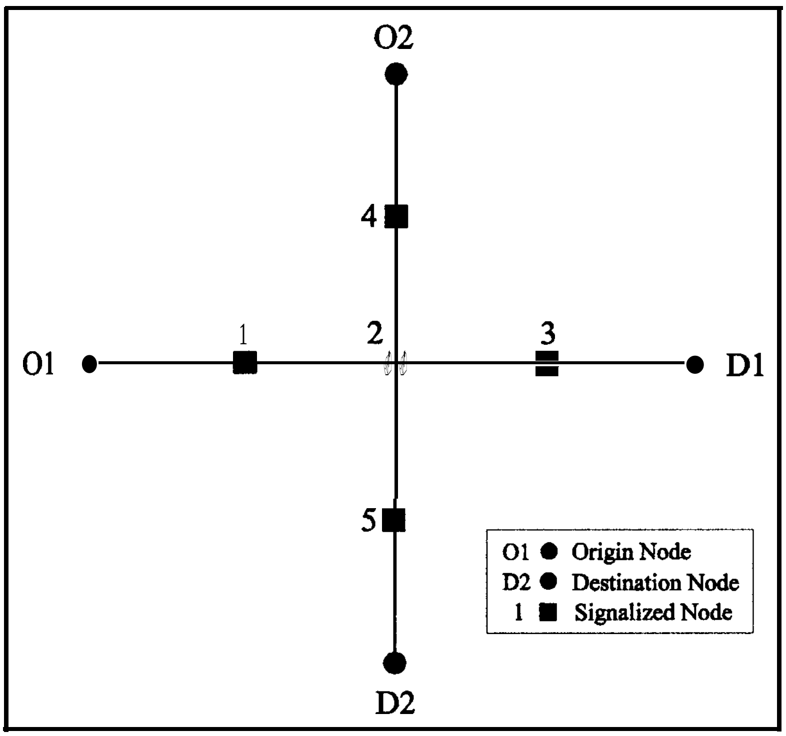

Figure 6.3: Simple Arterial Network

Origin/Destination Demand	D1	D2
O1	1800	0
O2	0	1800

Table 6.5: Simple Arterial Network - Demand Pattern

Signal Number	Offset (seconds)
1	6
2	30
3	54
4	0
5	48

Table 6.6 Simple Arterial Network - Signal Offsets

The performance of the INTEGRATION versions 1.5d and 1.5e in simulating arterial traffic performance was not as detailed as the previous two freeway network tests. The predicted traffic performance by the INTEGRATION versions 1.5d and 1.5e were not compared to other models, such as TRANSYT-7F, or an analytical solution. The INTEGRATION version 1.5d predicted arterial traflic performance was compared to the INTEGRATION version 1.5e predictions. The predicted traflic performance by both INTEGRATION versions was checked on a qualitative level for reasonableness.

The predicted total travel times along the horizontal and vertical arterials by both the INTEGRATION versions is contained in Table 6.7. Analysis of these values indicates that INTEGRATION version 1.5e is predicting slightly lower total travel times, but the predicted travel times are similar enough to conclude that both versions are predicting equivalent arterial traffic performance. Visual inspection of the queuing patterns at signalized intersections, on both the horizontal and vertical arterial, indicated that both the INTEGRATION versions modeling of arterial traffic flow was reasonable.

Model	Total Travel Time (minutes)	Total Travel Time (minutes)
INTEGRATION 1.5d	2821	2808
INTEGRATION 1.5e	2769	2758

Table 6.7: Simple Arterial Network - Total Travel Time Predictions

The INTEGRATION model tests on the hypothetical arterial network were not very extensive, but the tests did indicate that the INTEGRATION model was reasonably Simulating arterial traffic flow. These arterial tests also indicated that the INTEGRATION version 1.5d and 1.5e modeling of arterial traffic flow was essentially equivalent. Ideally these arterial network test could have been expanded and results compared to other simulation models, but time limitations did not allow detailed analysis of the simple arterial network.

6.2.4. Conclusions Concerning the INTEGRATION Model Tests on Simple Networks

The INTEGRATION model tests with the two simple freeway networks indicated that the INTEGRATION versions 1.5d and 1.5e were predicting similar traffic performance and that these predictions were similar to traflic performance estimates from a well established simulation model (FREQ) and analytical solutions. The INTEGRATION model tests with the simple arterial network indicated that the INTEGRATION versions were predicting similar arterial traffic performance and that these predictions, on a qualitative level, appeared reasonable.

The INTEGRATION model tests discussed in this section could not find any significant differences between INTEGRATION versions 1.5d and 1.5e or any significant modeling problems with the INTEGRATION model's ability to model traffic on *simple* networks.

6.3 INTEGRATION SIMULATED FREEWAY PERFORMANCE

Earlier versions of the INTEGRATION model had been tested for a smaller portion of the freeway, as described previously in Section 2.2.4. To test the INTEGRATION model on a larger scale the freeway-only portion of the Santa Monica freeway corridor was coded and calibrated in the INTEGRATION model. The ability of the INTEGRATION model to realistically simulate the freeway-only portion of the corridor is discussed in this section. INTEGRATION versions 1.5d and 1.5e along with an earlier version (1.5a) of INTEGRATION were all analyzed in this section.

Recall that the FREQ model was accurately calibrated to the empirical freeway data (refer to Section 5.2.2). The INTEGRATION model was then calibrated to match the FREQ model's output. The origin/destination demand matrix for INTEGRATION is usually developed by the QUEENSOD submodule. However, the synthetic origin/destination demand matrix derived by QUEENSOD with the available empirical data was found to be different from the demand data synthesized by the FREQ model. It would have been difficult to compare the results from FREQ and INTEGRATION if the demand inputs (i.e., the synthesized origin/destination data) were different. Therefore, it was decided to copy the demand data synthesized by FREQ for use in the INTEGRATION runs. This enabled comparisons between the two models to be made based on identical input data. Refer to Bloomberg and May [9] for a detailed discussion of the calibration process followed to calibrate the INTEGRATION model for the freeway-only portion of the corridor.

To calibrate the INTEGRATION model, contour maps of flows, densities, and speeds were analyzed. The congestion patterns between INTEGRATION version 1.5a and FREQ were approximately the same. The congestion patterns predicted by INTEGRATION versions 1.5d and 1.5e and the patterns predicted by FREQ were roughly the same, but there was significantly more congestion predicted by these newer versions of INTEGRATION.

It was not feasible nor necessarily desirable to tune the INTEGRATION simulation to perfectly match the FREQ output for several reasons. First, there are intrinsic differences in the simulations that make getting a perfect match very unlikely. At the simplest level, FREQ is a macroscopic simulation, while INTEGRATION uses elements of microscopic modeling, so there are bound to be some differences. However, if both models are valid, the same general results should be found for each model.

Detailed analysis of the INTEGRATION version 1.5d and 1.5e models revealed that INTEGRATION was not modeling freeway off-ramps reasonably. Vehicles exiting the freeway (vehicles traveling from a link of high capacity to a link with much lower capacity) were experiencing a slight delay which resulted in significant freeway congestion. This vehicle delay was not very significant for the one vehicle exiting the freeway, however all the freeway vehicles behind this exiting vehicle were subject to this delay.

It was not possible to modify the INTEGRATION program to correct this problem, thus the freeway off-ramp coding was modified instead. The off-ramp link that intersects the freeway was split into two links. The upstream link created from this split was assigned a capacity of 10,000 vehicles per hour. This modification allowed vehicles to exit the freeway without any undue delay, as these exiting vehicles were now traveling from a high capacity link to another high capacity link. The delay in traversing from a high capacity link to a low capacity link was now on the off-ramps and did not impact freeway traffic flow. The additional delay imposed on the off-ramps was determined to be fairly small. Modifying the off-ramp capacities resulted in the off-ramps being coded differently from the actual off-ramp geometry. It was felt that the impact of coding a portion of the off-ramp links with a capacity of 10,000 vehicles per hour was minimal due to the short length of these links.

INTEGRATION runs with the modified off-ramps resulted in substantial improvements in the INTEGRATION model's ability to model the freeway-only portion of the corridor. Figure 6.4 highlights the difference between the models in terms of travel time on the eastbound freeway (from end to end). There is a fairly good correlation between the earlier version of INTEGRATION (version 1.5a) and FREQ. The INTEGRATION version 1.5d without the off-ramp capacities modified predicted significantly different travel time then the FREQ and INTEGRATION version 1.5a. The INTEGRATION version 1.5e predicted similar travel times with the unmodified off-ramps as the INTEGRATION version 1.5d did with the unmodified off-ramps (the results of this run are not shown in Figure 6.4, as they are repetitive with the INTEGRATION version 1.5d run without the off-ramp capacities modified).

The INTEGRATION version 1.5e with the modified off-ramp capacities predicted travel times that were close to the FREQ predicted travel times, as illustrated in Figure 6.4. Therefore, the INTEGRATION 1.5e simulation with modified off-ramp capacities for the freeway-only portion of the corridor was fairly reasonable. However, there were still some questions about the performance of INTEGRATION in simulating the freeway-only portion of the corridor. The morning peak period congestion ends approximately a half-hour earlier with the INTEGRATION version 1.5e than with the FREQ model. The INTEGRATION version 1.5e also predicts lower travel times throughout the midday period and evening peak period. Interestingly, even though the INTEGRATION version 1.5e morning peak period ends earlier than that of the FREQ model, the INTEGRATION version 1.5e predicted higher freeway end-to-end travel times from 7: 15 to 8:30 a.m. than that of the FREQ model. Thus, INTEGRATION version 1.5e predicted heavier freeway congestion during the morning peak period. This significant problem in the INTEGRATION version 1.5e, which could not be resolved, proved to be a persistent problem throughout the entire calibration effort, as discussed in Chapter 7. Another significant problem with the INTEGRATION version 1.5e simulation of the freeway-only portion of the corridor was revealed in the analysis of the INTEGRATION predicted link flows. The INTEGRATION version 1.5e predicted link flows showed differences on the order of plus or minus 500 vehicles per hour from the FREQ predicted link flows. Efforts to reduce these differences were unsuccessful.

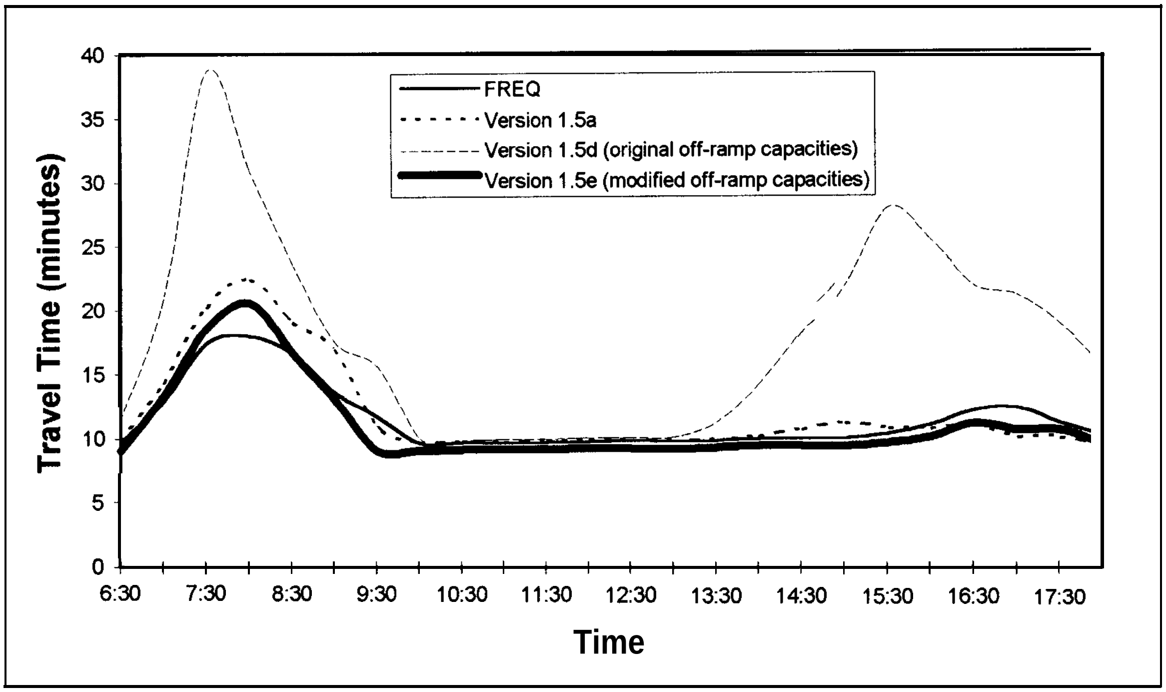

Figure 6.4 : Average End-to-End Freeway Travel Time (Eastbound)

6.4 SIMULATION OF HIGH-OCCUPANCY VEHICLE FACILITIES

One of the important features of the INTEGRATION program that was not tested in previous PATH research by Gardes and May [20, 22] was the simulation of High-Occupancy Vehicle (HOV) facilities. Also, there were no known applications of the model to such facilities. Since part of the proposed investigations involved the simulation of HOV facilities, it was decided that the model should be tested on a simple network to assess the model's capabilities in the simulation of HOV facilities. The research discussed in this section is presented in greater detail in reports by Bacon and others [4,5].

6.4.1 HOV Simulation Using the Straight-Pipe Freeway Network

The first set of simulation runs with HOV facilities involved a simple straight-pipe bottleneck network. This network consisted of ten linear links of mainline freeway. All of the links except one have four lanes of traffic, each lane having a capacity of 2,000 vehicles per hour. The penultimate link in the network was a three-lane bottleneck. The location of the bottleneck was selected to insure that congestion would occur in the non-HOV lanes and that the resulting queue would not block HOV vehicles from entering the HOV lane.

Initial simulation runs were performed with this straight-pipe freeway network as described above. Next, an additional lane of freeway was added to the network for the entire length of the freeway. This lane was first simulated as a mixed-flow lane and then as an HOV lane. The network with the additional HOV lane added is shown in Figure 6.5. With the HOV lane in place sensitivity analyses were performed by varying the percentage of passengers usiig HOV vehicles from 2 percent to 32 percent in 2 percent intervals. The sensitivity analyses were performed with 8,000 and 10,000 passengers in the network. For simplicity, it was assumed that each HOV vehicle carried two people.

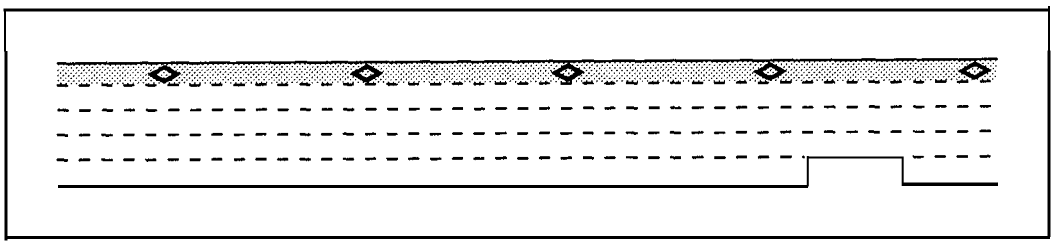

Figure 6.5: Freeway Straight-Pipe Bottleneck Network with an HOV Lane

The results of the simulation with 10,000 passengers in the network are shown in Figure 6.6. As expected, the benefits to HOV passengers decreased as the percentage of HOV passengers in the network rose. The travel time for HOV passengers increased as the percentage of HOV passengers increased and the HOV lane became more congested. Conversely, system-wide travel time decreased and the travel time for HOV passengers and all passengers converged when the level of HOV passengers increased to the point that there are no benefits to constructing an HOV lane. The system-wide travel time decreased with increasing percentage of HOV passengers for two reasons. First, the total number of vehicles in the network decreased. Secondly, as the percentage of HOV passengers rose the distribution of traffic on each of the lanes becomes more equal and hence more efficient. The results from the sensitivity analyses with 8,000 passengers are similar in form with less benefits to HOV passengers.

6.4.2 HOV Simulation of the Santa Monica Freeway Coded by Gardes and May

The next step was to test the model's ability to simulate HOV facilities on a more complex network. Since the Santa Monica Freeway corridor to be simulated in this research effort was not coded at the time of these initial tests, the smaller Santa Monica Freeway corridor network coded by Gardes and May [20, 22] and discussed in Section 2.2.6 was used. Sensitivity analyses were performed with an HOV lane added to the freeway for its entire length and with a mixed-flow freeway lane converted to an HOV lane for the entire length of the freeway. Figure 6.7 shows the results of the investigations with an HOV lane added to the freeway and Figure 6.8 shows the results where a freeway lane has been converted to an HOV lane.

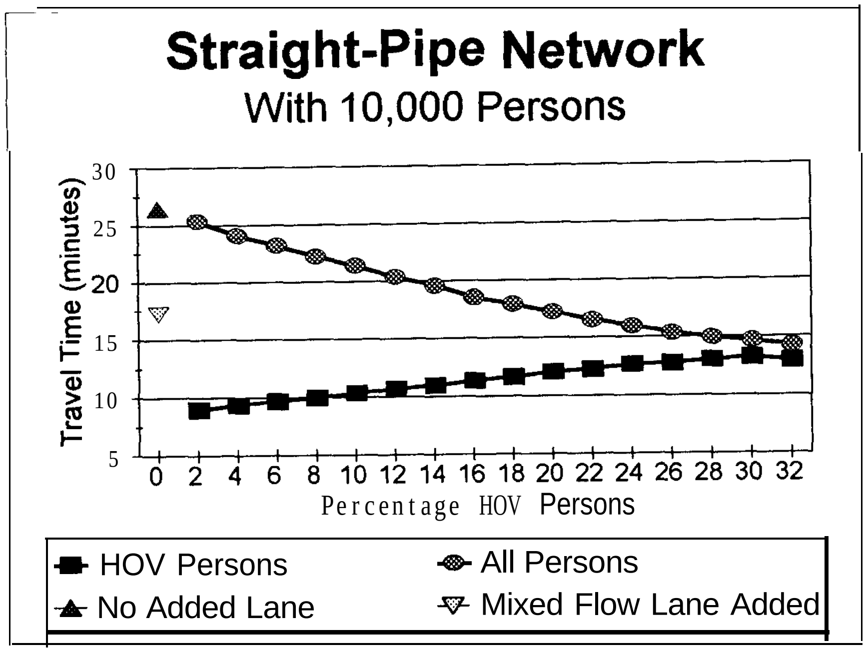

Figure 6.6: Straight-Pipe Network with 10,000 Passengers

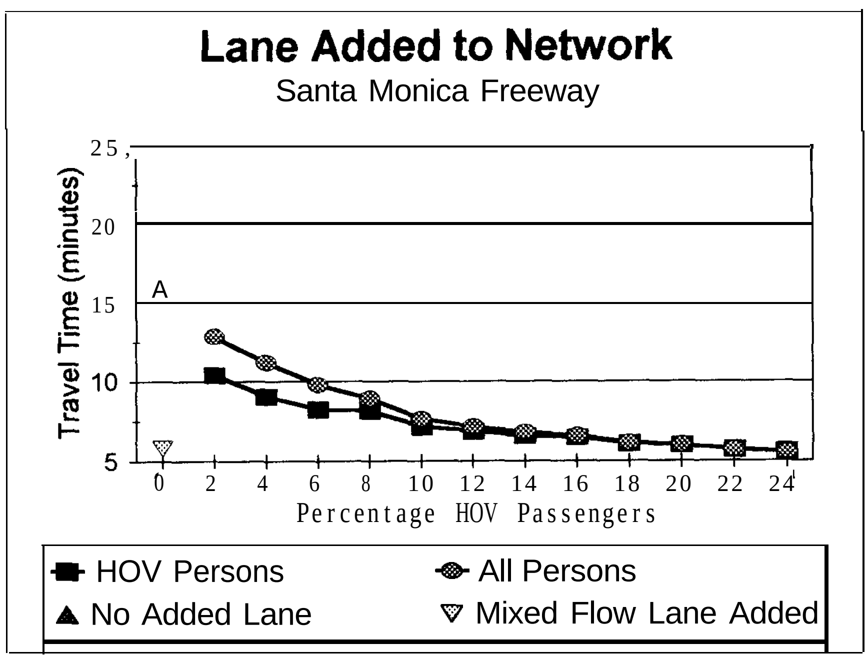

Figure 6.7: Santa Monica Freeway Network -HOV Lane Added

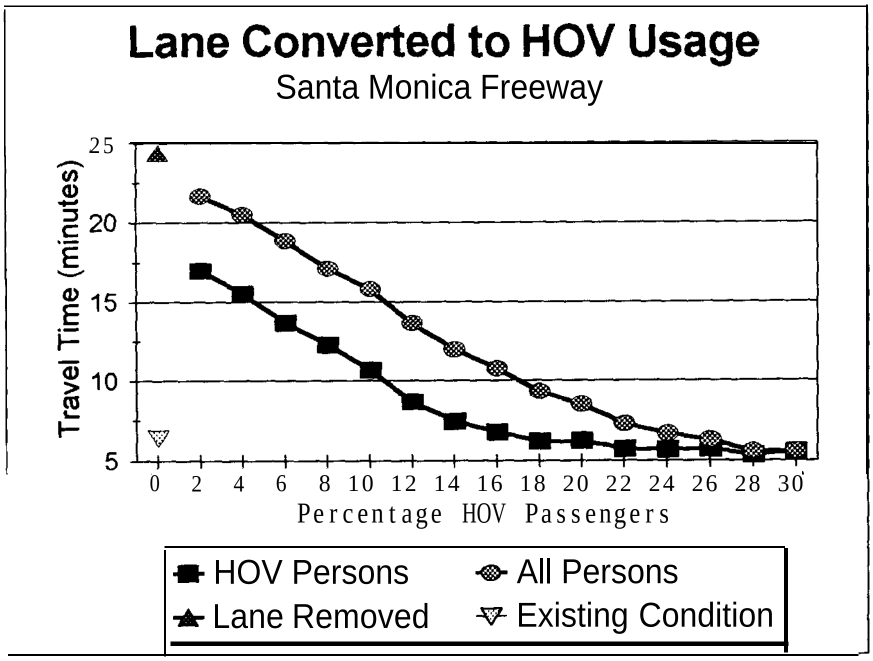

Figure 6.8: Santa Monica Freeway Network -Lane Converted

The most striking aspect of these results is that the travel time for HOV passengers actually decreased as the percentage of HOV passengers rose. This is likely caused by the heavy congestion that is present in both networks. As the percentage of HOV passengers rose, overall freeway congestion decreased which decreased the delays to vehicles entering the freeway. Since HOV vehicles are given no preferential treatment in entering the freeway, they also benefit from the decreased on-ramp delay which resulted from the decreased freeway congestion.

Another interesting aspect of these results is that with an HOV lane added to the freeway, the travel times for HOV and non-HOV vehicles converges rapidly. At 16 percent HOV passengers, the average travel times for all vehicles and HOV vehicles have essentially converged. This is most likely due to the fact that the Santa Monica Freeway, including the collector/distributor lanes, is seven lanes wide at many points. Thus, the point at which the HOV lane is as congested as the non-HOV lanes occurs at a lower percentage of HOV passengers,

6.4.3 Conclusions Concerning the Simulation of HOV Facilities with INTEGRATION

Overall, the model was found to be a robust model capable of simulating a wide range of HOV facilities. One potential problem with the model was lanes whose status changes with time. Since the time that these tests were performed, the developers of the model have included a left-turn prohibition feature that allows one to “turn off’ a link for a specified period of the simulation. This feature could also be used to simulate an HOV lane whose status changes with time.

Another problem encountered with the model was that the model functions in terms of vehicles and not in terms of passengers. Any output of the model must be converted from average vehicle travel time to an average passenger travel time. Also, if one is performing a sensitivity analyses such as the one above, the number of vehicles in the network must be altered to reflect the loss of vehicles as the percentage of HOV passengers increases. However, these problems were an inconvenience and not a serious problem.

6.5 CONCLUSION

The traffic performance predicted by the 1.5e version of the INTEGRATION model was found to be very similar to that of a well established simulation model (FREQ) and analytical solutions for simple freeway networks. The INTEGRATION version 1.5e predicted arterial traffic performance for a simple arterial network appeared reasonable. The 1.5e version of the INTEGRATION model was found to be as good as or better than the earlier 1.5a version in simulating the freeway-only portion of the corridor. Thus, a decision was made to use the 1.5e version of the INTEGRATION model in the future calibration effort.

A number of problems were identified in the INTEGRATION model program. In some cases the modal developers modified the program to overcome the identified problems, while in some other cases the research team overcame these problems by adding special features in the input data set. There still remained some identified problems which were not completely rectified.

While not completely satisfied with the results of the model testing effort, a decision was made to move ahead into the calibration effort for the entire Santa Monica Freeway corridor with the anticipation that these remaining problems could be overcome in the calibration effort. If these problems could not be overcome, then the calibration effort and the later planned investigations would be redesigned considering these limitations.

Chapter 7: MODEL CALIBRATION

This chapter describes the calibration effort that was undertaken in an attempt to calibrate the Santa Monica Freeway corridor network using the INTEGRATION model. This stage of the research project was one of the most extensive, involving over one year's efforts. By May of 1994, the supply, demand and control aspects of the freeway corridor network had been adequately coded, allowing the INTEGRATION model testing and calibration stages of the project to begin. The INTEGRATION model testing and the calibration of a single intersection were completed by July of 1994. The calibration of the Santa Monica Freeway corridor began in July of 1994 and continued until June of 1995. Due to time and fund restrictions, coupled with problems encountered with the INTEGRATION model, the calibration stage of the project only attempted the calibration of fourteen of the twenty-eight time slices to be simulated. Only six of these fourteen time slices were classified as calibrated at the conclusion of the calibration effort.

The need for a well-calibrated model is obvious. It would have been simple to develop a hypothetical complex freeway corridor network and perform ATIS and ATMS investigations on such a network. However, the results of investigations using a hypothetical network are not easily transferred to real-world conditions. By calibrating the INTEGRATION model on an actual freeway corridor, the simulation results give a more realistic assessment of which ATIS and ATMS strategies would be the most successful. Also, the effectiveness of potential ATIS and ATMS strategies will most likely vary from network to network making tests from a hypothetical network of little value. Thus, it is critical to calibrate a real life network and perform simulation runs with the calibrated network in order to realistically determine the optimum ATIS and ATMS strategies for that network.

During the INTEGRATION demand coding the FREQ model was accurately calibrated to the empirical freeway data (refer to Section 5.2.2). In early 1994, as part of the INTEGRATION model testing, the INTEGRATION model was calibrated to match the FREQ model in simulating the freeway-only portion of the corridor. This calibration effort, discussed in Section 6.3 and by Bloomberg and May [9], was satisfactorily completed with only a few misgivings (such as unrealistic queues at some of the freeway off-ramps). However, the calibration of the entire corridor proved to be far more complex for a number of reasons. First, for the freeway-only portion of the corridor a parallel model, FREQ, was available. The FREQ model was able to provide numerous statistics of freeway performance that the INTEGRATION model could use as measures of an effective calibration. Coding and calibrating a parallel model for simulating the freeway corridor network would have required an extensive effort. Thus, a parallel model was not coded for the freeway corridor network. Secondly, the potential for multiple path trees between the origins and destinations is virtually non-existent in the freeway-only network. In the freeway-only network, which is a linear system, a group of vehicles traveling from a given origin to a given destination will be assigned to a known set of links. This is not true in the corridor network, there are numerous paths between each origins and each destinations and vehicles may change their paths dynamically as a result of real-time traffic conditions.

Finally, the corridor calibration was difficult because a network of the size and complexity of the Santa Monica Freeway corridor had never been successfully calibrated using the INTEGRATION model nor equivalent model. While the model has been successfully used on freeway-only networks and small arterial networks, a detailed calibration of a large freeway/arterial corridor has not been performed. The calibration was further complicated by the fact that the eight node/twelve link signalized intersection configuration, which was coded at many of the intersections in the Santa Monica Freeway corridor, had not been utilized by any of the earlier networks simulated by the INTEGRATION model. The use of this intersection configuration is critical to accurately simulate a complex signalized intersection with significant turning movements.

Section 7.1 presents an overview of the calibration efforts. Section 7.2 discusses the calibration of a single eight node/twelve link signalized intersection.. Section 7.3 discusses the efforts to calibrate a single one half-hour time slice, while Sections 7.4 and 7.5 discuss the calibration of the morning and midday periods respectively. Finally, Section 7.6 presents the conclusions of the calibration efforts.

7.1 OVERVIEW OF THE MODEL CALIBRATION EFFORTS

As mentioned, the calibration efforts consumed more than a year's effort. Figure 7.1 presents a time line that shows approximately when each of the tasks in the calibration effort were conducted.

Figure 7.1: Time Frame of Calibration Efforts

The first phase in the calibration efforts was the calibration of a single signalized intersection. Once the calibration of a single signalized intersection and the INTEGRATION model tests discussed in Chapter 6 were completed, the calibration of the Santa Monica Freeway corridor began. Due to the complexity of the corridor network, it was decided that the calibration of the Santa Monica Freeway corridor should begin by focusing on the first half hour time slice by itself, as opposed to all 28 time slices. Once the problems associated with the calibration of the first time slice were resolved, additional time slices were added incrementally and attempts made to calibrate them.

Special versions of both the QUEENSOD and INTEGRATION programs were compiled to simulate the corridor network. Early in the calibration efforts it was discovered that the INTEGRATION simulation of all 28 time slices would tax the computing resources available, even though a personal computer with 64 megabytes of random access memory (PAM) was available. To avoid the extremely long INTEGRATION simulation run times on the order of a week and the huge memory requirements, it was decided that the simulation should be conducted in three parts; the morning peak period from 6:00 A.M. to 10:00 A.M., the midday period from 10:00 A.M. to 3:00 P.M. and the evening peak period from 3:00 P.M. to 8:00 P.M.

The first time period to be simulated was the morning peak period. Numerous problems were encountered in the calibration of the morning peak period that required assistance from the developers of the model at Queen's University. The model developer's provided guidance as how to utilize the QUEENSOD and INTEGRATION models during the research teams efforts to calibrate the morning peak period. The final attempts to calibrate the morning peak period were performed at Queen's University. The model developer's efforts at Queen's University resulted in a number of recommendations; which included disabling the opposing flow feature of INTEGRATION at expanded intersections and to increase the saturation flow rates for left-turning and right-turning movements to 2,800 vehicles per hour per lane at various over saturated intersections. These recommendations were not implemented as they were deemed inappropriate, since adjustments to input data during the calibration effort took great care in keeping input data values within ranges of acceptable values. The model developers also recommended disabling the signal optimization feature of INTEGRATION at expanded intersections and manually adjusting timing plans to avoid queue spill back problems. Manual adjustment of timing plans would be far too complicated in a network as complex as the Santa Monica Freeway corridor. Thus, the research team did not follow this recommendation. The output from the simulation runs which were performed by the model developers are not reported in this report as they were not received by the U.C. Berkeley research team. It should be noted that the model developers were not under contract to perform modeling for this research project.

Once the morning peak period was sent to Queen's University for final calibration, the U.C. Berkeley research team directed their attention to attempting to calibrate the midday period. Due to time limitations and unforeseen problems in calibrating the model, no attempts were made to calibrate the evening peak period.

7.2 CALIBRATION OF A SINGLE EXPANDED INTERSECTION

Simulation runs with the medium network (unexpanded intersections) quickly revealed that the one node signalized intersection configuration was not accurately simulating signalized intersections with significant turning movements. It was found that intersection approaches in which the left-turning vehicles were delayed had left-turning vehicles queued on the approach, which resulted in unrealistic delay for the through and right-turning vehicles on that approach. The congestion in the medium network was very significant, even for the time slice from 6:00 A.M. to 6:30 A.M., when in reality there is very little congestion during this time slice. Thus, the efforts to calibrate the medium network (with unexpanded intersections) were abandoned. It was determined that the critical signalized intersections in the Santa Monica Freeway corridor required expansion to the eight node/twelve link configuration in order to achieve a calibrated network.

Two parallel processes arose in the conversion from the medium network to the large network. Recall that the large network (refer to Figure 5.4) was created by expanding 167 of the 353 signalized intersections in the medium network. First, the network supply coding had to be modified at each signalized intersection to be expanded to include the additional links and nodes introduced in the expansion process and to reassign existing nodes impacted by the expansion process (see Section 5.1.4). Secondly, since there was no documented use of the expanded intersection configuration, the precise characteristics of each of the twelve links within the expanded intersection had to be determined. Once these processes were complete, the calibration of the complete freeway corridor network could be begin,

It should be noted that in conversation with the model developer it was learned that the INTEGRATION model had been successfully calibrated on networks with isolated traffic signals in which detailed observed link f-lows, speeds and travel times on all links at the intersection were available. Such calibration has indicated that the model correctly matches either analytical or field estimates of capacities, speeds, delays and queue sizes. This analysis indicated that traffic behavior, for congested conditions, is very sensitive to the exact magnitude of the prevailing demands and saturation flows, such that small errors in estimating these values may produce proportionally much larger errors in the traffic performance at a signalized intersection.

A limitation of the data available for most networks is that the available accuracy/resolution of intersection turning movements, exceeds the permitted error range in the input data. For example, the lack of knowledge of actual turning movement percentages at congested intersections not only creates errors in the opposing flow rates, and therefore the resulting opposed saturation flow rates, but also introduces errors in the turning demand. Such errors, while sometimes producing flow conditions that are better than expected in the field, can at other times cause major queue spill backs when they interact to produce flow conditions that are worse than those experienced in the field. Efficiencies experienced at intersections, that are modeled as operating better than observed in the field, do not offset the extra delays encountered at those locations where errors in turning demand and/or opposing flow create larger delays than expected.

For a network the size of the Santa Monica Freeway corridor, determining the exact magnitude of prevailing demands and saturation flows of all turning movements for all approaches at all signalized intersections is an extremely hard and time consuming task. Since the determination of these prevailing demands and saturation flows was not possible during the data collection stage of the project, only link flows on intersection approaches were obtained. With a network the size of the Santa Monica Freeway corridor, determination of just the approach flow at all signalized intersections in the network was not possible. Although the research team recognized the need for exact prevailing demands and saturation flows of all turning movements for all approaches to all signalized intersections and the lack of this data for the Santa Monica Freeway corridor, it was still felt the expansion of signalized intersections was required to realistically calibrate the network. This decision was largely based on the extreme difficulties encountered in the initial attempts to calibrate the medium network (unexpanded intersections).

7.2.1 Selection of Link Parameters for the Expanded Intersection Links

There are numerous link parameters to be specified for each link within the network. The selection of these factors for each of the twelve links in an expanded intersection configuration determines the traffic performance that would occur at that intersection. The main link factors to be specified for each of these links were the length of the link, link capacity, free-flow speed, and data on the links that are opposed by another link within the expanded intersection.

The length of links within the expanded intersections is critical in that it determines the queue length on a specific expanded link that will result in delay to other movements at that approach. Without expanded intersections, this length is effectively zero and a queue developing for one movement will immediately result in delay to the other movements at the approach. A link length of 100 meters (328 feet) was decided upon since the majority of left-turn pockets in the network were between 91.5 meters (300 feet) and 107 meters (350 feet). Thus, queues from one movement, usually the left-turn lane, would have to generate queues over 100 meters to result in delay to other movements at the approach. Of course, in expanding the intersections, the lengths of the original approach links were reduced by 100 meters.

Since saturation flows for the three turning movements at an intersection approach were not observed in the field, capacities of the various links within the intersection were developed based on values contained in the 1985 Highway Capacity Manual [53]. The final capacities used were 1601, 1800, and 1402 vehicles per hour per lane (vphpl) for the protected left-turning, through, and right-turning movements respectively. Free-flow speed was set at 30 km/h, 50 km/h, and 40 km/h for the left-turning, through, and right-turning movements respectively. It should be noted that while a constant value of capacity and free-flow speed was used for all intersection approaches, the INTGEN program allows the user to specify the capacity and free-flow speed for each approach movement.

Coding right-turning movements presented an additional problem due to right-turns on red. It was decided that some delay should be given to right-turning movements, but that they should not be under signal control as are the adjacent through and left-turning movements. One way to assign delay to the right-turning vehicles was to assign the corresponding through movement as the opposing link of the right-turn movement, However, it was felt that reducing the free-flow speed on the right-turn links to a value of 10 km/h was an easier way to impose a reasonable amount of delay on right-turning vehicles.

7.2.2 The Opposing Link Feature of INTEGRATION

With the eight node/twelve link intersection configuration, the opposing link feature of the INTEGRATION model was utilized to allow for the coding of left-turning movements which are delayed by an opposing vehicle flow. However, initial calibration efforts revealed that the existing opposing link feature of INTEGRATION was not properly assigning delay to left-turn movements given the traffic volumes on the opposing link. Working with the developer of the model, a new method of assigning delay to an opposed movement was developed for the INTEGRATION model. The new method developed is based on gap acceptance.

The new opposing link feature allows the user to alter the various parameters used in calculating the capacity reduction of a link based on the opposing flow in order to obtain a desired vehicle delay on the opposed link as a function of the opposing flow. The INTEGRATION model reads an auxiliary file entitled "OPS_FACT.DAT" that contains the factors that will be used to calculate the capacity reduction of the opposed link. If this file is not provided in the same directory that the INTEGRATION program is being executed, the default values for this file will be used. Figure 7.2 shows the format of the file along with the default va ıes for the tile.

Title Line	
1	**0**
2	**2**
3	**0**
4	**0**
5	**0**
6	**0**
7	**0**
8	**20**
9	**0**
10	**0**

Figure 7.2: INTEGRATION Optional Input File OPS_FACT.DAT

The format of the "OPS_FACT.DAT" file is rather straightforward, as it simply lists the ten K factors used in the determination of gap acceptance. The first line of the file is a title line and is ignored by the program. The first number on all subsequent rows is the K factor identification number. The second number of each row is the K factor itself.

K factors number one, two and three are used to describe the equation that determines the mean critical gap based on the minimum headway (saturation flow rate) of the opposing link. For example, if the saturation flow rate of the opposing link is 1,800 vehicles per hour then the minimum headway would be two seconds. The mean critical gap is defined by the following equation;

$$\text{Mean Critical Gap} = K, + K_2 * h + K_3 * h^2 \qquad \textbf{Eq. 7-1}$$

where h is the headway of the opposing link at the saturation flow rate.

K factor number four determines the randomness of the critical gap. A value of zero for this will produce a uniform distribution of critical gaps. Increasing this value towards one introduces a shifted negative exponential distribution, with a value of one generating critical gaps based completely on a shifted exponential distribution. For example, a value of 0.2 would indicate that 20 percent of the critical gaps are determined from a negative exponential distribution and 80 percent are determined from a uniform distribution.

K factor number five is the width of the uniform distribution of critical gaps when K factor four is set to zero. This number is given as a percentage of the mean critical gap. For example, if the mean critical gap is 6.0, K factor four is zero, and K factor five is 0.5, then the critical gaps will be uniformly distributed from 3.0 to 9.0.

K factor number 6 determines how fast the critical gap (minimum acceptable gap) decreases as a function of the length of time that a vehicle has been waiting at the intersection. This is based on the function;

$$h = e^{-K_6 t} \qquad \textbf{Eq. 7-2}$$

where: h is the headway of the opposing link at the saturation flow rate,
e is the natural logarithm, 2.718
K_6 is K factor number six, and
t is the length of time that the vehicle has been waiting at the intersection.

K factors seven and eight specify the lower and upper limits respectively of the critical gap accepted. K factors nine and ten are not yet used by the program.

The opposing flow feature was designed to accommodate left-turning movements that have permitted phases, protected phases, or both. For permitted left-turning movements, the opposing link feature was activated by specifying the link number of the opposing through movement in the left-turning link's description (refer to Figure 5.10). For protected left-turning movements, the opposing flow feature was not activated and would have no effect if it were activated. For left-turning movements where the movement is both protected and permitted, the opposed link would be coded the same as if it were permitted only. The INTEGRATION program checks to determine if the opposing movement is allowed to discharge during the same phase as the opposed movement. If it is not discharging during the same phase, then the movement is protected, and the opposing link feature is not activated. Thus, movements with protected and permitted movements can be coded with the model.

7.2.3 Calibration of the Opposing Link Feature of INTEGRATION

As mentioned, the opposing link feature of the INTEGRATION program was modified during the calibration stage of the research project. Thus, it had never been tested by the INTEGRATION model developer to determine if, by using this feature, the model could accurately estimate the delay imposed on left-turning vehicles under various levels of opposing flow.

The SIDRA (Signalized and unsignalized Intersection Design and Research Aid) computer simulation model was used as the base model against which the opposing link feature of the INTEGRATION model would be calibrated. The SIDRA model was developed by Dr. Rahmi Akcelik at the Australian Road Research Board. The model was first released in 1984 and usage of the model has grown steadily since then. Today, the model is recognized as one of the most precise single intersection simulation modeling tools. The model allows the user to input a wide range of parameters related to intersection geometry, signal phasing, and traffic flows.

While there are a number of different intersection approach types in the Santa Monica Freeway corridor network, there were not ample resources to calibrate the INTEGRATION model for all of these approach types. Only the most common approach type was calibrated using the model. This approach type consisted of one left-turning lane, two through lanes, and one right-turning lane. The left-turning movement at this approach was opposed by two opposing through lanes. In order to accurately simulate a wide range of intersection approach types, similar experiments should be performed on other approach configurations.

Sensitivity analysis was performed by varying the level of opposing flow from 100 vph to 2,100 vph in increments of 200 vph. The volume of the opposed left-turning movement was held constant at 150 vph. Delay to the left-turning movement, as predicted by both the SIDRA and INTEGRATION models were compared to one another. The K factors in the "OPS_FACT.DAT" file were modified with each run to force the INTEGRATION predicted delay of the left-turning movement to match that predicted by the SIDRA model.

7.2.4 Results of the Single Intersection Calibration Process

The K factors determined from this calibration effort are shown in Figure 7.3. Figure 7.4 displays the results of the single intersection calibration process.

Title Line	
1	0
2	3
3	0
4	0
5	0
6	50
7	0
8	20
9	0
10	0

Figure 7.3: Calibrated "OPS_FACT.DAT" File

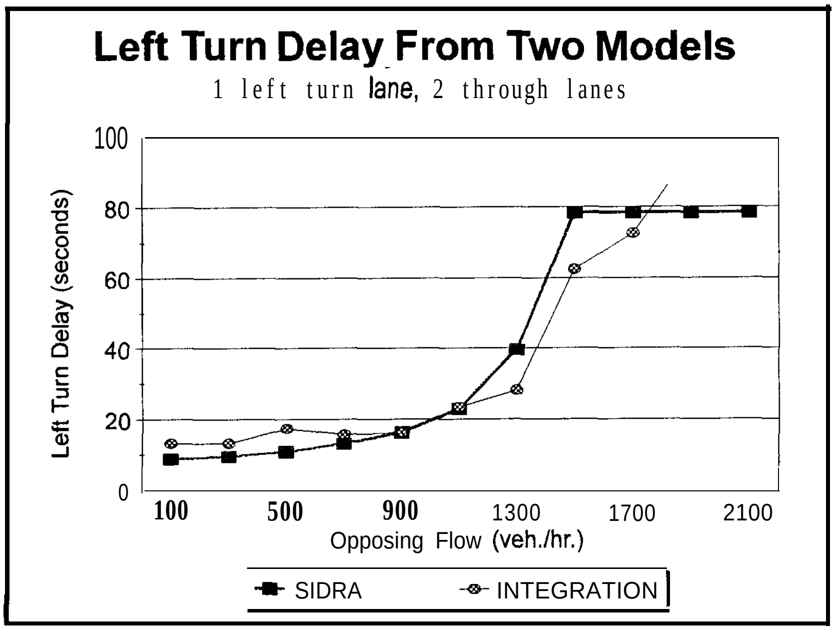

Figure 7.4: Left-Turn Delay as Predicted by SIDRA and INTEGRATION

The INTEGRATION model is a stochastic model, thus there is significant variation in the simulated delay values. SIDRA, on the other hand, is a deterministic model whose results do not exhibit significant fluctuations. The left-turn delay, as measured by the SIDRA program reaches a maximum value of 78 seconds at an opposing flow of 1,500 vehicles per hour. The reason for this maximum value is that a number of "sneakers" (usually two vehicles) will complete the left-turning movement at the end of each signal cycle. At this maximum delay point, the opposing flow is high enough so that left-turning vehicles can complete their turn only at the end of each cycle. The same effect is not seen in the INTEGRATION model. A modification to the INTEGRATION program to capture the effect of sneakers has been proposed for future versions of the model, but was not available for this research project. Overall, the estimated left-turn delay exhibited by both models for this specific intersection configuration was considered to be within a reasonable degree of error as shown in Figure 7.4

The calibration of a single expanded intersection determined precise link characteristics for each of the twelve links within the expanded intersection configuration. This calibration effort also developed and calibrated the opposing link feature of the INTEGRATION model to accurately simulate delay imposed on left-turning movements. In conclusion, this stage of the project led to an expanded intersection configuration that could more accurately simulate traffic performance at a signalized intersection.

7.3 CALIBRATING THE FIRST TIME SLICE

The calibration of a single expanded intersection and the INTEGRATION model tests were completed by July of 1994. The next step in the calibration stage of the project was to calibrate the INTEGRATION model to simulate the first time slice of the Santa Monica Freeway corridor network. The calibration of the first time slice was important in refining the demand estimation process (see Section 5.2.5). Recall that the demand estimation process must be repeated for each half-hour time slice in order to generate an origin/destination matrix for each time slice. The origin/destination matrix generated for each time slice could then be linked together to form the INTEGRATION origin/destination input file. The refinements to the demand estimation methodology developed in this section were applied to all subsequent time slices to generate an origin/destination matrix for each of these subsequent time slices.

The goal of the first time slice calibration efforts were to produce an origin/destination matrix for the first half-hour that will generate reasonably accurate traffic performance in the network. To accomplish this, the demand estimation process described in Section 5.2.5 and Figure 5.17 was refined. Sections 7.3.1 through 7.3.7 describe the refinements to the demand estimation process as it applies to the first time slice. It should be noted that there were no modifications to the control data (i.e. signal timing plans) in order to calibrate the network for the first time slice, however the control data was modified in the calibration of the morning peak period.

7.3.1 Refinement of the Demand Estimation Process

One of the most difficult aspect of generating an acceptable origin/destination matrix is that the actual origin/destination matrix itself is unknown. Recall that the demand estimation process merely uses existing link flows to develop an origin/destination matrix that will best approximate the observed link flows. Conversely, the only way to determine if an origin/destination matrix is acceptable is to see if the observed link flows are being reasonably approximated by the simulation model with this origin/destination matrix as input. It should be noted that the link flows predicted by the QUEENSOD program will differ from those predicted by the INTEGRATION program using the demand pattern, i.e. origin/destination matrix, generated by the QUEENSOD program. The link flows predicted by QUEENSOD are based on the routing trees available to the model, which remain constant for the entire time slice. Whereas, with the INTEGRATION model traffic is assigned to links based on routing trees which are updated every 60 seconds. The link flows predicted by the INTEGRATION model, which are considered to the ultimate test of a successful calibration, were used as the final test of determining how closely the simulated link flows matched the empirical link flows.

Figure 5.17 shows the demand estimation process as a streamlined, sequential process with only a single iterative loop between the dynamic origin/destination matrix and the INTEGRATION program. For time slices with heavy demands, this iterative loop was repeated until the difference between the dynamic origin/destination matrix generated in each iteration had become small enough that the demand estimation process could be considered to have converged to a final origin/destination table. For subsequent time slices, the demand estimation process did proceed relatively smoothly for each time slice. However, the execution of the demand estimation process for the first time slice proved to be very difficult. Numerous problems were encountered with the limitations of the INTEGRATION and the QUEENSOD programs. The most restrictive limitation was with the INTEGRATION model in generating minimum path routing trees. The INTEGRATION model could only handle one half-hour time slice when generating routing trees from all origins to all destinations. Also, the user could not specify specific times to output routing trees. Thus, to obtain routing trees at 29 and 30 minutes, the output from step number 3 of the demand estimation process, two INTEGRATION runs were required with each run taking approximately eight hours to complete.

A number of errors in the data collected and the coding of the network were discovered during the calibration of the first time slice that had to be corrected. Also, since the demand estimation process had never been attempted on a network this large and complicated, many decisions of exactly how the process should be performed had to be investigated and answered. Thus, a more accurate diagram of the demand estimation process, as it was applied to the first time slice, would consist of many additional iterative loops, demonstrating where problems had to be overcome and many additional arrows to illustrate the many tests that had to be performed to develop the most effective method of executing the process.

The following six subsections (7.3.2 to 7.3.7) present the efforts to calibrate the first half hour time slice of the simulation period. These efforts can also be considered the efforts to develop and refine the demand estimation process so that it can function effectively and easily for all subsequent time slices. Subsections 7.3.2 and 7.3.3 discuss the creation of the minimum path trees representing the conditions where no traffic is present in the network (step number 1 in Figure 5.17). Subsection 7.3.4 discusses the use of the QUEENSOD model to produce a static origin/destination matrix. (step number 2 in Figure 5.17). Subsection 7.3.5 presents the development of dynamic minimum path trees with the INTEGRATION program (step number 3 in Figure 5.17). Section 7.3.6 presents the use of the dynamic minimum path trees with the QUEENSOD program in developing a dynamic origin/destination matrix (step number 4 in Figure 5.17). Finally, subsection 7.3.7 discusses the final modifications that were made to the network to produce a final, acceptable INTEGRATION simulation run of the first time slice (step number 5 in Figure 5.17). Subsection 7.3.7 also presents the results of the first time slice calibration effort.

7.3.2 Creation of the Minimum Path Trees for the Empty Network

The first step in the demand estimation process is the generation of the minimum path trees that will be first input to the QUEENSOD program. This is accomplished by running the INTEGRATION program with virtually no traffic loaded onto the network. The INTEGRATION model generates two output files that contain minimum path trees, output files numbers 13 and 17. Output file number 13 contains one set of minimum path trees that are output at a user-specified interval. Output file number 17 is generated only if the simulation time specified in the master file is a negative number. This output file will generate a fixed number of minimum path trees at the end of the time slice. The number of path trees generated is specified within the INTEGRATION program and was set at three for the version of the program compiled for this research project.

The path trees generated from output file number 17 did not appear to be reasonable and were not used in this research project. For this reason, all of the minimum path trees generated for this research came from output file number 13. In order to produce a single set of path trees for a single half hour time slice, the user-specified output file interval was set to the equivalent of the simulation time. This produced a single set of minimum path trees that are based on the traffic conditions present at the end of the simulation.

Two problems encountered in creating these path trees should be mentioned. First, recall that the network was coded with only 41 of the 111 zones labeled as macro clusters. For those zones that are not macro clusters, the path trees generated will lead to the macro cluster and not to the zone itself. Specifically, the use of macro clusters results in minimum path trees being built for only one zone within the cluster (the macro destination). Thus, to create a complete set of minimum path trees that lead to each of the individual zones, the node file needed to be recoded so that each zone is a macro cluster. This node file was used whenever the goal of the INTEGRATION simulation run was to produce path trees.

While it is relatively easy to alter the node input file to create 111 macro clusters, the additional macro clusters require a new version of the program that is capable of handling 111 macro clusters. (The version of the program used for the majority of the project had a limit of 45 macro clusters.) These additional macro clusters required significantly more computer memory. Thus, the maximum number of vehicles that could be simulated by the INTEGRATION version capable of handling 111 macro cluster had to be reduced so that the model could operate on the computer that was available, which contained 64 megabytes of random access memory. The version of the program with 111 macro clusters had a vehicle capacity of 75,000 vehicles, which would only allow for the simulation of one of the time slices with the highest demand. This meant that minimum path trees would have to be generated one time slice at a time.

The other problem encountered in generating the minimum path trees was that the INTEGRATION program would not generate path trees to a destination that had no trips assigned to it. Thus, an origin/destination matrix had to be created that contained only a few trips going to each of the destinations in the network. Given the small number of vehicles on the links in the network, it was felt that the travel times on the network, and the resultant minimum path trees, would be virtually the same as if there were absolutely no vehicles on the network. The final path trees generated from the empty network are produced 30 minutes into the simulation and indicate the minimum vehicle paths present at that precise moment in the simulation.

7.3.3 Analysis of the Minimum Path Trees for the Empty Network

Initial tests with the QUEENSOD model indicated that the model output was very sensitive to the minimum path trees that were used. The reason for this is that the QUEENSOD model assumes an all-or-nothing traffic assignment, which is based on the minimum path trees, when only a single set of minimum path trees is provided to the model. Since only a single set of minimum path trees was generated for the empty network, it was very important to analyze these minimum path trees in great detail. To accomplish this, the MPT (Minimum Path Tree) program discussed in Section 3.4.3 was used extensively. This program determines the minimum path from a given origin to a given destination based on the set of minimum path trees output from the INTEGRATION program.

The minimum path for over 200 different origin/destination pairs were analyzed with the MPT program in order to determine any problems with the minimum path trees. It was found that the link volumes at many points on the freeway were very sensitive to which on-ramps and off-ramps the minimum path trees were utilizing. Among the problems encountered were links which had no minimum path trees assigned to them. The QUEENSOD program will not assign any vehicles to such a link. For example, if an on-ramp with a link flow of 500 vehicles per hour had no paths assigned to it then the estimated link volume would be zero. This discrepancy of 500 vehicles per hour would result in significant link flow errors for many adjacent links. Since the QUEENSOD program attempts to minimize the total link flow error in the system, this error would be distributed across much of the freeway mainline and many of the adjacent ramps.

The reason a ramp had no path trees assigned to it was usually due to the roadway network being configured in such a manner that using the downstream ramp resulted in slightly lower travel times for many of the origin/destination pairs that are likely to use either ramp. To relieve these problems the free-flow speeds and/or link lengths on some of the ramps and nearby surface streets were slightly modified so that all of the on-ramps and off-ramps would be utilized by at least some of the minimum path trees. Some of the problems encountered were due to input coding errors that needed to be corrected.

Much effort was put into developing a set of acceptable routing trees for the empty network since these trees would be the starting point of the demand estimation process for all of the subsequent time slices to be simulated. The final set of minimum path trees developed, when used with the QUEENSOD program, resulted in observed link flows and the estimated link flows for the mainline freeway and all of the ramps matching within plus or minus 200 vehicles per hour.

7.3.4 Tests with the QUEENSOD Model

With an acceptable set of minimum path trees for the empty network, a number of tests were performed to determine the optimum usage of the QUEENSOD model. The effect of differing the seed origin/destination matrix, the number of iterations performed by the QUEENSOD model, the link flow reliability factors, and the link flow values were all tested.

7.3.4.1 Varying the Origin/Destination Seed Matrix

Recall that QUEENSOD successively modifies an initial seed origin/destination matrix in order to minimize errors between the estimated link flows and observed link flows. If a seed origin/destination matrix is not provided to the QUEENSOD program, the program assumes a seed demand matrix where 1,000 vehicles per hour are assigned to each origin/destination pair. Given that there are over 11,000 possible origin/destination pairs in the network, this assumed seed demand matrix seemed inappropriate. Thus, the TOSEED program, discussed in Section 5.2.1, was developed to generate a seed demand matrix that represented the actual origin/destination matrix more realistically. The program works by using the volume of traffic going into or out of each zone in the network to estimate the seed demand for each origin/destination pair. The traffic volumes for the external zones where there are no traffic counts available were estimated based on other arterial link flows in close proximity to the zone. The traffic volumes into and out of the internal zones were based on the amount of residents and employees in the census tract corresponding to that zone.

The TOSEED program allows the user to specify an overall scaling factor that is applied to each seed value. The effect of a uniform scaling of the seed origin/destination matrix was very small. For example, a seed matrix with all of the demand values four times higher than another demand matrix would produce a very similar origin/destination matrix. The fact, that uniform scaling of the seed origin/destination matrix had no impact on the estimated origin/destination matrix is a feature of the QUEENSOD model, which mathematically can be shown to not only be desirable but also required. It is also noted in the literature that the generation of different estimates for uniformly scaled origin/destination matrices is a limitation of many other models. Thus, the fact that the QUEENSOD model produces the same output for differently uniformly scaled seed inputs is not a problem with the QUEENSOD model.

For origin/destination pairs whose minimum path trees do not use any links with observed link flows, there is no indication whether the seed value is too high or too low. Thus, the QUEENSOD program would not change the seed value for this origin/destination pair at all. These instances were felt to be rare and not to have significant influence on traffic performance in the network. The final decision was to use a low seed matrix value so that in these instances only a few vehicles would be assigned to the origin/destination pair.

7.3.4.2 Varying the Number of Iterations

The advantage of using a large number of iterations is that the QUEENSOD program will have more opportunity to adjust the origin/destination matrix and hence reduce the difference between the observed and estimated link flows. The disadvantage to the additional iterations is the increase in run time for the QUEENSOD simulation runs. Given that a 30 iteration run of QUEENSOD on a 486SX 50 MhZ personal computer takes over 90 minutes and the large number of QUEENSOD runs to be performed, the latter was a significant concern.

Simulation runs were made using 30, 45 and 60 iterations. The output from these runs indicated that there were minor but significant differences between the 30 and 45 iteration runs. The difference between the 45 and 60 iteration runs was also noticeable but even less significant than the previous difference. Thus, the decision was made to use 45 iterations for all subsequent runs of the QUEENSOD model.

The model was also tested to determine if two simulation runs of 30 iterations each in series would produce the same results as a single 60 iteration run. The QUEENSOD runs were conducted in series by taking the output (origin/destination matrix) from an initial 30 iteration run was using it as the input (seed origin/destination matrix) to another 30 iteration run. The output from the second QUEENSOD run was then compared to the output from a single 60 iteration run. The results of these runs were nearly identical.

7.3.4.3 Varying the Link Flow Reliability Factors

Another way to vary the input to the QUEENSOD program is to vary the link flow reliability factors for the link flows that are provided in input tile number five. Varying the link flow reliability factors allows the user to accommodate the knowledge that some data are more reliable than others. By default, the QUEENSOD program assumes that all link flows are equally reliable (all link flow reliability factors are equal to 1.0). The user may specify that some of the links are less reliable than others by assigning a lower link flow reliability factor to those links. The QUEENSOD program will put less weight on the link flow error for links with lower reliability factors when minimizing the mean of the absolute squared difference between the observed link flows and the estimated link flows.

QUEENSOD runs were performed with all of the arterial link flow reliability factors set at 0.2, 0.5, and 0.8. Only the arterial link flow reliability factors were reduced since the freeway and ramp link flow data was considered more reliable than the arterial link flow data. The output from these runs had only minor differences. The cause of this is expected to be the same reason that the seed matrix has little effect on the final origin/destination matrix that is generated. Given the large number of iterations, the program will still arrive at nearly the same final demand matrix. The only difference the link flow reliability factors cause is the rate at which this final matrix is arrived at. Thus, the final decision was to use a link flow reliability factor of 1 .0 for all of the observed link flows.

Significant difficulties arise from the fact that synthetic origin/destination demands need to be generated for a network within which many of the flows were unknown, and in which the absolute magnitude of links may vary by up to an order of magnitude (i.e. flow on a freeway link may be ten times greater than the flow on a minor arterial). The modeling problems due to a lack of arterial link counts was discussed in Section 7.2, while the impact of varying magnitudes of flows is discussed below.

For example, the impact of an error of 350 vph in a freeway of 7,000 vph may be less severe than the impact of a similar error of 350 vph on a 200 vph turning movement at an intersection. However, in most synthetic origin/destination models, these similar absolute errors are treated as being equivalent. Similarly, an error of 3,500 vph on a freeway flow of **7,000** vph may produce a significantly greater modeling error than an error of 100 vph on a 200 vph turning movement flow. In most synthetic origin/destination models these relative errors are considered as being equivalent. As a result, not only is there a need to have accurate and complete inputs (observed link flows) into the synthetic origin/destination generation process, but it is important to be able to reliably trade-off errors on different parts of the network, even though the relative/absolute errors may be similar. As shown above, even though QUEENSOD provides an ability to make either absolute or relative trade-offs, neither approach is fully satisfactory. Thus, in hind site, further experimentation with adjustments of the arterial link flow reliability factors would have probably improved the demand estimation process. This improvement in the demand estimation process would have probably led to a more accurate origin/destination matrix, which would have produced a more reasonable calibration. The need for a more accurate demand origin/destination matrix became more apparent in the calibration of the morning peak period, as will be discussed in Section 7.4

7.3.4.4 Varying the Link Flow Values

The arterial data collection stage of the project did analyze the validity of the arterial link flows obtained from the ATSAC system, however it was not possible to analyze the validity of each link flow in detail given the large number of observed arterial link flows in the network. At many points in the network, an observed link flow that was half or double the actual link flow might have passed through the initial validity check. By using the QUEENSOD program, it was possible to more thoroughly analyze the validity of the observed arterial link flows in order to identify which links have suspicious observed link flows.

This part of the calibration process consisted of comparing the observed link flows with the link flows estimated by the QUEENSOD program. It should be noted that while this test is important, the critical test will be to compare the observed link flows with those estimated by the INTEGRATION program. Remember that the QUEENSOD program assigns vehicles to links using a single set of path trees to determine the estimated link flows. On the other hand, the INTEGRATION program assigns vehicles using path trees that are updated every 60 seconds to reflect the current traffic conditions in the network. The QUEENSOD estimated link flows were compared to observed link flows because it was discovered that while the link flows estimated by both programs were different, gross errors in the QUEENSOD program would usually lead to corresponding errors in the INTEGRATION estimated link flows.

The method of comparing the estimated link flows to the observed link flows utilized QUEENSOD output file number ten, which lists the observed link volumes (where present), the estimated link volumes (based on the final origin/destination matrix and the path trees provided), and the difference between the two. This file was imported into a spreadsheet and the links were ordered by the absolute difference between the estimated and observed link flow. Any differences that were greater than 100 vehicles per hour in absolute magnitude were analyzed in detail. This detailed analysis compared the link flow in question to the neighboring link flows. This process allowed for the discovery of certain links that were most likely erroneous. In these instances, the observed link flow was discarded and the simulation was performed again. The effect of eliminating a suspect link flow on the final origin/destination was often quite dramatic.

An example of this dramatic change in the final origin/destination table predicted by QUEENSOD was the impact of eliminating the observed flow on one link (link 780). With the observed link flow inputted to the QUEENSOD model the total demand generated at zone 33 was 1,052 vph. When the observed flow was not inputted into the QUEENSOD model the total demand generated at zone 33 was 1,500 vph, an increase of almost 50 percent.

The fact that slight modifications in the observed link flows has a dramatic influence on the final origin/destination matrix generated by the QUEENSOD model highlights the need to have accurate observed flows for all turning movements for all approaches to a signalized intersection in order to reasonably model traffic performance at an expanded signalized intersection.

In summary, varying the seed origin/destination matrix, the number of QUEENSOD iterations, and the link flow reliability factors had relatively no effect on the final origin/destination matrix, while slightly modifications in the observed link flows would often have radical results.

7.3.4.5 Comparison of Total Origin and Destination Traffic Volumes

As mentioned, the actual origin/destination matrix of the network is unknown. However, there are certain checks that were performed on the output from the QUEENSOD model to assess the validity of the final origin/destination matrix. The first test was to determine if the total number of vehicles assigned to each origin and destination agreed with the link flows into and out of them. The second test was to analyze the difference between the observed and QUEENSOD estimated link flows for all of the links in the network where observed link flows exist.

Output file number nine from the QUEENSOD program contains the final origin/destination matrix tile that may be used as an INTEGRATION input file number four. Analysis of this output file is difficult with a network the size of the Santa Monica

Freeway corridor due to the fact that it contains over 10,000 lines of data. The ODDATA program, refer to Section 3.4.2, was developed to provide summary statistics from this file. From a user specified list of origins and destinations, the ODDATA program sums all of the vehicles leaving each specified origin and arriving at each specified destination. Link flow data that gives the amount of traffic that should be entering the network via a zone (i.e. traffic enters the network from the origin via a single link whose link flow is known) was available for 22 of the 48 external origins. The output from the ODDATA program for these 22 external origins was compared to the known flow leaving the origin. Figure 7.5 shows the results of this comparison. Estimated demands on the dark line represent a perfect match. Estimated demands that lie between the two light lines are within 10 percent of the observed demand. The number associated with each of the data points in the figure are the origin zone numbers. Note that the number of vehicles assigned from each origin from the QUEENSOD program compares reasonably well with the observed values for these 22 origins, but the produced demands were generally less than the known demands, Observed link flow data was not available for the other 26 external origins and for none of the 56 internal zones to perform a similar comparison.

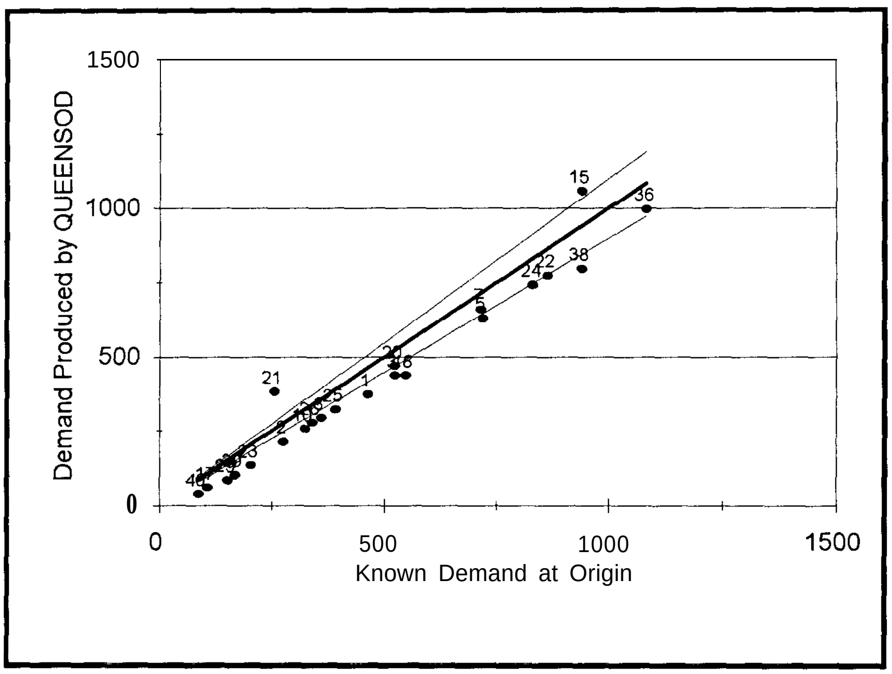

Figure 7.5

Comparison of Traffic from Various Origins - Observed vs. Estimated

There were no destinations where the precise traffic volume to the destination was known. However, there were 24 zones where the link volumes in proximity to the destination zone were known. These link volumes allowed for a rough approximation of the volume exiting the network via the given destination. Figure 7.6 shows the results of this comparison. Again the estimated demands on the dark line perfectly match the observed flow leaving the network and estimated demands that lie within the two light lines are within 10 percent of the observed flow leaving the network. The number associated with each of the data points in the figure are the destination zone numbers. Note that the difference between the observed and estimated link volumes is significantly greater for the destination volumes than it was for the origin volumes.

This difference is due to three factors. The first being errors in approximating the destination flow. The second being errors in the QUEENSOD estimated link flows. Recall that the QUEENSOD estimated link flows are based on a single set of minimum path trees for the entire half-hour time slice. Whereas the INTEGRATION estimated link flows are base on routing trees which are updated every 60 seconds. Thus, some errors are expected in the flows estimated by the QUEENSOD model. The third and most significant factor is the fact that observed traffic volumes leaving at a destination may not be the demand for that destination. Note that the observed traffic volume entering at an origin could be accurately described as demand, but the observed traffic volume leaving the network at a destination can definitely not be described as demand. Observed flows to a destination reflect the number of vehicles that reach the destination during a given time slice, which is not necessarily the volume that wanted to (i.e. demanded that destination). Whenever there was congestion in the network, cars were delayed and observed volumes exiting at a given destination are not a true reflection of the demand for that destination.

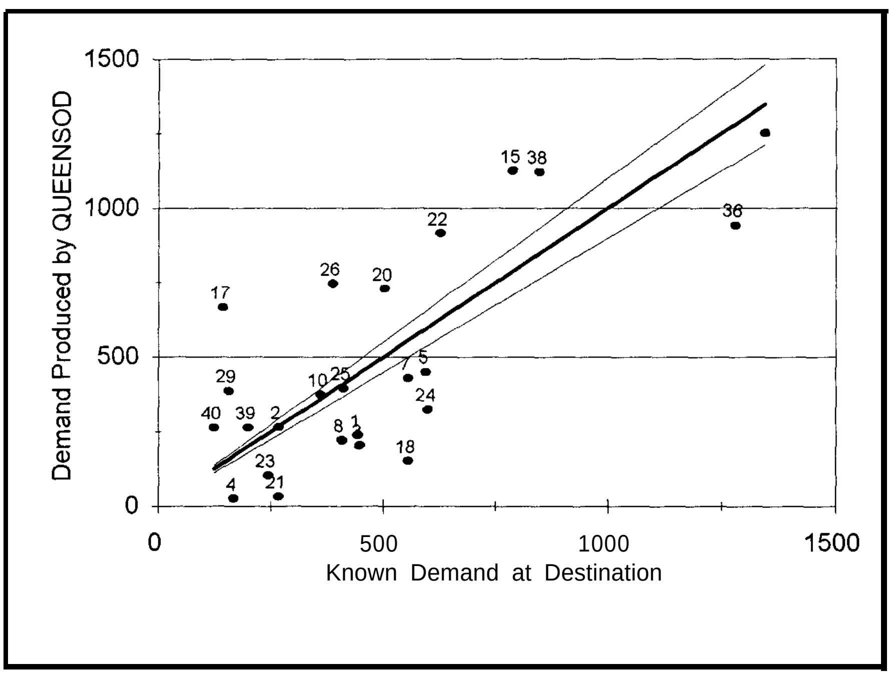

Figure 7.6

Comparison of Traffic from Various Destinations - Observed vs. Estimated

7.3.5 Routing Trees with Traffic on the Network

The result of the tests with the QUEENSOD program in the previous section was an acceptable origin/destination matrix for the first half hour time slice. The next step in the demand estimation process for the first time slice was to use this matrix as input to the INTEGRATION program and develop minimum path trees that represent vehicle routings with the appropriate traffic loaded onto the network (step number 3 in Figure 5.17). The same process of having the INTEGRATION program output a single set of path trees via output file number 13 was used to generate minimum path trees for the network with traffic loaded onto it.

Section 7.3.5.1 discusses the generation of a set of minimum path trees for the loaded network. Section 7.3.5.2 discusses the advantages of using multiple minimum path trees and why it was decided to use two minimum path trees.

7.3.5.1 Dynamic Minimum Path Trees

The major difference between generating minimum path trees with and without traffic on the network is that the minimum path trees for a network with vehicles loaded onto it are dynamic. As the number of vehicles on a link changes significantly, so does the travel time. Since the minimum path trees are based on link travel times the minimum path trees for the empty network will be significantly different from the minimum path trees for the loaded network.

The calibration effort was conducted entirely using INTEGRATION runs with 100 percent type 2 vehicles, which are "guided" vehicles (see Section 3.6.3), with a minimum path update time of 60 seconds. Recall that type 2 vehicles have access to updated minimum path trees and will perform dynamic traffic assignment based on these trees. The distortion factor of the information, refer to Section 3.6.4, provided to these vehicles was zero. This results in a dynamic assignment of vehicles identical to the situation where all vehicles have perfect information on minimum path trees that are updated every 60 seconds. It is important to note that using 100 percent type 2 vehicles and perfect information represents the condition, under recurring congestion, where all vehicles have had ample opportunity to determine their minimum paths and do use these paths. Therefore, it is implicitly assumed throughout the calibration effort that the observed link flows were generated by commuters familiar with the recurring traffic congestion patterns.

The dynamic path trees created using the loaded network were analyzed in the same manner as the static path trees. Roughly 20 path trees were identified from the static path tree analysis as critical and prone to problems. These path trees were investigated in order to determine whether problems existed with the minimum path trees generated for the loaded network. Some problems were identified and were relieved with minor modifications to the arterial and on-ramp free-flow speeds. This stage of the calibration process was very time consuming and required numerous INTEGRATION simulation runs.

7.3.5.2 Using Multiple Minimum Path Trees

It was quickly discovered that dynamically generated path trees were far less predictable than static trees. For example, in order to force a minimum path tree to use one of two alternate paths in a simulation run with no traffic, the free-flow speed could be adjusted to create the desired minimum path tree. On the other hand, in a network loaded with traffic, the minimum path tree will oscillate between two alternate paths with nearly equivalent travel times. As vehicles are loaded onto one path, the travel times increase and eventually the minimum path becomes the alternate path. The alternate path is then loaded to the point where it is no longer the minimum path tree and this oscillating process continues.

During a 30 minute simulation with the minimum path trees updated every 60 seconds, and given the complexity of the network and the potential for competitive alternate paths, the degree in which changes can occur in the minimum path trees throughout the simulation is undoubtedly enormous. Thus, the path trees that happen to be in effect at the end of the 30 minute simulation with a loaded network are somewhat random in nature and may not reflect the path trees that are present most often during the simulation period. The use of only a single minimum path tree has been shown to create considerable calibration errors.

It was determined that the best method of overcoming this problem was to use more than a single set of minimum path trees. Initially three INTEGRATION simulation runs were completed to generate three sets of path trees; one at 28 minutes, one at 29 minutes, and one at 30 minutes into the simulation. The three INTEGRATION output number 13 output files were combined manually using the DOS copy command. Before the separate files were combined, they had to be edited using a conventional disk editor. The header lines of the last two path tree files had to be eliminated, and the proportion of demand using each path tree had to be changed from 1.00 to 0.33 for each of the three path tree files. This yielded a viable minimum path tree file with three sets equally utilized minimum path trees.

The computer memory requirements for simulating a network as complex as the Santa Monica Freeway corridor were such that the QUEENSOD program could not simultaneously accommodate multiple time slices and multiple sets of path trees. Working with the developer of the QUEENSOD program, a version of the QUEENSOD program was compiled that could handle multiple path trees, but would only simulate a single half-hour time slice of the Santa Monica Freeway corridor network. This reaffirmed the decision to execute the demand estimation process a single time slice at a time for all subsequent time slices.

An analysis of the QUEENSOD program to determine the difference in the output from the QUEENSOD program when using one, two and three sets of minimum path trees was conducted. The QUEENSOD run with only one minimum path tree used a minimum path tree generated at 30 minutes into the simulation. The QUEENSOD run with two minimum path trees used minimum path trees that were created at 29 and 30 minutes into the simulation. Finally, the QUEENSOD run using three minimum path trees used minimum path trees that were generated at 28, 29 and 30 minutes into the simulation. The measure of effectiveness was the ability of the QUEENSOD program to minimize the link flow error. The final link flow error from the QUEENSOD demand matrix estimation process, as well as the absolute difference between the QUEENSOD estimated link flows and the observed link flows, were analyzed

The results indicated that the addition of a second set of minimum path trees improved the QUEENSOD model's ability to minimize the mean of the absolute squared difference between the observed link flows and the QUEENSOD estimated link flows. Interestingly, the addition of a third set of minimum path trees did little to decrease the link flow error, and in many instances actually increased it. Given the additional effort of generating a third set of minimum path trees, the final decision was to use two sets of path trees, which are generated 29 and 30 minutes into the simulation, as the dynamic path trees for each of the time slices to be simulated. The use of two minimum path trees significantly improves the QUEENSOD model's performance and avoids the problems associated with an all-or-nothing traffic assignment.

Minimum path trees at 29 and 30 minutes into the simulation are likely highly correlated, since any congestion within the network is unlikely to change significantly during this short a time frame. Ideally, minimum path trees would be outputted with a longer time frame between them. The network must be loaded with vehicles and be at or close to the user travel time equilibrium when the minimum path trees are generated. Thus, a simulation to generate minimum path trees should initially simulate a warm-up time slice to fill the network with vehicles and then simulate the time slice in which minimum path trees are to be generated for. With this approach minimum path trees could be generated with a longer time frame between them, say. at 20 and 30 minutes into the simulation. However, the INTEGRATION version that outputted minimum path trees could only simulated one time slice at a time. Although minimum path trees generated at 29 and 30 minutes into the simulation are probably highly correlated, generating minimum path trees at the end of the time slice allowed the network to fill with vehicles and for the network to approach the user travel time equilibrium.

Further testing to determine the optimal number of minimum path trees and optimal generation time of those minimum path trees would surely have resulted in a better set of minimum path trees to input to the QUEENSOD model. Since these tests are very time consuming and due to limited project time and funding, further work to refine this step of the demand estimation process was not possible.

7.3.6 QUEENSOD Runs with Multiple Dynamic Minimum Path Trees

This section of the calibration process presents the results of the QUEENSOD run with the routing trees developed in the previous section. This QUEENSOD run is step number 4 of the demand estimation process, refer to Figure 5.17. The effort to generate acceptable multiple dynamic minimum path trees for the first time slice resulted in the generation of an origin/destination matrix which produced reasonable QUEENSOD estimated link flows. Usually links with an absolute difference between the QUEENSOD estimated link flow and the observed link flow that was less than 250 vehicles per hour were considered reasonable. However, a QUEENSOD run which had a few links with an absolute difference between the estimated and observed flow that was greater than 250 vehicles per hour, but less than 500 vehicles per hour, would also be considered reasonable.

A portion of the link flow errors for the final simulation run of QUEENSOD for the first time slice are shown in Figure 7.7. Note that this figure only contains the links with QUEENSOD estimated link flows that are overestimated (observed minus estimated link flow is negative). Only a total of two links in the figure have an absolute difference greater than 250 vehicles per hour.

Listing of estimated and actual link flows

Link flows resulting from estimated O-D
- Note: Links not listed have both no observed nor estimated flow.

Period	Link	From	To	Cap	Obs	Est	(obs-est)
1	870	652	581	4000	840	1120	-280
1	736	153	820	4000	432	703	-271
1	1222	236	237	8200	5608	5839	-231
1	1221	235	236	10000	6158	6382	-224
1	951	176	1620	4000	828	1010	-182
1	959	40	1886	10000	940	1099	-159
1	1279	283	1915	3600	916	1072	-156
1	1205	220	221	10000	7358	7511	-153
1	355	633	1813	6000	732	882	-150
1	141	361	778	4000	360	494	-134
1	861	18	1564	10000	707	840	-133
1	916	1720	1353	6000	925	1045	-120
1	1180	199	200	12600	8446	8566	-120
1	1136	1361	1366	4000	400	519	-119
1	1181	200	201	13601	8190	8305	-115
1	1179	198	199	10998	7364	7473	-109
1	1186	205	206	11400	8842	8951	-109
1	1184	203	204	13300	8842	8951	-109
1	1185	204	205	13300	8842	8951	-109
1	1190	207	208	11100	9106	9207	-101
1	565	1648	807	4000	304	404	-100

Figure 7.7: Link Flow Errors from QUEENSOD - Time Slice One

7.3.7 Final Calibration of First Time Slice

The final origin/destination matrix generated above was then run with the INTEGRATION program. This is step number 5 in the demand estimation process, refer to Figure 5.17. One iteration of the loop in the demand estimation process was completed to determine the impact of the iteration. However, the results discussed in this section are from the INTEGRATION run using the dynamic origin/destination matrix generated in Section 7.3.6.

To test the impact of the iterative loop shown in the demand estimation process, the final origin/destination matrix for the first time slice generated in Section 7.3.6 was input to the INTEGRATION model and steps number 3 and 4 of the demand estimation process were repeated. Comparison of the dynamic origin/destination table generated from this iterative loop of the demand estimation process to the dynamic origin/destination table generated in Section 7.3.7 revealed only minor differences. Due to the effort required to complete the iterative loop of the demand estimation process, the iterative loop should only be followed in time slices with heavy demand patterns.

7.3.7.1 The Initial Warm-Up Time Slice

One problem with running the first time slice, or any time slice by itself, is that the network begins empty. Any statistic taken over the half hour period, such as link flow, will include the beginning of the simulation period where the network is empty. Of course, the time period when the network is being loaded should not be considered in the calculation of link statistics. Unlike minimum path tree output, which represents instantaneous conditions, the link flow statistics are averages over a user-specified period of time, one half hour in this case.

To eliminate the impact of the initial loading of the network in the calculation of link summary statistics, a warm-up time slice was added to all simulation runs. This time slice was always identical to the first time slice that was to be simulated. A simple program was developed that allows the user to specify a list of origin/destination matrices in a certain order, and then it creates a single origin/destination matrix file, which is INTEGRATION input file number 4, that contains all of the time slices requested in the specific order requested. For simulation runs of the first time slice, the program was used to create the INTEGRATION input file number 4 that contained two identical matrices, each with the demand for the first half-hour time slice.

The problem of network loading was also observed in the. transition period between two different time slices. For example, if the freeway mainline flow doubles from the first time slice to the second, the second time slice begins with the lower level of flow. Given that the freeway is in the center of the network, it may be several minutes before the increased number of vehicles from the second time slice arrive at the freeway links. This resulted in an average link flow value for the second time slice that is significantly below the observed link flow value for the second time slice. This problem was circumvented by specifying that the program produce link flow summary data every 10 minutes as opposed to every 30 minutes. Varying the interval at which this output is produced has no effect on the simulation itself The summary statistics, such as estimated link flows, were analyzed for the last ten minutes of the time slice. For the simulation of the first time slice, which was actually a one hour run with the warm-up time slice, the output from the last ten minutes of the second time slice simulated was analyzed.

7.3.7.2 Tests of Vehicle Routing within INTEGRATION for the First Time Slice

The minimum path trees generated by the INTEGRATION model changed every 60 seconds. Thus, a half hour simulation would produce a set of 30 different minimum path trees. Clearly, all these minimum path trees could not be analyzed in detail. Thus, methods had to be developed to determine if vehicles were being properly routed through the Santa Monica Freeway corridor for the first time slice.

The goal of analyzing the vehicle routing was to check if the vehicle assignment between the freeway and arterial routes was reasonable. There was also concern that the total number of vehicles assigned may be too large or too small. The most obvious test of the freeway/arterial assignment was to compare the arterial and freeway link flow volumes crossing a cordon. To analyze the vehicle routing and total network traffic flow, a cordon count analysis was performed at four locations in the network. Each cordon was defined by a continuous north-south line. The cordons were selected so that each eastbound and westbound link crossed by this line had an observed flow. This analysis summed all of the arterial and freeway estimated link flows in both the eastbound and westbound directions and comparing these values to the corresponding observed link flow sums. The difference between the estimated and observed sums of eastbound and westbound traffic at the four locations were used to determine if too much or too little traffic was on the network. The estimated and observed traffic split between the arterial and the freeway in both the eastbound and westbound directions was compared to determine if too much to too little traffic was being routed along the arterial routes. Too much traffic on the arterial routes in a given direction implies that too little traffic is assigned to the freeway in that direction, and vice versa.

This analysis indicated that vehicles were being under-assigned to the freeway and over-assigned to the arterial streets. To overcome this problem the free flow speed on most of the major arterials was reduced.

7.3.7.3 Results from the INTEGRATION Simulation of the First Time Slice

The RED12 program, refer to Section 3.4.4, was used to create a subset of the extensive output files that are created by the INTEGRATION program. This condensed file could then be imported to a spreadsheet so that graphs of link flow along each of the arterials and the mainline freeway for both the eastbound and westbound directions could be quickly prepared for each simulation run. Given the number of simulation runs that were made in the calibration process, the development of a spreadsheet that could quickly create graphs of performance along the freeway or an arterial proved to be very effective in allowing for the efficient analysis of the INTEGRATION output files. The spreadsheet was developed so that an output file reduced by the RED 12 program could be imported and graphs of observed versus estimated link flow, such as Figure 7.8, could be created for the freeway mainline, the freeway on-ramps and off-ramps, and many of the major arterials in the network.

Figure 7.8 shows the observed versus INTEGRATION estimated link flows for the links that comprise the westbound portion of the mainline freeway for the first time slice. The increasing link numbers from left to right indicate progression along the freeway from east to west. Note that the fit between the two is within an acceptable level of error, never deviating by more that 350 vehicles per hour. It was felt that estimated link flows for the freeway mainline that were within 500 vehicles per hour of the observed link flows constituted a reasonable calibration. For the first time slice, the westbound mainline freeway required relatively little modifications during step number 5 of the demand estimation process to reach an acceptable calibration.

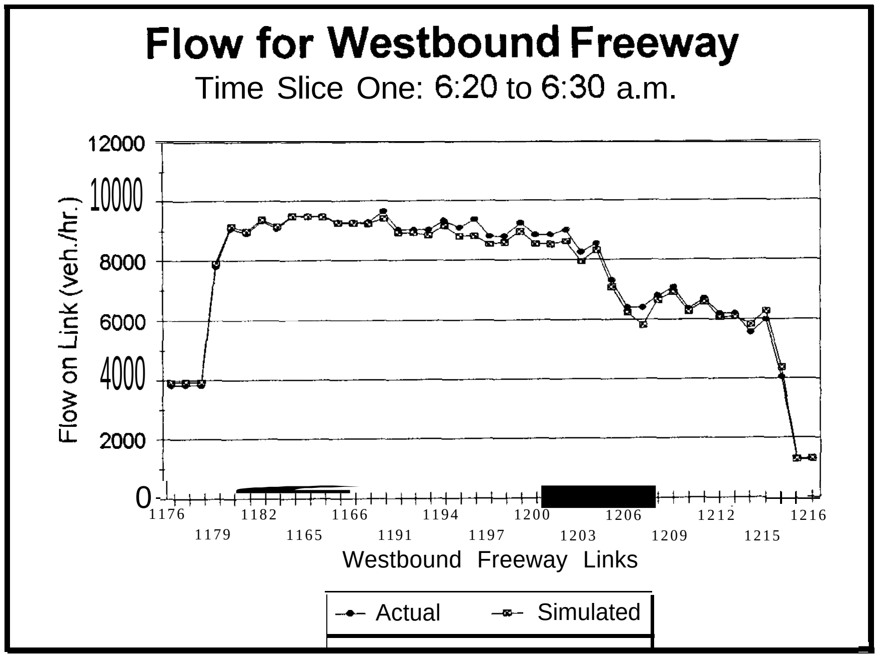

Figure 7.8: Time Slice One Link Flow Analysis - Westbound Freeway

Figure 7.9 shows the observed versus INTEGRATION estimated link flows for the links that comprise the eastbound portion of the mainline freeway for the first time slice. The increasing link numbers from left to right indicate progression along the freeway from west to east. Note that for the western portion of the eastbound freeway, the match between observed and estimated link flow values is rather good. However, between link numbers 1235 and 1237 the simulation run fails to load roughly 400 to 500 vehicles per hour onto the freeway. This gap between observed and estimated link flows continues along the eastbound mainline freeway until the end of the network. The difference between the observed and estimated link flows is greater than 500 vehicles per hour for some of the eastern portion of the eastbound freeway. Further modifications to the network coding during the refinement of step number 5 of the demand estimation process could not improve the INTEGRATION predicted traffic performance along the eastbound freeway.

This problem of a drop in the link flow on the eastbound freeway near links 1234 and 1237 (the Washington Boulevard/La Cienega Boulevard interchange) proved to be a persistent problem throughout the entire calibration effort that could not be easily solved.

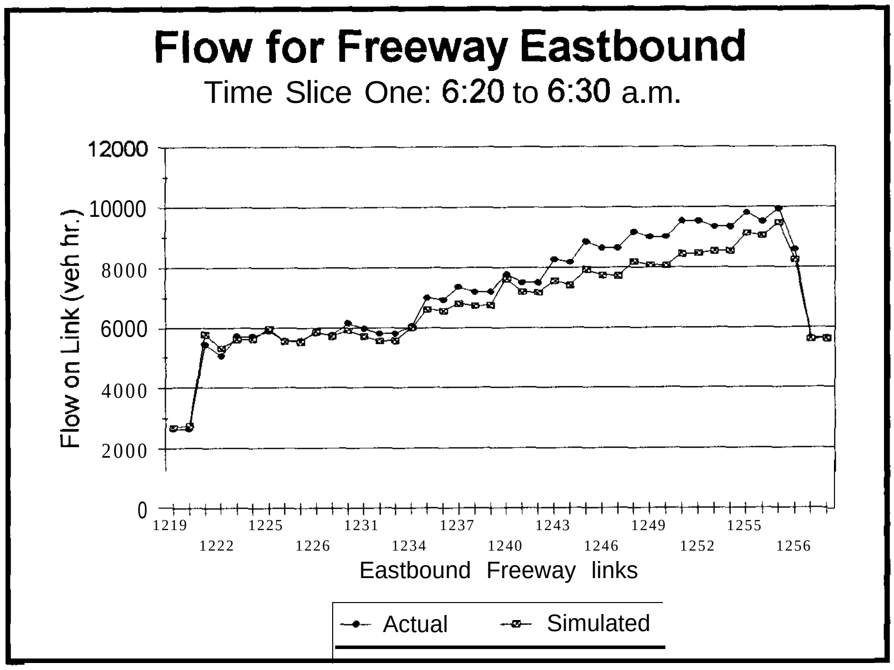

Figure 7.9: Time Slice One Link Flow Analysis - Eastbound Freeway

Figures 7.10 and 7.11 show similar graphs for two of the major arterials in the network. Figure 7.10 shows the flow along Olympic Avenue eastbound for the time period 6:20 to 6:30 a.m., which is the last ten minutes of the first time slice. Unlike Figures 7.8 and 7.9, Figures 7.10 and 7.11 also show the link flow values that were estimated by the QUEENSOD program. These figures highlight the differences between the QUEENSOD estimated link flows and the INTEGRATION estimated link flows. Note that the deviation between the INTEGRATION estimated link flows and the actual counts are fairly small for most links with the exception of link number 66. Thus, the estimated link flows do not perfectly match the observed link flows along Olympic Avenue eastbound, but for the most part they are fairly reasonable.

Figure 7.11 shows the flow along Venice Boulevard eastbound for the time period 6:20 to 6:30 a.m. Note that the simulated link flow values are usually within 250 vehicles per hour of the actual counts with the exception of link number 293. At links 292 and 293 there is a dramatic increase in the simulated flow along Venice Boulevard eastbound. These two links are part of a path where vehicles destined for the eastbound freeway may postpone their entrance to the freeway by a single interchange. This dramatic increase in flow along Venice Boulevard eastbound was found to be associated with the shortage of vehicles entering the eastbound freeway mentioned above. Thus, examining the graphs of freeway and arterial flow together proved very useful in calibrating the first and subsequent time slices.

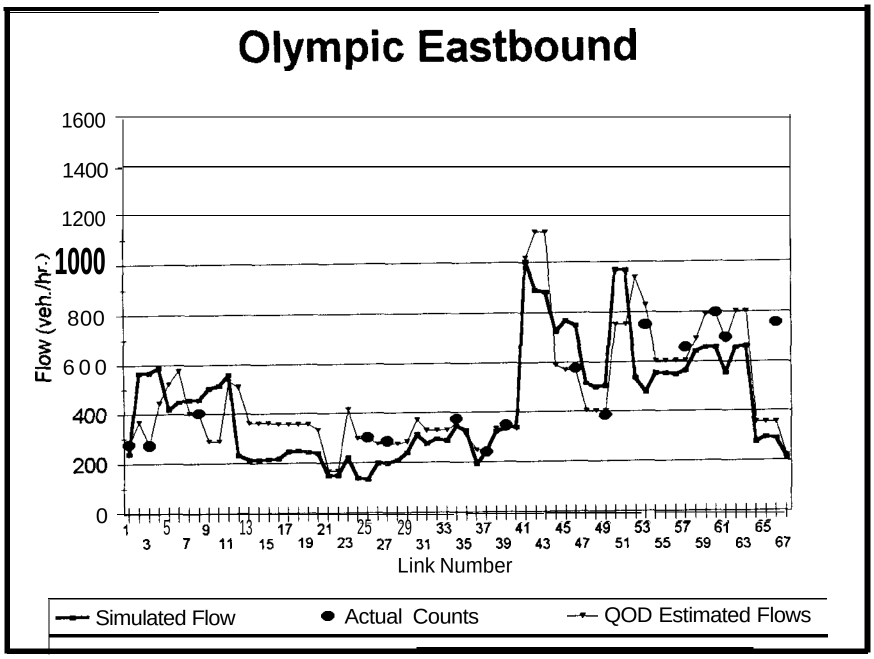

Figure 7.10: Time Slice One Link Flow Analysis - Olympic Blvd. Eastbound

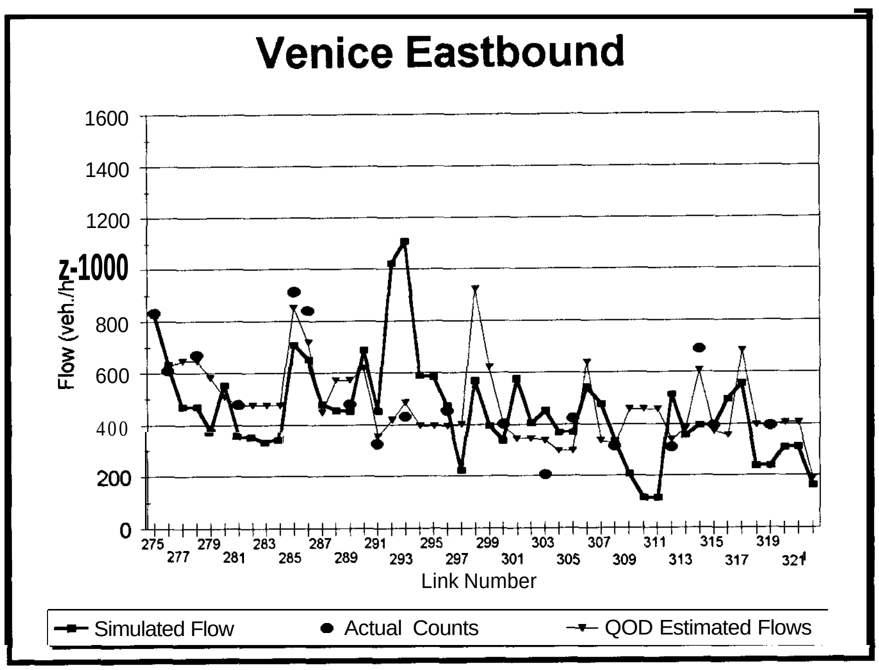

Figure 7.11: Time Slice One Link Flow Analysis - Venice Blvd. Eastbound

The analysis of the INTEGRATION simulation of the first time slice also made use of the link summary statistics contained in the INTEGRATION output, such as link speed, travel time, and volume-to-capacity (v/c) ratio. The RED12 program reads all the link summary statistics (INTEGRATION output tile number 12) and produce files that contain all of the links that are under the given speed, over the given delay, and under and over two given v/c ratios. The RED12 program allows the user to specify the speed, the delay value, and the two v/c ratios which are used as threshold values by the RED12 program. This proved to be an effective way of determining problem areas in the network where the delay was excessively high.

The lack of detailed empirical data on link travel times or delays meant that these statistics could only be used to determine gross errors and not as a precise calibration tool. For the first time slice, the program was able to determine that there were no links in the network where the delay was over 100 seconds and no links where the v/c ratio was greater than 0.85.

7.3.8 Conclusion of the Calibration of the First Time Slice

This attempt to calibrate the INTEGRATION model for the first early-morning uncongested time slice required three months of effort on the part of the research team. While the final calibration efforts were marginally acceptable, more time on this first time slice calibration effort would have improved the realism of the simulation. However considering the amount of future calibration effort remaining for all of the other time slices and the remaining limited project time and resources, it was decided to stop further work on this time slice and move ahead to additional time slices.

A major accomplishment was the identification of problems in the modeling effort, and incorporating refinements in the demand estimation and traffic simulation process in an attempt to overcome or minimize these problems. The research team worked very closely with the model developer and both participated in these model refinements.

It should be noted that the refinement of the demand estimation process was based on work completed by the research team with guidance from the INTEGRATION model developers. The INTEGRATION model developers did not recommend this refined demand estimation process to the research team. The refined demand estimation process detailed in this section was not discussed in any of the literature on the previous applications of the INTEGRATION model available to the research team.

7.4 CALIBRATING THE MORNING PEAK PERIOD

The calibration of the morning peak period was done incrementally. First, the origin/destination matrix for the second time slice (6:30 - 7:00 a.m.) was created and added to that of the first time slice. Simulation runs were then made with time slices one, one, and two (the first time slice being the warm-up slice). Before proceeding towards the calibration of the third time slice, it was ensured that the second time slice was fairly accurately calibrated. This process was then repeated for the third (7:00 - 7:30 a.m.) and fourth (7:30 - 8:00 a.m.) time slices. The calibration of the fourth time slice was marginally reasonable at best. The calibration of the fifth (8:00 - 8:30 a.m.) and sixth (8:30 - 9:00 a.m.) time slices was also attempted, but reasonable traffic performance could not be obtained.

Efforts to generate the dynamic origin/destination matrix for time slice two, which began in October of 1994, were considered the starting point of the morning peak period calibration effort. The calibration of the morning peak period continued at U.C. Berkeley until February of 1995. After an extensive calibration effort, the research team at U.C. Berkeley was unable to achieve a reasonable calibration for time slices four, five and six. Efforts to calibrate the morning peak period were continued at Queen's University from February of 1995 to June of 1995. It should be noted that the model developers were not under contract to perform modeling for this research project. The recommendations that resulted from the model developer's efforts were discussed in Section 7.1. The output from the simulation runs conducted by the model developer are not reported in this section as they were not received by the U.C. Berkeley research team.

The refined demand estimation process created in the previous section was used to generate the dynamic origin/destination tables for each of the morning time slices. The linking of the various origin/destination matrices into a single, multiple time slice, origin/destination matrix to form INTEGRATION input file number 4 was accomplished using the utility program described in Section 7.3.7.1.

This incremental approach to the morning peak period calibration simplified the calibration effort. Most of the gross errors in the input data coding were identified during the calibration of the first time slice, as discussed in the previous section, however a few gross errors were identified during the calibration of the second time slice. Once the gross errors from one run were identified and the input coding modified, another INTEGRATION run was required. This run also required analysis to identify any gross errors. This process was repeated until all of the gross errors in the input data coding were eliminated.

Since the traffic demand was relatively light in the first two time slices, an INTEGRATION run for these two time slices could be completed overnight. Thus, gross errors in the input coding could be identified relatively quickly. On the other hand, if the morning peak period had began with the analysis of all eight morning time slices, with an eight time slice INTEGRATION run taking approximately six days to complete, the identification of gross errors would of been much slower.

This incremental approach to calibrating the Santa Monica Freeway corridor network for the morning peak period also proved advantageous due to the fact that each time slice from one to five had an increasingly larger traffic demand, with the traffic demand for time slice six being just slightly lower than that of time slice five. The number of problems in calibrating each time slice increased significantly as time slices with larger demands were considered. As more and more traffic is assigned to the network, more and more instances of excessive congestion were revealed, far greater than the level of congestion existing in the field. The incremental calibration approach for the morning peak period addressed the problems in a given slice before considering the next time slice. By incrementally adding traffic to the network, the most severe congestion causing problems could be alleviated first.

The calibration of the second time slice was done using the same methods as those used for the first time slice such as adjustment of free flow speed, removal of erroneous link flows, generation of new origin/destination matrices, varying the link parameters within the expanded intersections, and obtaining new versions of the QUEENSOD and INTEGRATION programs. In addition, many of the tests and analyses that were performed for the first time slice and discussed in the previous section had to be revisited as the level of traffic in the network rose.

The demand pattern for the third time slice was greater than that of time slices one and two. One iteration of the iterative loop of the demand estimation process was completed in order to obtain a more realistic dynamic origin/destination table for the third time slice. The addition of the traffic for the third time slice caused a significant amount of additional congestion in the network; far greater than the level of congestion existing in the field. In addition to the methods used in the attempts to calibrate the first and second time slices, another method was developed. At this point in the calibration efforts, the possibility was raised that the simulated link flows at many of the signalized intersections in the network were significantly different from the actual volumes. Thus, the signal timing plans provided by the City of Los Angeles were not optimal for these intersections. By optimizing these signals within the INTEGRATION program, it was believed that a number of heavily congested locations could be relieved. Thus, the signal optimization feature of the INTEGRATION model was enabled at all the signalized intersections in the network. However, the use of the signal optimization feature of INTEGRATION revealed a number of problems that needed to be addressed. This portion of the calibration efforts is discussed in Section 7.4.1. The quality of the calibration for time slice three was similar to the results for time slices one and two, and marginally acceptable.

The number of vehicles to be simulated during the fourth time slice was significantly greater than that of time slices one, two and three. Due to the heavy demand pattern of time slice four, the iterative loop of the demand estimation process was utilized. After a few iterations, the difference between the dynamic origin/destination table generated in each iteration had become small enough that the demand estimation process was considered to have converged to a final origin/destination table. The addition of this fourth dynamic origin/destination matrix to the simulation led to significant problems that could not be addressed using any of the methods discussed above. The final simulation run performed with all four time slices was considered calibrated but with some rather serious misgivings. The results of the simulation runs for the first through the fourth time slices is presented in Section 7.4.2.

Attempts were also made to calibrate the fifth and sixth time slices. However, the level of congestion for the fifth and sixth time slices far exceeded the level of congestion observed in the field and further calibration efforts by the research team were unsuccessful. Due to the unresolvable difficulties in calibrating beyond the third time slice by the research team, the model developers agreed to attempt to continue the calibration effort for the morning peak period in February 1995. The model developers effort led to some recommendations which were not followed by the research team because they were deemed inappropriate, as discussed in Section 7.1. As of the time of the preparation of this report, the output from the simulation runs completed by the model developers were not available to the research team. Thus, the results of the model developers efforts to calibrate the morning peak period are not discussed in this report.

7.4.1 Calibration of the Signal Optimization Feature of INTEGRATION

Since it was felt that the simulated link flows at many of the signalized intersections in the network were significantly different from the actual volumes, the signal optimization feature of INTEGRATION was enabled at all signalized intersections in the network. Recall that the INTEGRATION model signal optimization involves the optimization of green splits for a single intersection. Thus, optimization of signals in a corridor (i.e. signal coordination optimization) is not possible with the current version of INTEGRATION. The procedure utilized by the INTEGRATION model for the signal timing plan optimization allocates green time based on an intersection approach's volume/saturation flow ratios, according to the procedures specified in the Canadian Capacity Guide [61].

The cycle length was held fixed at 120 seconds for all optimized signal timing plans by specifying the minimum and maximum cycle lengths allowed during optimization at 120 seconds. These parameters were specified in the INTEGRATION input file number three.

Analysis of traffic performance at expanded intersections in which the INTEGRATION signal optimization feature had been enabled revealed a number of problems. This section discusses the problems found with the signal optimization feature of the INTEGRATION model.

To determine the performance of the signal optimization feature of INTEGRATION the user travel time on all of the twelve links of an expanded intersection were analyzed. This analysis was conducted at various critical expanded intersections in the corridor, one intersection at a time. For each signal phase, the maximum user travel time on the expanded links operating during that phase was determined. The maximum user travel times determined for each signal phase at each intersection were then compared. The performance of the signal optimization feature of INTEGRATION would be considered reasonable at an expanded intersection if the maximum user travel times determined for each signal phase at that intersection were all approximately equal.

Analysis at various critical intersections in the network indicated that this feature of INTEGRATION was not reasonable. Initial analysis determined that the signal timing plans implemented at a signalized intersection which was optimized were different from those determined by the internal optimization of the INTEGRATION model. This problem was only observed at intersections in which the initial cycle length of the field signal timing plan was not equal to the minimum and maximum cycle length set as constraints on the INTEGRATION signal timing plan optimization, which were both 120 seconds. The field signal timing plan should not have any impact on the performance of the INTEGRATION optimization, but modifications to these plans did have an impact on the signal optimization. To rectify this problem the field signal timing plan coded in INTEGRATION input file number three was altered so that all the optimized signals had an initial cycle length of 120 seconds. The phase splits of the field signal timing plans were also modified so that the sum of the phase splits was 120 seconds. Since all the intersections in the network were optimized, these modifications were required at all signalized intersections.

Once the input coding modifications were complete, the signal optimization feature of INTEGRATION was re-tested. These tested revealed that the performance of the signal optimization feature of INTEGRATION had improved, but was still unreasonable. One of the critical expanded intersections analyzed during this re-testing was Venice Boulevard and Robertson Boulevard. A four phase signal timing plan was implemented at this intersection. The analysis revealed that the maximum user travel time for phases one, two, three and four were 57, 45, 213 and 276 seconds respectively. All of the approaches in which these user travel times were simulated were not experiencing any queue spill back. Thus, these user travel times were a result of the implemented optimized signal timing plan. The huge difference in these maximum user travel times indicates that the traffic performance at signalized intersection with the INTEGRATION signal optimization enabled is still unreasonable. As of the conclusion of the calibration stage of the project, a signal optimization feature of INTEGRATION that generates signal timing plans that result in reasonable traffic performance at signalized intersections was not available.

7.4.2 Results of Simulation Runs with Time Slices One Through Four

All of the calibration results discussed in this section were from the INTEGRATION simulation of the first four time slices. Signal optimization was implemented at all of the signalized intersections in the network for this INTEGRATION simulation run.

As with the first time slice, the best measure of a successful calibration effort was the comparison of simulated versus observed link flows. Using the RED12 program to produce summaries of all of the links with excessive delay was also found to be a useful tool in locating areas with heavy congestion. An additional analysis was used for multiple time slices, the speed contour map.

Table 7.1 shows the summary statistics for the final simulation run conducted with the first four time slices of the morning peak period. The most significant figure in this table is the number of links with a delay value greater than 100 seconds. The first and second time slices have very few instances of heavy delay as one would expect. Time slice number three shows a small number of links with heavy delay. While this value may be greater than the actual amount of links with heavy delay in the third time slice, it was considered to be an acceptable number. The fourth time slice exhibits a significant increase in the number of links with unacceptable delay. The increase in this number indicates that the problem(s) causing the congestion are not dissipating. Attempts to calibrate the fifth and sixth time slices were completed, but the results were an unacceptable level of congestion.

Interestingly, the number of links with a volume to saturation (V/S) ratio of greater than 0.75 actually drops from the third to the fourth time slice. The cause of this is expected to be the fact that in a congested network there are numerous bottlenecks and links downstream of these bottlenecks are less congested.

The number of links where the speed is less than 10 km/h is quite high, but was not considered to be a problem. Many of these links are likely to be links within an expanded intersection. Since these links are directly upstream of a traffic signal and are only 100 meters (328 feet) in length, it is not surprising to see a large number of them with average speeds less than 10 km/h. For future research, a better diagnostic measure would be the number of non-expanded links that have an average speed less than 10 km/h.

		Number of Links with.. .		
Time Slice	10 Minute Period	V/S > 0.75	Delay > 100 Seconds	Speed < 10 km/h
First	6:00 - 6:10 a.m.	36	0	696
	6:10 - 6:20 a.m.	37	0	668
	6:20 - 6:30 a.m.	24	0	672
Second	6:30 - 6:40 a.m.	44	0	735
	6:40 - 6:50 a.m.	47	2	731
	6:50 - 7:00 a.m.	40	0	723
Third	7:00 - 7:10 a.m.	65	14	824
	7:10 - 7:20 a.m.	70	19	850
	7:20 - 7:30 a.m.	71	19	869
Fourth	7:30 - 7:40 a.m.	69	112	1,056
	7:40 - 7:50 a.m.	55	200	1,158
	7:50 - 8:00 a.m.	65	302	1,253

Table 7.1: Summary Results from RED12 - First Four Time Slices

When analyzing more than a single time slice, it is useful to develop a speed contour map of the freeway to determine when and where queues originate and dissipate and as a measure of traffic performance on the freeway. Figure 7.12 shows a speed contour map of the eastbound freeway for the first four time slices of the morning peak period. Each row of the figure indicates a 10 minute time slice. The number at the bottom of each column is the link number. The freeway progresses from east to west from link number 1220 to 1260. Each cell shows the speed in miles per hour for each link during each ten minute time slice. Darker shading indicates a lower speed.

Time Slice	1220	1221	1222	1223	1224	1225	1226	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
1 (6:00 - 6:10 a.m.)	57	58	57	58	57	55	57	58	58	58	58	58	57	55	56	48	53	55	51	54	51
1 (6:10 - 6:20 a.m.)	57	58	57	58	58	58	58	58	59	58	58	57	55	56	55	59	58	58	57	55	55
1 (6:20 - 6:30 a.m.)	59	57	58	58	58	55	56	57	57	58	59	58	57	55	55	57	58	58	57	54	53
2 (6:30 - 6:40 a.m.)	57	55	55	58	57	49	56	56	58	53	45	43	52	52	55	56	52	55	50	48	49
2 (6:40 - 6:50 a.m.)	58	58	52	57	57	53	56	57	57	54	56	52	55	52	56	55	52	56	52	52	49
2 (6:50 -7:00 a.m.)	57	56	53	55	57	55	55	57	58	55	55	52	53	51	55	55	53	55	52	53	57
3 (7:00 - 7:10 a.m.)	53	53	47	53	48	46	55	54	45	50	32	44	50	50	51	51	50	25	44	37	41
3 (7:10 - 7:20 a.m.)	53	52	53	56	49	47	51	51	48	50	41	43	48	47	49	52	52	17	24	22	38
3 (7:20 - 7:30 a.m.)	52	54	47	53	49	47	51	53	48	50	47	43	49	52	55	48	29	16	27	24	37
4 (7:30 - 7:40 a.m.)	51	32	42	46	43	43	48	47	47	50	54	51	55	57	52	53	56	58	55	27	37
4 (7:40 - 7:50 a.m.)	50	52	52	49	44	44	52	48	27	14	18	45	49	53	53	56	55	57	52	36	40
4 (7:50 -8:00 a.m.)	37	42	40	44	45	43	53	54	42	43	36	44	57	57	53	43	48	53	50	51	42

Time Slice	1241	1242	1243	1244	1245	1246	1247	1246	1249	1250	1251	1252	1253	1254	1255	1256	1257	1256	1259	1260
1 (6:00 - 6:10 a.m.)	55	55	53	53	56	55	57	57	58	57	57	58	57	56	55	50	53	50	55	58
1 (6:10 - 6:20 a.m.)	56	58	53	53	56	57	57	57	57	57	55	55	55	54	51	50	54	57	56	58
1 (6:20 - 6:30 a.m.)	54	52	51	52	56	55	57	57	57	56	57	58	58	58	57	54	55	57	57	58
2 (6:30 - 6:40 a.m.)	49	52	53	51	55	51	56	56	56	53	52	55	56	56	55	52	53	50	54	57
2 (6:40 - 6:50 a.m.)	51	48	51	53	55	56	57	53	55	44	42	52	52	53	29	35	41	52	52	57
2 (6:50 -7:00 a.m.)	55	55	52	52	52	52	53	52	55	47	48	53	54	52	35	40	41	47	50	57
3 (7:00 -7:10 a.m.)	47	49	52	52	46	48	54	53	55	50	43	52	52	51	40	42	43	47	50	57
3 (7:10 - 7:20 a.m.)	47	40	29	39	42	45	53	51	41	27	42	52	52	52	47	45	45	50	52	56
3 (7:20 - 7:30 a.m.)	40	30	22	38	46	46	45	16	16	25	42	52	52	52	49	42	43	49	55	57
4 (7:30 - 7:40 a.m.)	45	26	22	35	49	52	16	16	14	20	41	52	50	17	32	29	41	51	54	59
4 (7:40 - 7:50 a.m.)	45	50	47	33	45	31	12	20	14	20	42	52	57	56	23	21	27	25	51	55
4 (7:50 - 8:00 a.m.)	44	45	29	16	39	28	14	16	11	11	4	[illegible]	62	62	47	41	42	50	53	57

Figure 7.12: Speed Contour Map for the Eastbound Mainline Freeway

Minor congestion begins on the eastbound freeway during the second time slice and increases steadily throughout the simulation of the first four time slices. There are a number of bottlenecks evident from this figure such as links 1225, 1251 and 1257. The cause of the severe bottleneck at link 1252 that arises during the last ten minutes of the fourth time slice is due to an off-ramp queue. At this location, the average speed of the mainline freeway drops to 3 mph.

Figures 7.13 to 7.20 show the graphs of observed versus simulated flow for the eastbound and westbound portions of the mainline freeway for the last ten minutes of the first four time slices of the morning peak period. It should be noted that the graphs for the first time slice (Figures 7.13 and 7.14) differ from those that resulted from the calibration of the first time slice (Figures 7.8 and 7.9) because the signal optimization feature of INTEGRATION was enabled for all time slices in the calibration of the morning peak period. Comparison of these two sets of first time slice graphs reveals that signal optimization had no significant impacts on the simulated link flows for the eastbound mainline freeway. However, signal optimization resulted in significantly lower predicted link flows for the westbound mainline freeway in the middle portion of the freeway. It is not surprising that the freeway mainline flow was reduced with the implementation of signal optimization as more vehicles will travel on the arterial network with improved signal timing plans.

A further problem encountered in the simulation of multiple time slices was that modifications made to the network to calibrate later time slices would often impact the INTEGRATION predicted traffic performance in earlier time slices. It should be noted that there are significant differences between the observed and simulated flows for the fourth time slice in both directions (Figures 7.19 and 7.20).

In summary, the incremental approach to calibrating the morning peak period resulted in a reasonable calibration of time slices one, two and three. However, after an extensive effort, both at U.C. Berkeley and Queen's University, a reasonable calibration of time slices four, five and six was not attained.

Further improvements in the refined demand estimation process may have resulted in a reasonable calibration for more of the morning peak period time slices. Improvements in the generation of multiple minimum path trees would have resulted in a better set of minimum path trees to input to the QUEENSOD model. Efforts to determine optimal arterial link flow reliability factors would have improved the demand estimation process. These improvements in the demand estimation process would have probably led to a more accurate origin/destination matrix, which would have produced a more reasonable calibration. However, further experiments to determine the optimal number of minimum path trees and optimal generation time of those minimum path trees, and to determine the optimal arterial link flow reliability factors, were not possible due to limited project time and funding.

The calibration of the morning peak period time slices would surely of been much more successful if the exact magnitude of prevailing demands and saturation flows of all turning movements for all approaches at all signalized intersections in the network were available. The availability of this data would have improved the modeling of traffic performance at signalized intersections. Since unrealistic delay at many signalized intersections in the corridor was one of the main reasons why the time slices with heavy demands could not be calibrated, improved modeling of traffic at intersections would have resulted in a much more successful calibration effort of the morning peak period. However, for a network the size of the Santa Monica Freeway corridor, determining the exact magnitude of prevailing demands and saturation flows of all turning movements for all approaches at all signalized intersections is an extremely hard and time consuming task. Thus, a more successful calibration would be possible if project time and funds allowed for a much more detailed data collection and coding effort than was undertaken for this research effort. It should be noted that the data collection stage of the project did result in one of the most comprehensive and complete corridor data bases ever assembled. Thus, the need for more data than was collected by the research team is a major requirement of the INTEGRATION model (and most other similar simulation models).

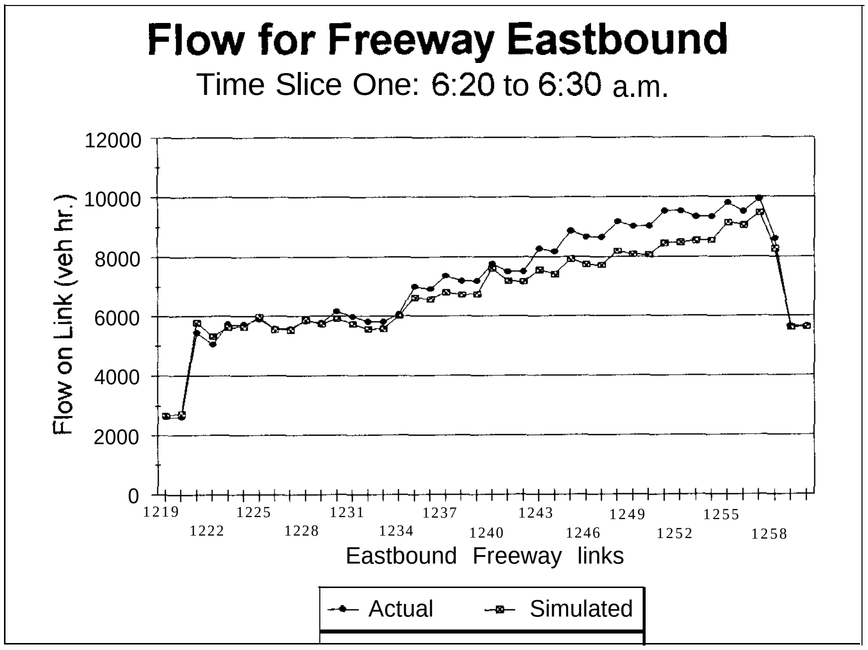

Figure 7.13: Eastbound Freeway Flow 6:20 to 6:30 A.M.

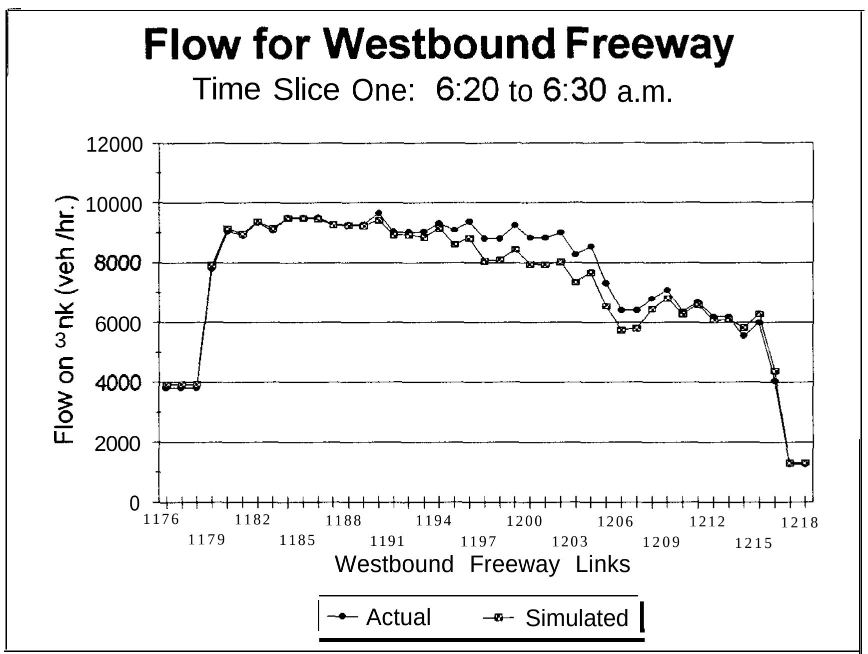

Figure 7.14: Westbound Freeway Flow 6:20 to 6:30 A.M.

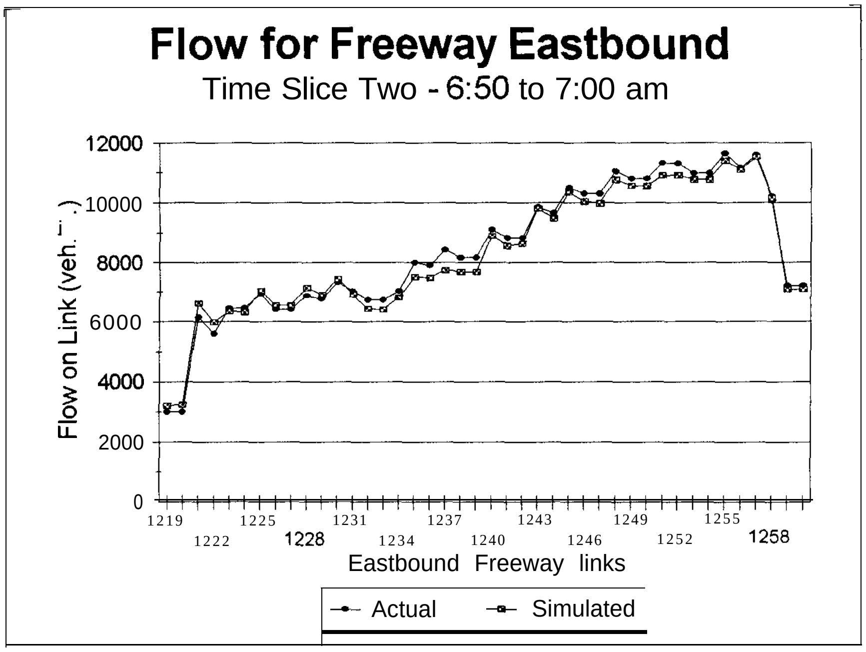

Figure 7.15: Eastbound Freeway Flow 6:50 to 7:00 A.M.

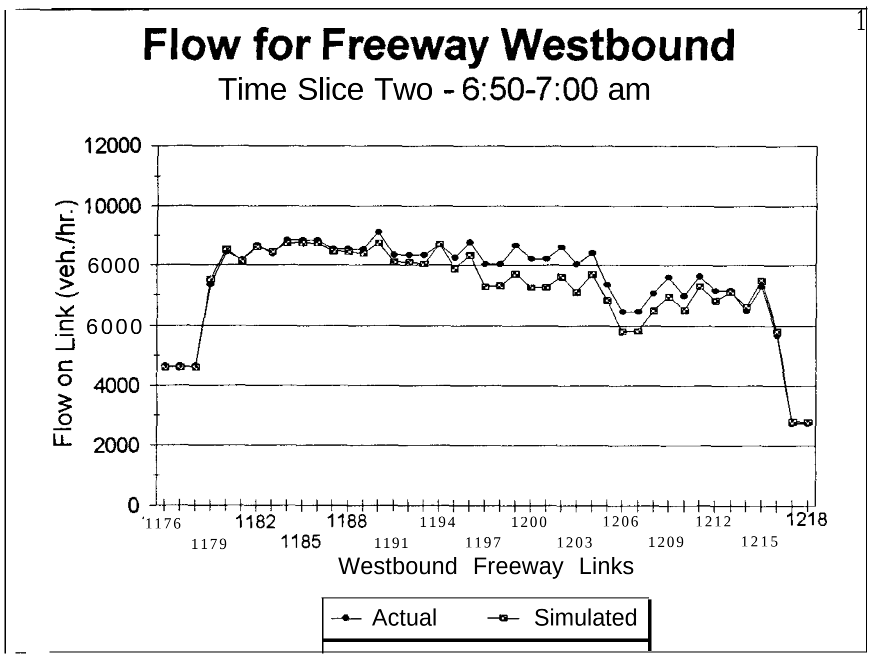

Figure 7.16: Westbound Freeway Flow 6:50 to 7:00 A.M.

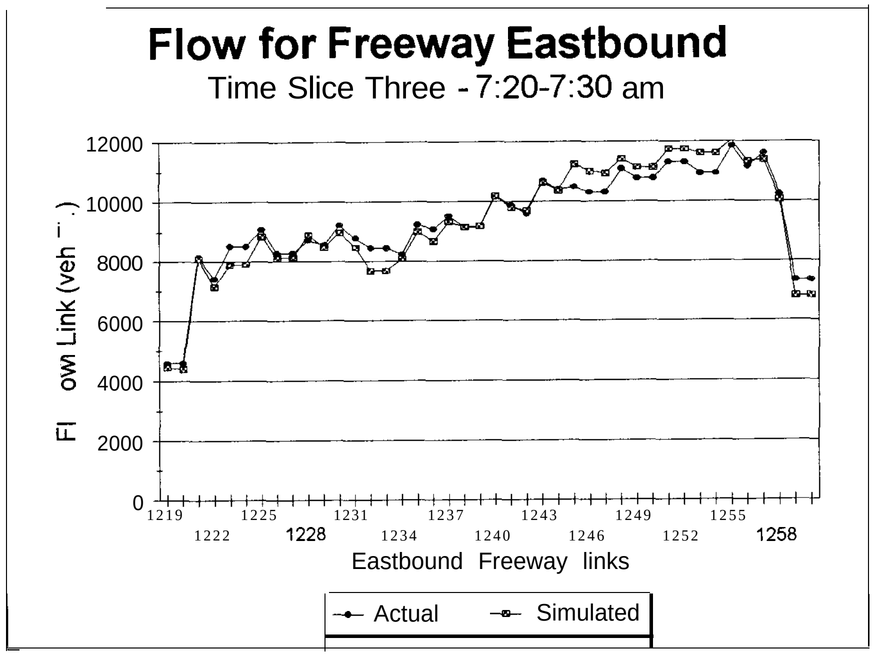

Figure 7.17: Eastbound Freeway Flow 7:20 to 7:30 A.M.

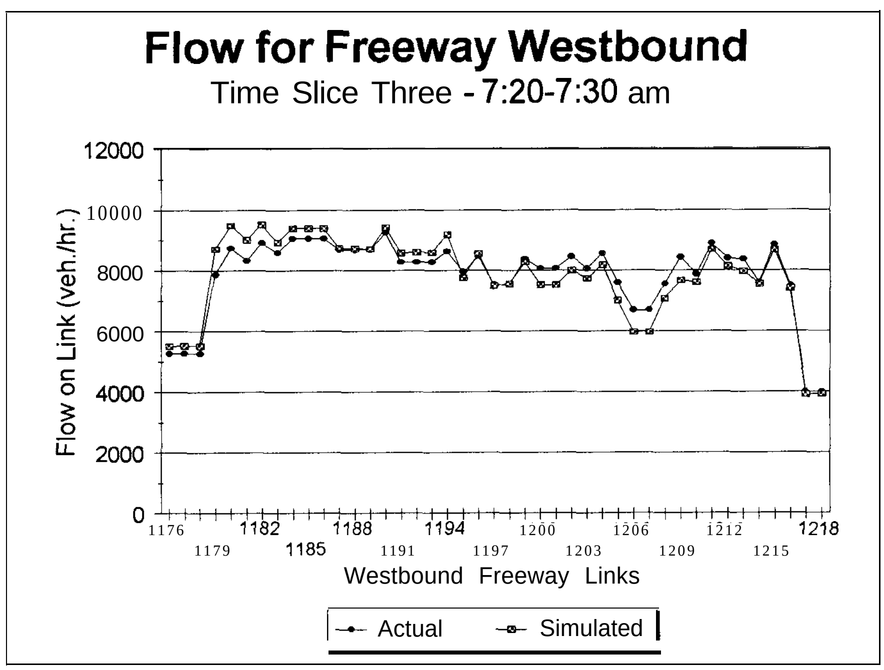

Figure 7.18: Westbound Freeway Flow 7:20 to 7:30 A.M.

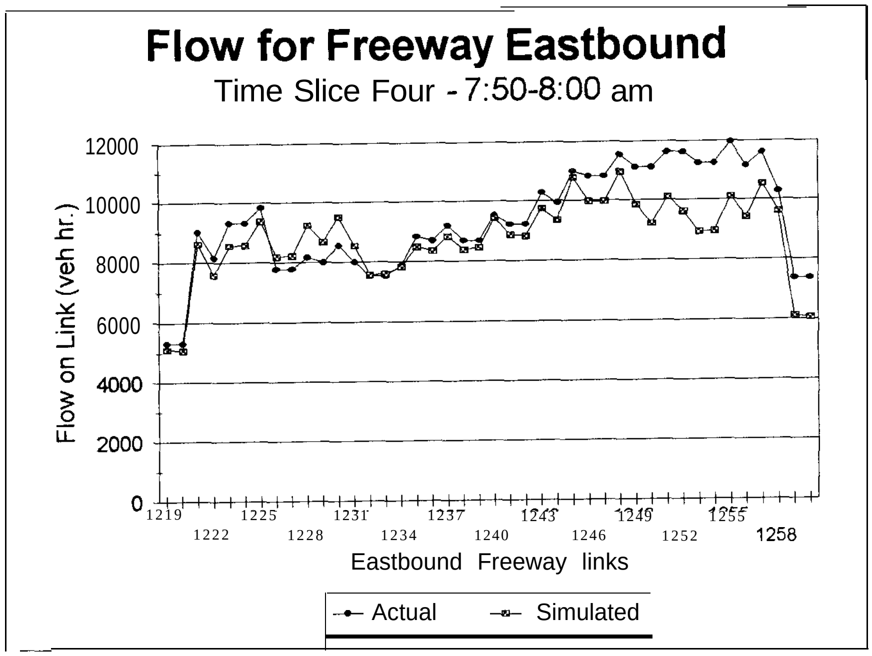

Figure 7.19: Eastbound Freeway Flow 7:50 to 8:00 A.M.

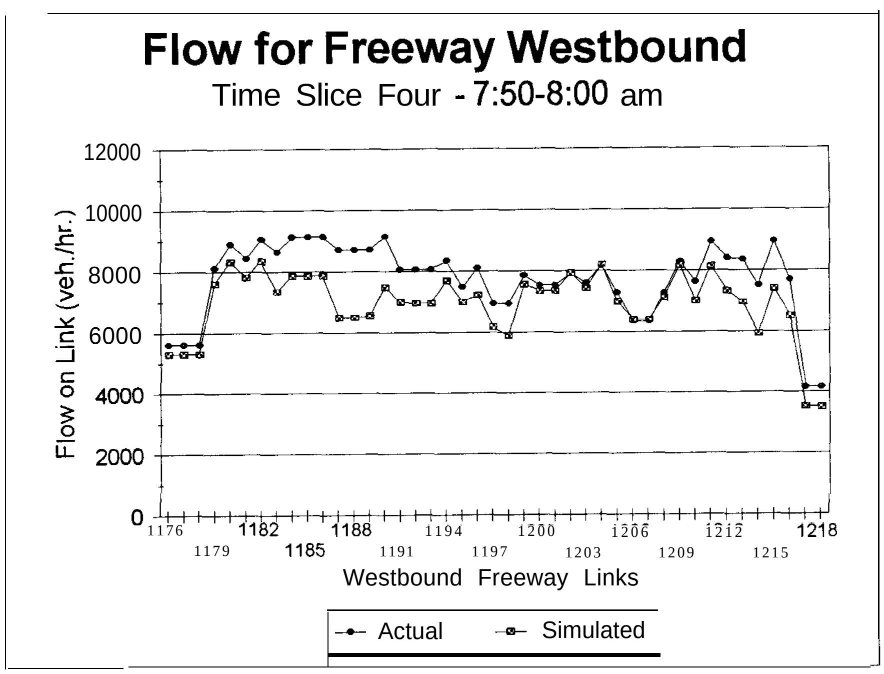

Figure 7.20: Westbound Freeway Flow 7:50 to 8:00 A.M.

7.5 CALIBRATION OF THE MIDDAY PERIOD

While the model developers addressed the issue of calibrating the morning peak period, the research team began to address the calibration issue for the midday period in February of 1995 and continued its efforts until May of 1995. An easier calibration effort was anticipated for the midday period since there was little congestion observed in the field in the freeway corridor. Unfortunately, the calibration problems previously encountered in the morning peak period continued to plague the research team in attempting to calibrate the model for the midday period.

The methodology used to create the origin/destination matrices was identical to that done for the morning peak period. Due to time and fund restrictions, the iterative loop of the demand estimation process was not utilized in the generation of the dynamic origin/destination matrices for any of the midday time slices.

The results of the calibration efforts were also similar to the morning peak period. The first three time slices of this period (time slices nine through eleven) were calibrated to an acceptable level with some misgivings. The main misgiving was the large number of links with a delay greater than 100 second; when in reality very little congestion was observed in the field for these three time slices. At time slice number twelve, the level of congestion in the network approaches an unacceptable level. By time slice number thirteen, the level of congestion is completely unacceptable. This unacceptable level of congestion continues for the remaining time slices simulated for the midday period.

Again, further improvements in the demand estimation process would have probably resulted in a more successful calibration of the midday period. Also a more detailed data collection effort would have improved the modeling of traffic performance at signalized intersections. Poor traffic performance at signalized intersection resulted in unrealistic delays on many links. Eliminating or minimizing the number of links with unrealistic delays would have certainly improved the success of the midday calibration effort. However, the data needed to improve the modeling of signalized intersections would have required a much more extensive data collection effort than was conducted for this research effort.

Table 7.2 shows the link flow summary statistics from the RED12 program for time slices number nine through sixteen. Clearly, the trend throughout these time slices is an increasing amount of congestion that does not decrease with time but actually increases. Again, the only parameter listed in the table that does not increase with time is the number of links with a volume to saturation (V/S) ratio greater than 0.75. The cause of this is expected to be the fact that as the congestion increases the number of links with significant flow, and hence the potential for a high V/S ratio, decreases.

The number of links with a speed lower than 10 km/h and the number of links with a delay greater than 100 seconds increases steadily throughout the simulation of the midday period. Incredibly, by the end of the simulation over half of the links in the network have speeds less than 10 km/h.

Figures 7.21 through 7.30 show the observed versus simulated link flows for the midday period simulation run. Note that the difference between the two appears marginally acceptable for time slices nine, ten and eleven, but starts to fail by time slice twelve, and has dompletely failed by the thirteenth time slice.

Time Slice	10 Minute Period	Number of Links with.. V/S > 0.75	Delay > 100 Seconds	Speed < 10 km/h
Nine	10:00 - 10:10 a.m.	43	71	948
	10:10 - 10:20 a.m.	47	59	921
	10:20 - 10:30 a.m.	44	64	948
Ten	10:30 - 10:40 a.m.	57	89	954
	10:40 - 10:50 a.m.	65	87	976
	10:50 - 11:00 a.m.	44	70	944
Eleven	11:00 - 11:10 a.m.	62	94	1,005
	11:10 - 11:20 a.m.	42	144	1,017
	11:20 - 11:30 a.m.	43	157	1,057
Twelve	11:30 - 11:40 a.m.	70	209	1,100
	11:40 - 11:50 a.m.	46	259	1,113
	11:50 a.m.- 12:00 p.m.	49	256	1,150
Thirteen	12:00 - 12:10 p.m.	59	355	1,213
	12:10 - 12:20 p.m.	46	445	1,238
	12:20 - 12:30 p.m.	14	501	1,292
Fourteen	12:30 - 12:40 p.m.	12	578	1,389
	12:40 - 12:50 p.m.	12	633	1,418
	12:50 - 1:00 p.m.	5	691	1,450
Fifteen	1:00 - 1:10 p.m.	5	779	1,498
	1:10 - 1:20 p.m.	4	852	1,536
	1:20 - 1:30 p.m.	2	962	1,596
Sixteen	1:30 - 1:40 p.m.	6	1,081	1,683
	1:40 - 1:50 p.m.	1	1,145	1,685
	1:50 - 2:00 p.m.		1,172	1,714

Table 7.2: Summary Results from RED12 - Time Slices Nine through Sixteen

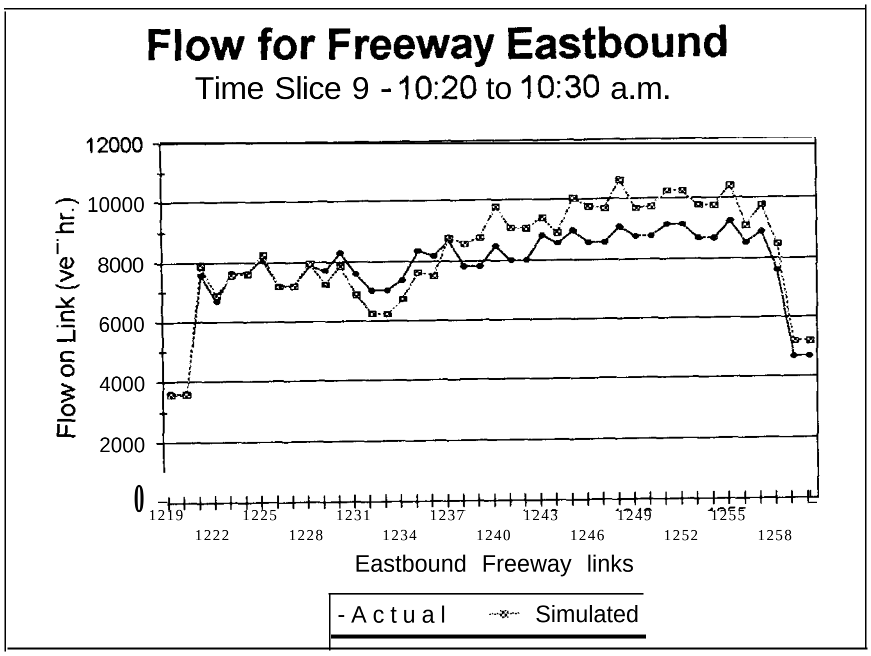

Figure 7.21: Eastbound Freeway Flow 10:20 to 10:30 A.M.

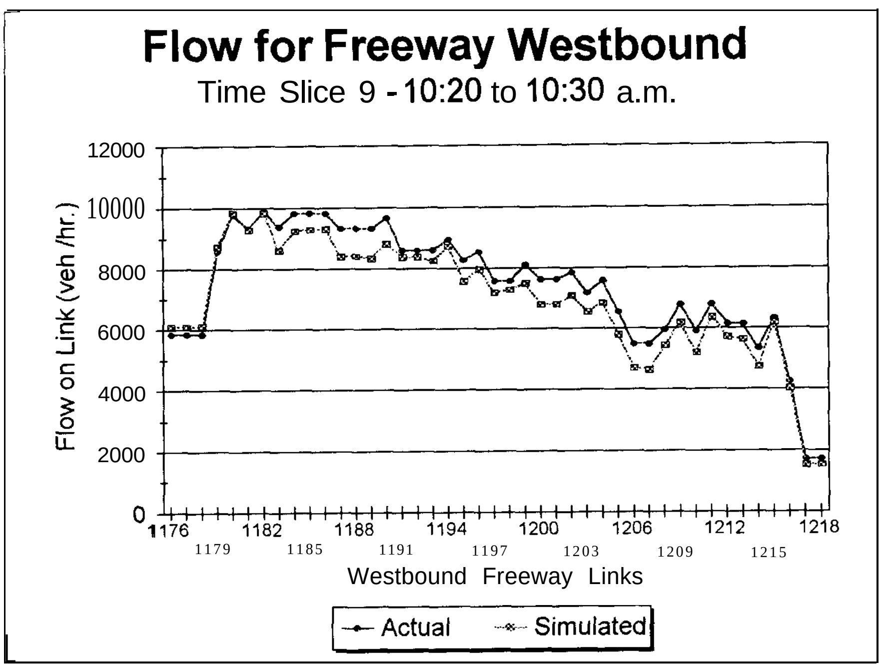

Figure 7.22: Westbound Freeway Flow 10:20 to 10:30 A.M.

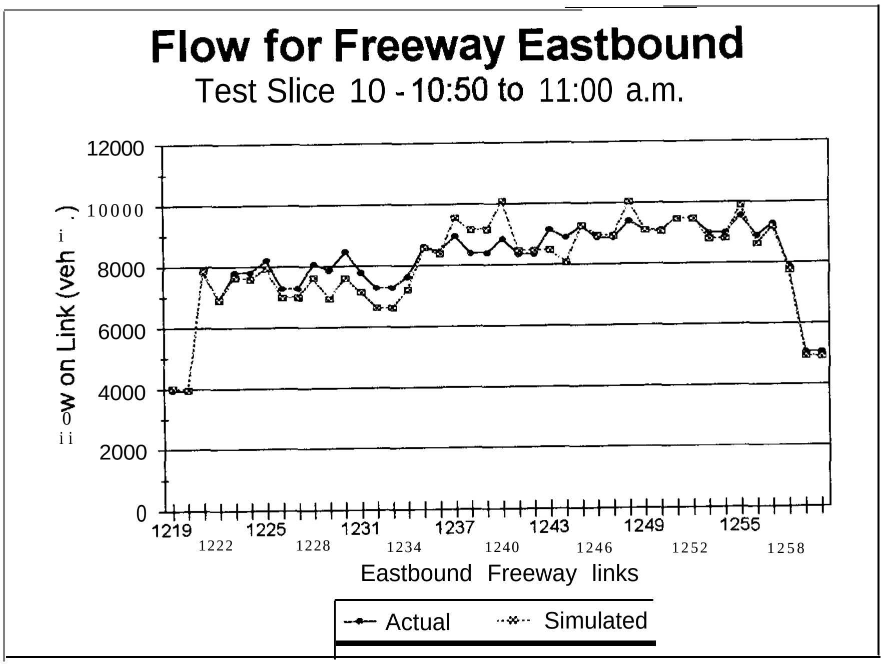

Figure 7.23: Eastbound Freeway Flow 10:50 to 11:00 A.M.

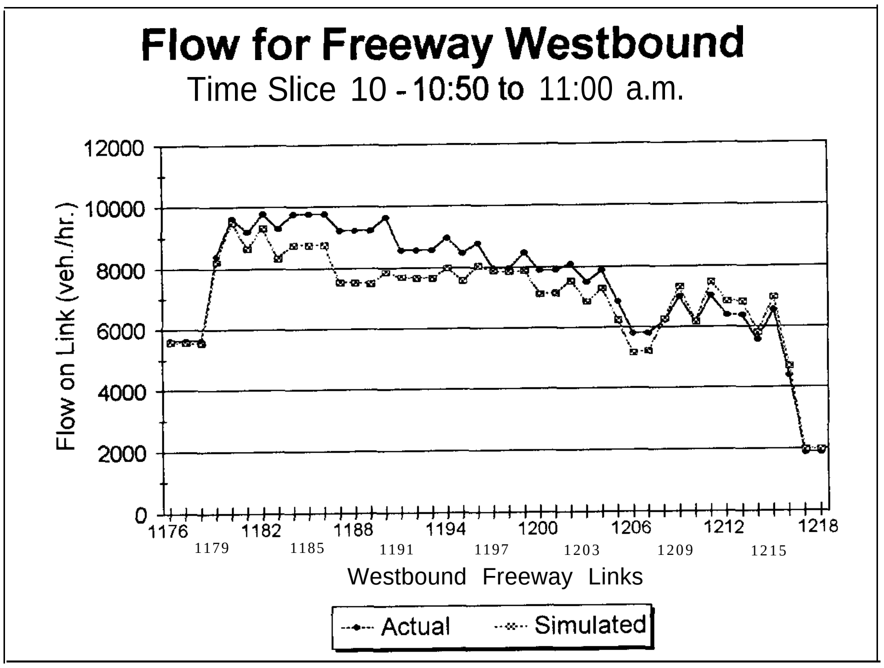

Figure 7.24: Westbound Freeway Flow 10:50 to 11:00 A.M.

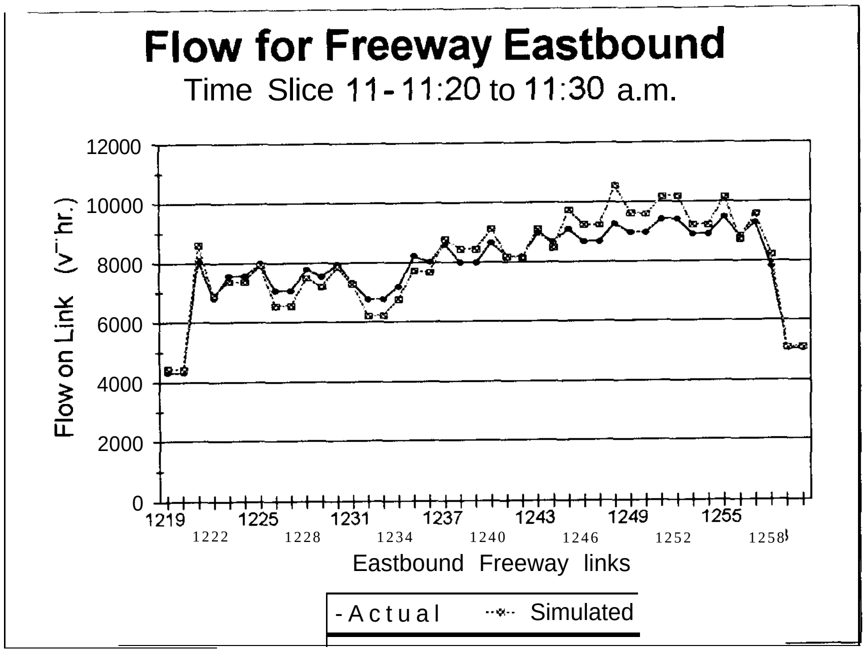

Figure 7.25: Eastbound Freeway Flow 11:20 to 11:30 A.M.

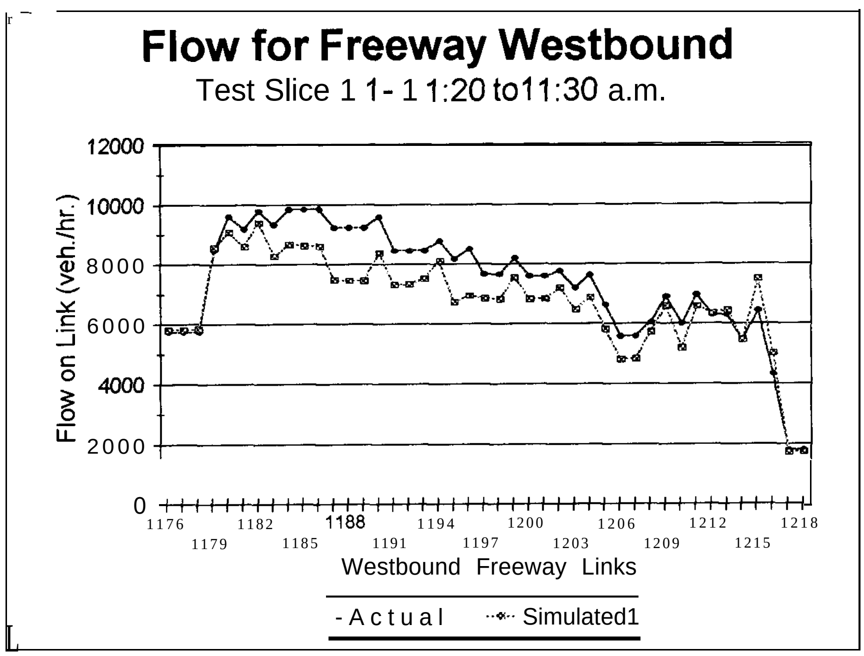

Figure 7.26: Westbound Freeway Flow 11:20 to 11:30 A.M.

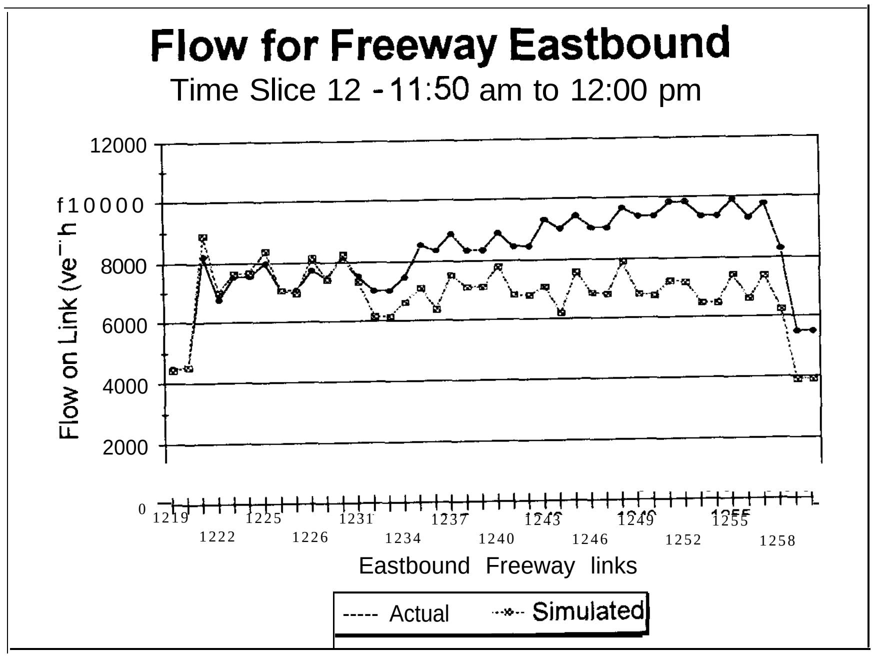

Figure 7.27: Eastbound Freeway Flow 11:50 A.M. to 12:00 P.M.

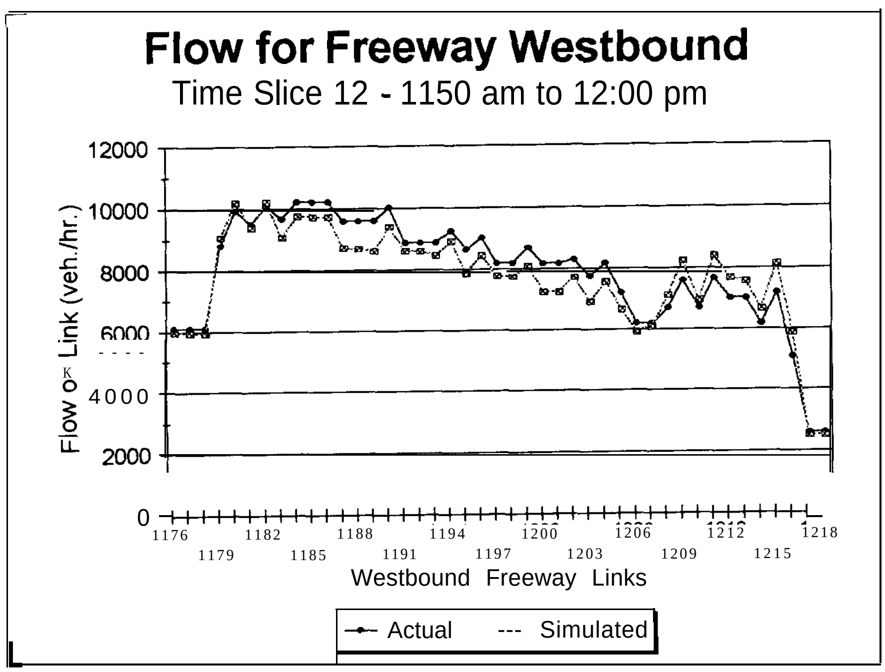

Figure 7.28: Westbound Freeway Flow 11:50 A.M. to 12:00 P.M.

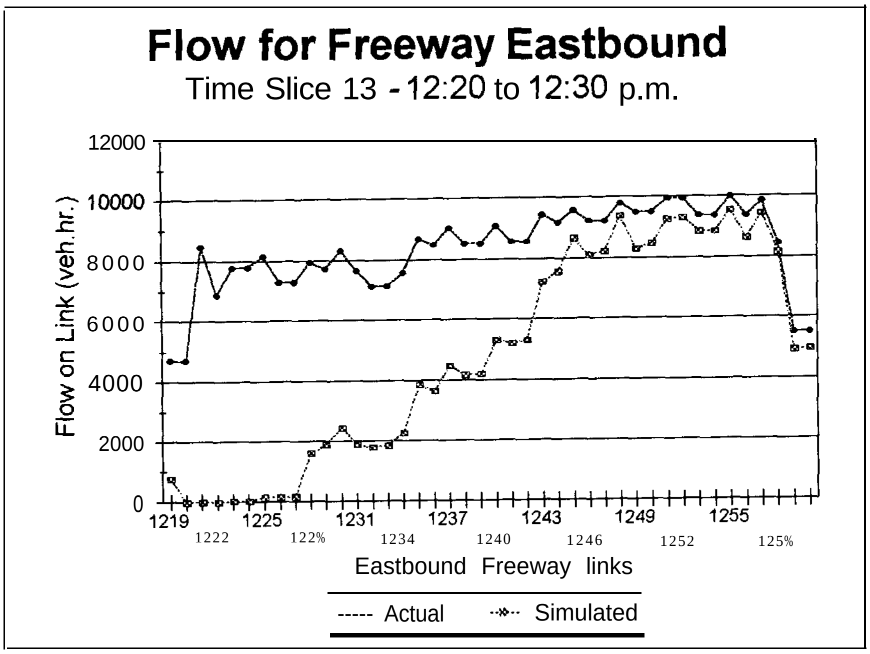

Figure 7.29: Eastbound Freeway Flow 12:20 to 12:30 P.M.

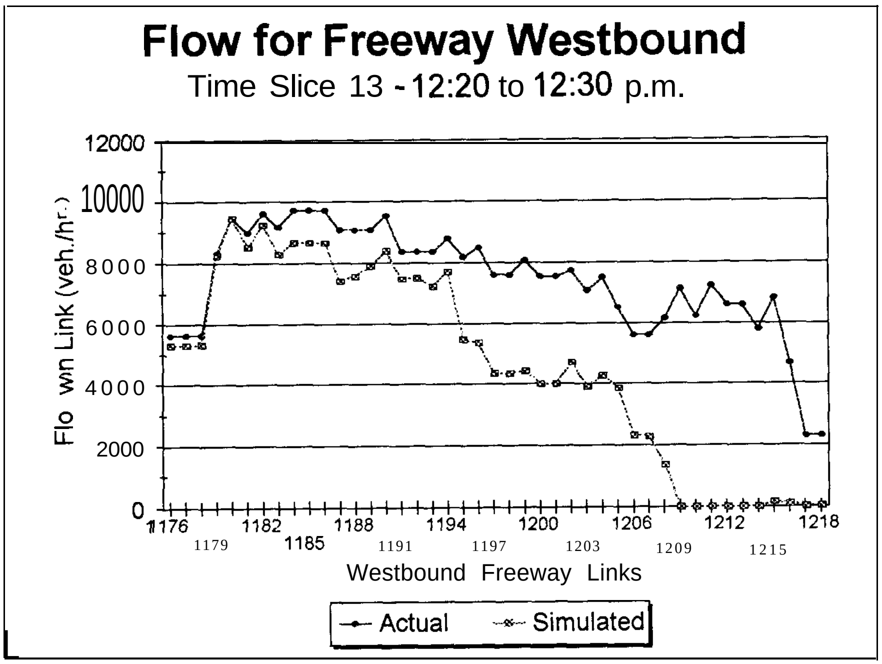

Figure 7.30: Westbound Freeway Flow 12:20 to 12:30 P.M.

7.6 CONCLUSION

The model calibration effort was extremely time-consuming and the simulation for only a few early morning uncongested time slices were reasonably successful. Calibration for the entire morning peak period and the midday period were unsuccessful after many valiant attempts. The research team devoted approximately eleven months to model calibration with almost weekly consultation with the model developers, and with their devoting several months to attempting to calibrate the morning peak period.

In fairness to complex simulation models in general and particularly to the INTEGRATION model, calibration of complex models is a difficult undertaking requiring considerable effort. This was the first concentrated effort given to calibrating the INTEGRATION model and as far as it is known, the most comprehensive in terms of size and complexity of the network, and in terms of size and complexity of the model input. The network included 3 12 signalized intersections, 1747 nodes, and 3286 links. Input data was prepared for twenty-eight thirty-minute time slices with over three million vehicles to be simulated. Severe congestion occurred on portions of the freeway and many signalized intersections included complex geometric designs and traffic control plans.

On a positive note, solutions to many of the problems encountered in model calibration were found. A systematic procedure for the calibration process was formulated and refined. Techniques for assessing reasonableness of results were developed and implemented. With this report, detailed documentation is available of problems encountered, problem solutions tested, and success/failure results. Future model calibration efforts should be less time consuming and more successful because of the research team and the model developers efforts.

Chapter 8: INVESTIGATIONS

This chapter describes the investigations that were conducted with the Santa Monica Freeway corridor network. This stage of the research was conducted during May and June of 1995. The investigations were to be conducted within each of the three time periods used for the calibration effort (morning peak period, midday period and evening peak period). However, due to time and fund restrictions, coupled with limitations of the INTEGRATION model, the investigation stage of the project was limited to the morning peak period. The demand pattern for the morning peak period was modified to reduce the level of congestion in the network due to problems encountered in the calibration of time slices with heavy traffic demands. The investigations primarily focused on the impact of in-vehicle information systems (MS) with different levels of information quality under non-incident and various incident conditions.

Section 8.1 discusses the development of the design of experiment. This section initially lists all the investigations that would be of interest to simulate with the Santa Monica Freeway corridor network. Due to time and fund restrictions, and limitations of the INTEGRATION model, this list of desirable investigations was severely reduced. This section presents the reasons why the desirable list of investigations was reduced. The investigations conducted for this research effort were reduced to a limited assessment of the impact of ATIS control strategies. The ATIS investigations studied the impact of MS with different levels of information quality under non-incident and various incident situations in a network loaded entirely with guided vehicles.

Section 8.2 presents the development of the base run for the modified morning peak period. The modifications required to the demand pattern for the morning peak period are discussed in this section. An analysis of the traffic performance in this base run is also discussed in this section.

Section 8.3 presents the results from the investigations conducted with the first incident location. Section 8.4 presents the results from the investigations conducted with the second incident location. The traffic performance for each investigation was compared to that of the base run. The traffic performance for each investigation was assessed in terms of trip distances, trip travel times and average speeds.

Finally, Section 8.5 summarizes the ability of the INTEGRATION model to simulate a variety of ATIS and ATMS strategies. The results of the investigations conducted by this research effort are also presented in this section.

8.1 DESIGN OF EXPERIMENT

The initial goal of the project was to determine the potential benefits of ATIS and ATMS control strategies in the Santa Monica Freeway corridor. Recall that ATIS technologies are designed to provide the traveler with navigational information and/or routing advice based on real-time traffic data using audio or visual media contained in the vehicle or on the highway. ATMS technologies include urban traffic control systems, incident detection systems, highway and corridor control systems, High Occupancy Vehicle (HOV) priority treatment and ramp metering systems.

The ATIS control strategies which were of interest to simulate with the Santa Monica Freeway corridor were:

1. In-Vehicle Information Systems (MS);
2. Changeable Message Signs (CMS);
3. Highway Advisory Radio (HAR).

The ATMS control strategies which were of interest to simulate with the Santa Monica Freeway corridor were:

1. Optimized ramp-metering plans;
2. Optimized signal coordination along an arterial route;
3. High-Occupancy Vehicle (HOV) lanes (both take away and add a lane scenarios).

These control strategies were to be studied in isolation and in various combinations. The impact of these control strategies on traffic performance with changes in supply and/or demand were also to be studied. The most obvious example of a change in supply is an incident (which reduces the capacity of the roadway for a period of time). For demand, a good example are the impacts of future growth (which will increase traffic demand).

The following subsections discuss the ability of the INTEGRATION model to simulate the above control strategies. A discussion of the control strategies which can be simulated by the INTEGRATION model, but require testing and validation, is also contained in the following subsections. The control strategies that can be simulated with the INTEGRATION model, which have been tested and validated, were either classified as 'low priority' or 'high priority' investigations. During the investigation stage of the project, the low priority investigations were not studied and the high priority investigations were studied. The following subsections also note which control strategies were classified as low priority and which were classified as high priority.

8.1.1 In-Vehicle Information Systems (IVIS)

The investigations of IVIS were initially planned to study the impact of varying the level of market penetration of IVIS. These investigations would have varied the percentage of guided vehicles (vehicle type 2) in the network under various incident situations. Investigations to study the impact of varying the quality of information provided to motorists at the various levels of IVIS market penetration were also planned. Tests to determine the impact of varying the number of surveillance links in the network at various levels of IVIS market penetration were also planned.

Since attempts to establish an INTEGRATION run that simulated a base run with 100 percent type 1 vehicles (background or unguided vehicles) that reasonably matched the simulation run with 100 percent type 2 vehicles (guided vehicles) were unsuccessful, the IVIS investigations were severely limited. Recall that all of the vehicles simulated in the calibration effort were guided vehicles of type 2. Using 100 percent type 2 vehicles for the calibration effort assumed that all vehicles in the network have knowledge of recurrent congestion and that the network has reached user travel time equilibrium. Under this assumption there will be no benefit to using MS under non-incident scenarios. It should be noted that unguided vehicles of type 1 follow either user supplied minimum path trees or minimum path trees which are generated based on user supplied link travel times. Since the INTEGRATION model which output minimum path trees from each origin to each destination could only simulate one time slice, the routing trees that the guided vehicles in a multiple time slice calibration run were following could not be generated. Thus, the link travel times throughout the multiple time slice calibration run with 100 percent type 2 vehicles were outputted and these link travel times would be used to generate minimum path trees for the INTEGRATION run with 100 percent type 1 vehicles.

Two simulation runs were attempted using 100 percent type 1 vehicles that used the link travel times that were outputted every ten minutes from the calibration run with 100 percent type 2 vehicles. These link travel times, which were input to the INTEGRATION model as input file number seven, were considered historic information for the unguided vehicles. The unguided vehicles of type 1 used these link travel times to generate minimum path routing trees. The first attempt at using vehicle type 1 was coded so that the unguided vehicles updated their minimum path trees every two minutes. The coding was also such that the user supplied link travel times had a normally distributed error of ten percent introduced into them prior to tree building. Thus, although the historic link travel time was changed every ten minutes, the distortion of this link travel time resulted in variations in the routing trees that were generated every two minutes. The second attempt at using vehicle type 1, which was based on the model developer's recommendations, was coded so that the ASSIGN feature of the INTEGRATION model was enabled (i.e. the simulation time was coded as a negative number). This second attempt was coded so that the unguided vehicles updated their minimum path trees every thirty minutes with a normally distributed error of ten percent introduced into the historic link travel times prior to tree building. Both of these simulation runs resulted in extreme levels of congestion in the network; far above the level of congestion in the calibration run using 100 percent type 2 vehicles.

Since a run using 100 percent type 1 vehicles that follow minimum path trees which are based on the link travel times output from the calibration run with 100 percent type 2 vehicles could not be established, the simulation run with 100 percent type 2 vehicles was used as the base run. This restriction limited the scope of the MS investigations to studies with a network loaded entirely with guided vehicles. Thus, a market penetration study of MS was not possible. Another limitation is that the summary statistics from the IVIS investigations that were conducted are only relative since summary statistics from a network loaded entirely with unguided vehicles were unknown.

MS investigations to study the impact of varying the quality of information provided to motorists were possible to conduct with the INTEGRATION model. The method used to vary the quality of information provided to motorists in the INTEGRATION model is discussed in Section 3.6.4. Simulations that varied the quality of information provided to motorists were conducted by Gardes and May [22] in the development of the base run for the network illustrated in Figure 2.4. Thus, the ability of the INTEGRATION model to reasonably model variations in the quality of information provided to motorists was confirmed by this earlier research effort. Based on the INTEGRATION developer's recommendation, investigations to vary the level of information quality provided to motorists were classified as a high priority investigation.

IVIS investigations to determine the impact of varying the number of surveillance links in the network were possible to conduct with the INTEGRATION model. Links with the surveillance flag (refer to Figure 5.10) set to "0" are considered liis which are not monitored for real-time travel time data. Studies to determine the impact of varying the number of surveillance links in the network would vary the number and location of links with the surveillance flag set to "0". Tests to validate this feature of the INTEGRATION model were not conducted by the research team and could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Due to time and fund restrictions, it was not possible for the research team to validate this feature of the INTEGRATION model. Since this feature was not validated and project time was extremely limited, investigations varying the number of surveillance links in the network were not conducted.

8.1.2 Changeable Message Signs (CMS)

Simulations to determine the impact of CMS were possible to conduct with the INTEGRATION model. Section 3.6.5 discusses how CMS can be modeled with the INTEGRATION model. Tests to validate the INTEGRATION model's ability to simulate CMS were not conducted by the research team and could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Since the modeling of CMS was not validated and project time was extremely limited, investigations to study the impact of CMS were not conducted.

8.1.3 Highway Advisory Radio (HAR)

Investigations to study the impact of HAR were also possible to conduct with the INTEGRATION model. The nodes in the area covered by an HAR station could be coded as CMS nodes. Again, tests to validate the ability of the INTEGRATION model to simulate HAR were not conducted by the research team and could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Thus, investigations to study the impact of HAR were not conducted.

8.1.4 Optimized Ramp-Metering Plans

The version of the INTEGRATION model used for this research project did not allow for ramp metering rates to be optimized based on freeway flows. Thus, ATMS investigations to study the impact of ramp meter optimization were not conducted.

8.1.5 Optimized Signal Coordination along an Arterial Route

Recall from Section 3.6.2 that the INTEGRATION model is only capable of performing limited optimization of individual signals. Thus, optimized signal coordination along an arterial route could not be simulated with the version of the INTEGRATION model used for this research project. Since it was felt that the simulated link flows at many of the signalized intersections in the network were significantly different from the actual volumes, the optimization of individual signal timing plans was enabled at all signalized intersections in the network for all time slices during the calibration of the morning peak period and midday period. Since the optimization of individual signals was enabled during the calibration stage of the project and in all investigations, studies on the impact of individual signal timing optimization, or the lack thereof, were not an option for the investigation stage of the project.

8.1.6 High-Occupancy Vehicle (HOV) Lanes

The INTEGRATION model is very robust in its capabilities to simulate HOV lanes. The model can simulate both take-away and add-a-lane strategies. Refer to Section 3.6.6 for a detailed discussion on HOV lane modeling with the INTEGRATION model. Extensive testing of the INTEGRATION model's ability to simulate HOV lanes revealed that the INTEGRATION model was reasonably simulating HOV lanes, refer to Section 6.4. Due to time and fund restrictions, investigations with HOV lanes were considered lower priority and were not conducted as part of this research project.

8.1.7 Supply Coding Modifications

The impact of modifications to the supply coding can also be studied with the INTEGRATION model. The most interesting supply coding modification to investigate is link capacity reductions due to incidents. Refer to Section 3.6.7 for a detailed discussion on the modeling of incidents with the INTEGRATION model. The ability of the INTEGRATION model to simulate various incident situations was tested and verified in earlier research conducted by Gardes and May [22]. Investigations of the impact of modifications to the supply coding were classified as high priority.

For the investigations conducted in this chapter both non-incident and incident situations were considered. Two different incident locations and two different incident magnitudes (a minor incident and a major incident) were studied in the investigations. Section 8.1.9 discusses the two incident locations and the two different incident magnitudes. Section 8.3 contains the results from the investigations with incident location one. Section 8.4 contains the results from the investigations with incident location two.

8.1.8 Demand Coding Modifications

The INTEGRATION model has the ability to simulate changes in the demand data. The INTEGRATION origin/destination demand data file (INTEGRATION input file number four) contains a scale factor which permits the departure rates for all origin/destinations to be scaled up or down. Even though the INTEGRATION model has the ability to simulate various growth factors, modifications in the demand data were not possible with the Santa Monica Freeway corridor. Recall fi-om the calibration effort that time slices with heavy demand patterns could not be calibrated. Thus, investigations which increase the level of demand in the network were not possible. Investigations which decrease the level of demand in the network were possible, but were considered to be of less importance. Thus, the investigations did not analyze the impact of demand coding modifications.

8.1.9 Design of Experiment for Investigations

The investigations conducted studied the impact of IVIS with different levels of information quality under both non-incident and incident situations in a network loaded entirely with guided vehicles. Thus, the investigations conducted were primarily a limited assessment of the impact of ATIS control strategies. Since a base run with 100 percent type 1 (unguided) vehicles was not attainable, all the investigations were conducted with a network loaded entirely with type 2 (guided) vehicles. Thus, all simulations conducted for this stage of the project were only with vehicles containing IVIS.

Two levels of information quality were considered for these investigations. The first level was vehicles receiving perfect link travel time information. The second level was vehicles receiving link travel time information that had a normally distributed error of 20 percent introduced into the link travel times prior to minimum path tree building. For all the investigations the minimum path tree update frequency was set at 60 seconds.

The investigations considered two different incident locations and two different incident magnitudes (a minor incident and a major incident). The first incident location was at link 1243, which is on the eastbound mainline freeway between Crenshaw Boulevard and Arlington Avenue. The second incident location was at link 125 1, which is on the eastbound mainline freeway between Normandie Avenue and Vermont Avenue. Analysis of the base run, which will be discussed in the next section, reveals that incident location two is at one of the major bottleneck locations in the eastbound mainline freeway. Whereas, there are no major bottlenecks at incident location one. Thus, an incident with similar characteristics (same start time, end time and duration) will cause higher levels of congestion at incident location two.

The minor incident had a duration of one half-hour (one time slice), with a one lane reduction in roadway capacity. The major incident had a duration of one hour (two time slices), with a two lane reduction in roadway capacity for the first half-hour followed by a half lane reduction for the next half-hour. Both the minor and major incidents began at the beginning of the fourth time slice of the modified morning peak period used for the investigations. The demand pattern used for the investigations will be discussed in the next section.

Since a base run with unguided vehicles was not attainable, all of the investigation results were compared to a base run with 100 percent guided vehicles that received perfect information under non-incident conditions.

8.2 DEVELOPMENT OF BASE RUN

This section presents the development of the base run for a modified morning peak period. The demand pattern for the morning peak period was modified to reduce the level of congestion in the network due to problems encountered in the calibration of time slices with heavy traffic demands. Recall from the calibration effort that an acceptable calibration of the first three time slices of the morning peak period was attained. However, a reasonable calibration for the fourth time slice, which had a significantly higher demand level than time slices one, two and three, was not attained.

In order to conduct investigations with the morning peak period the demand pattern was modified to create an eight time slice run which had a demand pattern of 1 - l-2-3-2- 1- 1 - 1. The one's in this pattern represent the origin/destination matrix for the first time slice of the morning peak period. Similarly the two's and three in this pattern represent the origin/destination matrix for time slice two and three of the morning peak period respectively. The first time slice of this simulation, the warm-up time slice, had a time slice one origin/destination table. The first four time slices of this simulation had the same demand pattern as that of the morning peak period. However, the fourth to the eighth time slices followed different demand patterns. The actual origin/destination matrices for these latter time slices were not used for the simulations as these origin/destination matrices produced heavy demand patterns which could not be calibrated. The demand pattern for the investigations had a time slice one origin/destination matrix for the last three time slices to allow the queues which formed during the run to clear. It is important that the queues clear by the end of the simulation run, both for this base run and for the investigation runs, in order to properly measure the traffic performance in the network.

The 1-1-2-3-2-1-1-1 demand pattern used for the simulations generated 250,878 vehicles over a period of four hours (the base run simulated eight thirty minute time slices). The base run took approximately 36 hours to simulate with a 486SX 50 MhZ personal computer with 64 megabytes of random access memory (RAM) available.

The minor incident, which was a one lane reduction in roadway capacity for a half-hour, began at the beginning of time slice four and finished at the end of time slice four. Thus, the minor incident was only modeled in the fourth time slice, which simulated the origin/destination matrix for the third time slice of the morning peak period. Recall that the origin/destination matrix for the third time slice of the morning peak period generated significantly more vehicles than that of time slice's one and two.

The major incident, which was a two lane reduction in roadway capacity for the first half-hour and then a half lane reduction in roadway capacity for the next half-hour, began at the beginning of time slice four and finished at the end of time slice five. Thus, the major incident was modeled during time slices four and five.

Speed contour maps for both the eastbound and westbound mainline freeway were analyzed for the base run. The eastbound mainline freeway's speed contour map for the base run is illustrated in Figure 8.1. The horizontal axis is the subsection number, scaled by length of section, and the vertical axis is the time slice number. The time slices are numbered from two to eight because the first time slice was ignored as it was the warm-up time slice. So the time slices number two to eight in Figure 8.1 were simulating a demand pattern of 1-2-3-2-l -1 -1. The data in the chart indicate the speed of traffic on each subsection for each time slice, in units of 10 km/h. For example, a value of 9 indicates an average link speed between 90 km/h and 99 km/h. Where speeds of 50 kmih or lower are found, the value is shown in bold and strikethrough. Where speeds between 51 km/h and 69 km/h are found, the value is shown in bold.

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SPEED CONTOUR MAP
Eastbound Mainline Freeway (ver 1.5f)

 Time
Slice ....................................................................................................
    2.9999999999999999999999999999999999999999999998888899999999999999999999999999999988888889999999999999.
    3.9999999999999999999999988888888888888899999998888888888888899999999999999998888888888889999999999999.
    4.8888888888888888888888777777788888888888888887777788888888888777777777776788888888888888777777777777.
    5.9999999999999999999999988888899999888888888877777888888889999999999999999987777788888888888888888888.
    6.9999999999999999999999988888899999999999999999999999999999999999999999999999999988888899999999999999.
    7.9999999999999999999999988888899999999999999999999999999999999999999999999999888888899999999999999.
    8.9999999999999999999999999999999999999999999999999999999999999999999999999999999988888899999999999999.
      ....................................................................................................
      ^  ^                   ^     ^     ^        ^    ^         ^^         ^    ^^^    ^    ^    ^
link  #1 1       1           1     1     1        1    1         11         1    111     1 1     1
      2  2       2           2     2     2        2    2         22         2    222   2   2     2
      1  2       2           2     2     2        2    2         22         2    333     3 3     3
      9  0       1           2     3     4        5    6         78         9    012   3   4     5

 Time
Slice ....................................................................................................
    2.9999998888888888888888888888888888888888888888999999999999999999999999999999988888888888999999999999.
    3.9988888888888888888889999988888888888888888889988888888888888888886666668888888886655555566888887999.
    4.7744424333366666666666666666666644444444444555555577777776666622222333336888888888887777777788888899.
    5.8888888888999999999988888888888888888888888888222111111111113333344444683333333222444444668888878888.
    6.9999998888888888888889999999999999999999988888888888888899999999998888888211111111222333333668888889999.
    7.9999998888888888888888888888888999999999999888888999999999999999999988888999999999998877777788999999999.
    8.9988888888888888888888888888999998888888888888888889999999999998888888888888888888888888888899999999999.
      ....................................................................................................
        ^   ^^^   ^         ^     ^   ^          ^      ^ ^    ^    ^  ^ ^    ^^^       ^^ ^     ^ ^    ^^
link #  1   111   1         1     1   1          1      1 1    1    1  1 1    111       11 1     1 1    11
        2   222   2         2     2   2          2      2 2    2    2  2 2    222       22 2     2 2    22
        3   333   4         4     4   4          4      4 4    4    4  4 5    555       55 5     5 5    56
        6   789   0         1     2   3          4      5 6    7    8  9 0    123       45 6     7 8    90

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Figure 8.1: Speed Contour Map for the Eastbound Mainline Freeway (Base Run)

Analysis of Figure 8.1 reveals that the eastbound mainline freeway was experiencing congestion from time slice three to time slice six. The eastbound mainline freeway congestion patterns resulting from the simulation of the demand pattern for the modified morning peak period indicate that links 1251 and 1257 are major bottleneck locations for the eastbound mainline freeway. Recall that incident location two was link 125 1, which is one of the major bottleneck locations along the eastbound mainline freeway. It should be noted that the congestion along the eastbound mainline freeway ends by time slice seven and does not spill off of the freeway (i.e. the congestion does not reach the west end of the eastbound freeway).

Analysis of the speed contour map for the westbound mainline freeway for the base run revealed that only three links during the fourth time slice had speeds less than 50 km/h. Thus, there was essentially no queuing of vehicles on the westbound mainline freeway for the base

The base run is referred to as scenario one in the following sections and tables. The scenario one column of Table 8.1 (contained in Section 8.3) lists the total trip distances for various areas of the corridor network. The total trip distance along the eastbound freeway (430,428 vehkm) is approximately equal to that of the westbound freeway (433,741 veh.km). The total trip distance on the freeway ramps was 58,418 vehkm. The total trip distance on the arterial routes in all directions was 532,446 veh.km. Interestingly, the total trip distance on all arterial routes is not much greater than the total trip distance on just one direction of mainline freeway. The 405/1 10 row of the column indicates the total trip distance on the San Diego Freeway (I-405) and Harbor Freeway (SR-110) combined. The total trip distance on all links in the corridor network for the eight time slice simulation of the base run was 1,462,743 vehkm.

The scenario one column of Table 8.4 (contained in Section 8.3) lists the total trip time for various areas of the corridor network for the base run. The total trip time on the eastbound mainline freeway (5,771 vehhrs) was greater than that for the westbound mainline freeway (5,274 vehhrs). Greater travel times for the eastbound mainline freeway were expected based on the fact that congestion patterns were only observed on the eastbound mainline fi-eeway and not on the westbound mainline freeway. The total travel time on both the eastbound and westbound freeway ramps was 1,165 vehhrs. The total travel time on the arterial routes was 18,898 vehhrs. As expected, the total travel time on the arterials was much greater than the total travel time on both the eastbound and westbound mainline freeway. The overall travel time in the network was 3 1,264 vehhrs.

The scenario one column of Table 8.7 (contained in Section 8.3) illustrates the average speed for the various areas of the corridor network for the base run. As expected, the eastbound mainline freeway (which had an average speed of 74.6 km/h) was approximately 8 km/h slower than the westbound mainline freeway (which had an average speed of 82.2 km/h). The average speed on the freeway ramps was 50.2 km/h, with the ramps in both the eastbound and westbound directions having similar average speeds. The average speed on the arterial routes was 28.2 km/h. The overall average speed on all links in the corridor network was 46.8 km/h.

In Table 8.10 (contained in Section 8.3), the scenario one column lists some network summary statistics for the base run. The average trip time per vehicle in the Santa Monica Freeway corridor network was 7.48 minutes. The average trip length per vehicle in the network was 5.83 km. It should be noted that this average trip length appears short for a network the size of the Santa Monica Freeway corridor. The average network stops was 34.7 percent for the base run. This value indicates the level of congestion in the network; a higher percentage of network stops indicates a higher level of congestion.

8.3 INVESTIGATIONS AT INCIDENT LOCATION ONE

This section details the investigations which were conducted at incident location one. Recall that incident location one was at link 1243, which is on the eastbound mainline freeway between Crenshaw Boulevard and Arlington Avenue. A 2x3 matrix of investigations was conducted at this incident location. Two different levels of information quality were studied. For the first level, motorists were provided perfect information on link travel times. For the second level, motorists were provided with link travel times that were distorted by a normally distributed error of 20 percent. Three different levels of incidents were considered; a non-incident situation, a minor incident situation and a major incident situation. The characteristics of the minor and major incident were discussed in Section 8.1.9.

Table 8.1 contains the total trip distances in vehkrn for the various areas of the corridor network for the six investigations conducted with incident location one. Scenario one is the base run (0 percent distortion and no incident) that was discussed in the previous section. Scenario two and three investigated the impact of a minor incident and major incident at incident location one, respectively, when motorists were provided perfect link travel time data. Scenarios four through six were the investigations with motorists being supplied link travel time data with a 20 percent distortion. Scenarios four, five and six studied the impact of a non-incident, minor incident and major incident, respectively, when motorists were supplied distorted link travel time information. Table 8.2 contains the changes in total trip distances for various areas of the corridor network for scenarios two through six as compared to scenario one, the base run. Table 8.3 illustrates the percentage changes in total trip distances. One must be careful when viewing percentage changes, as the magnitude of the base run will influence the size of the percentage changes.

Tables 8.4, 8.5 and 8.6 are of the same format as Tables 8.1, 8.2 and 8.3, except Tables 8.4 through 8.6 are for the total travel time in veh.hrs. Tables 8.7, 8.8 and 8.9 are also of the same format as Tables 8.1, 8.2 and 8.3, except Tables 8.7 through 8.9 illustrate the average speeds in km/h for the various areas of the corridor network. Finally, Table 8.10 lists some summary statistics for scenarios one through six for the investigations conducted at incident location one.

Analysis of the speed contour maps for both the eastbound and westbound mainline freeway were also completed for scenarios two through six. As expected, the major incident generate more congestion than the minor incident, and the minor incident generated more congestion than the non-incident situation, along the eastbound mainline freeway. This was observed for the investigations at both levels of information quality. There were no noticeable changes in the westbound mainline freeway's speed contour maps for the non-incident situation, minor incident situation and major incident situation.

The congestion patterns for scenarios two though six did not extend into the eighth time slice or extend beyond the west side of the eastbound mainline freeway or the east side of the westbound mainline freeway. If the congestion did extend beyond the freeway end or beyond the simulation time then a complete measure of the level of congestion in the network would not be possible. Thus, the summary statistics in the tables contained in this section capture the full impact of the variation in traffic performance that resulted from the investigation.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	**0 %**	0 %	0 %	20 %	20 %	20 %
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	430428	429460	419830	426916	427055	417839
Fre_wb	433741	434049	43393 1	431269	43 1294	431139
Fre_tot	864169	863509	853761	858185	858349	848978
Ramp_eb	27650	27689	28692	28969	28683	29218
Ramp_wb	30768	30888	31006	31583	31583	3 1645
Ramp-tot	58418	58577	59698	60552	60266	60863
405/1 10	7710	7718	7721	7677	7665	7703
Arterial	532446	531571	543878	551287	550352	561567
Overall	1462743	1461375	1465058	1477701	1476632	1479111

Table 8.1: Incident Location One - Total Trip Distance (veh.km)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	**0 %**	**0%**	20 %	20 %	20 %
Incident	None	Minor	Maior	None	Minor	Maior
Fre_eb	0	**-968**	-10598	-3512	-3373	-12589
Fre_wb	0	308	190	-2472	-2447	-2602
Fre_tot	0	-660	-10408	-5984	-5820	-15191
Ramp_eb	0	39	1042	1319	1033	1568
Ramp_wb	0	120	238	815	815	877
Ramp-tot	0	159	1280	2134	1848	2445
405/1 10	0	8	11	-33	-45	-7
Arterial	0	-875	11432	18841	17906	29121
Overall	0	-1368	2315	14958	13889	16368

Table 8.2: Incident Location One - Difference in Total Trip Distance (veh.km)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	0 %	0 %	20 %	20 %	20 %
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	0.0	-0.2	-2.5	-0.8	**-0.8**	-2.9
Fre_wb	0.0	0.1	0.0	-0.6	-0.6	-0.6
Fre_tot	0.0	-0.1	-1.2	-0.7	-0.7	-1.8
Ramp_eb	0.0	0.1	3.8	4.8	3.7	5.7
Ramp-wb	0.0	0.4	0.8	2.6	2.6	2.9
Ramp-tot	0.0	0.3	2.2	3.7	3.2	4.2
405/1 10	0.0	0.1	0.1	-0.4	-0.6	-0.1
Arterial	0.0	-0.2	2.1	3 . 5	3.4	5.5
Overall	0.0	-0.1	0.2	1.0	0.9	1.1

Table 8.3: Incident Location One - Percent Change in Total Trip Distance

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	**0%**	**0 %**	**0%**	20 %	20 %	20 %
Incident	None	Minor	Maior	None	Minor	Major
Fre_eb	5771	5844	6248	5767	5738	6206
Fre_wb	5274	5267	5256	5225	5234	5236
Fre_tot	11045	11111	11504	10992	10972	11442
Ramp_eb	544	549	651	622	605	659
Ramp-wb	621	619	622	658	670	669
Ramp-tot	1165	1168	1273	1280	1275	1328
405/1 10	156	156	157	156	156	157
Arterial	18898	18721	19545	19774	19750	20332
Overall	3 1264	31156	32479	32202	32153	33259

Table 8.4: Incident Location One - Total Trip Time (veh.hrs)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	0 %	**0 %**	20%	20%	20%
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	0	73	477	-4	-33	435
Fre_wb	0	-7	-18	-49	-40	-38
Fre_tot	0	66	459	-53	-73	397
Ramp-eb	0	5	107	78	61	115
Ramp-wb	0	-2	1	37	49	48
Ramp-tot	0	3	108	115	110	163
405/1 10	0	0	1	0	0	1
Arterial	0	-177	647	876	852	1434
Overall	0	-108	1215	938	889	1995

Table 8.5: Incident Location One - Difference in Total Trip Time (veh.hrs)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	**0%**	0 %	20 %	20 %	20 %
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	0.0	1.3	8.3	-0.1	-0.6	7.5
Fre_wb	0.0	-0.1	-0.3	-0.9	-0.8	-0.7
Fre_tot	0.0	0.6	4.2	-0.5	-0.7	3.6
Ramp_eb	0.0	0.9	19.7	14.3	11.2	21.1
Ramp-wb	0.0	-0.3	0.2	6.0	7.9	7.7
Ramp-tot	0.0	0.3	9.3	9.9	9.4	14.0
405/1 10	0.0	0.0	0.6	0.0	0.0	0.6
Arterial	0.0	-0.9	3.4	4.6	4.5	7.6
Overall	0.0	-0.3	3.9	3.0	2.8	6.4

Table 8.6: Incident Location One - Percent Change in Total Trip Time

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	0 %	0 %	20 %	20 %	20%
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	74.6	73.5	67.2	74.0	74.4	67.3
Fre_wb	82.2	82.4	82.6	82.5	82.4	82.3
Fre_tot	78.2	77.7	74.2	78.1	78.2	74.2
Ramp-eb	50.9	50.4	44.1	46.6	47.4	44.4
Ramp_wb	49.6	49.9	49.8	48.0	47.2	47.3
Ramp-tot	50.2	50.2	46.9	47.3	47.3	45.8
405/ 110	49.3	49.3	49.2	49.1	49.2	49.2
Arterial	28.2	28.4	27.8	27.9	27.9	27.6
Overall	46.8	46.9	45.1	45.9	45.9	44.5

Table 8.7: Incident Location One - Average Speed (km/h)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	0 %	0 %	20 %	20 %	20%
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	0.0	-1.1	-7.4	-0.6	-0.2	-7.3
Fre_wb	0.0	0.2	0.4	0.3	0.2	0.1
Fre_tot	0.0	-0.5	-4.0	-0.1	0.0	-4.0
Ramp-eb	0.0	-0.5	-6.8	-4.3	-3.5	-6.5
Ramp-wb	0.0	0.3	0.2	-1.6	-2.4	-2.3
Ramp-tot	0.0	0.0	-3.3	-2.9	-2.9	-4.4
405/1 10	0.0	0.0	-0.1	-0.2	-0.1	-0.1
Arterial	0.0	0.2	-0.4	-0.3	-0.3	-0.6
Overall	0.0	0.1	-1.7	**-0.9**	-0.9	-2.3

Table 8.8: Incident Location One - Difference in Average Speed (km/h)

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion	0 %	0 %	0 %	20 %	20 %	20%
Incident	None	Minor	Major	None	Minor	Major
Fre_eb	0.0	-1.5	-9.9	-0.8	-0.3	-9.8
Fre_wb	0.0	0.2	0.5	0.4	0.2	0.1
Fre_tot	0.0	-0.6	-5.1	-0.1	0.0	-5.1
Ramp-eb	0.0	-1.0	-13.4	-8.4	-6.9	-12.8
Ramp_wb	0.0	0.6	0.4	-3.2	-4.8	-4.6
Ramp-tot	0.0	0.0	-6.6	-5.8	-5.8	-8.8
405/1 10	0.0	0.0	-0.2	-0.4	-0.2	-0.2
Arterial	0.0	0.7	-1.4	-1.1	-1.1	-2.1
Overall	0.0	0.2	-3.6	-1.9	-1.9	-4.9

Table 8.9: Incident Location One - Percent Change in Average Speed

	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Distortion Incident	0 % None	0 % Minor	0 % Major	**20%** None	**20 %** Minor	**20 %** Major
Avg. trip time per veh. (min.)	**7.48**	**7.45**	**7.77**	**7.70**	7.69	7.95
Avg. trip length per veh. (km)	**5.83**	**5.83**	**5.84**	**5.89**	5.89	5.90
Avg. network stops (%)	**34.7**	**34.7**	**35.5**	**36.4**	**36.3**	37.1

Table 8.10: Incident Location One - Summary Statistics

8.3.1 Scenario Two: Incident Location One, Perfect Information, Minor Incident

Scenario two investigated a minor incident at incident location one with motorists receiving perfect information. Analysis of total trip distances indicates that the total trip distance along the eastbound mainline freeway actually decreased over the base run, but the change is not very significant. The total trip distances were varied only slightly by the minor incident at incident location one. These variations could easily be due to noise in the simulation. Analysis of the total trip time indicates a slight increase in the eastbound mainline travel time (+1.3 %), but overall the total trip time variations are very small and could be due to noise. Analysis of the average speeds reveals similar patterns. There are essentially no changes in the network summary statistics over the base run.

Under this scenario the adverse effect of the minor incident was offset by the fact that all vehicles in the system were guided vehicles (comparing scenario one and two). In other words, for minor incidents at incident location one, a 100 percent IVIS system offset the adverse effects of the minor incident.

8.3.2 Scenario Three: Incident Location One, Perfect Information, Major Incident

Scenario three investigated a major incident at incident location one with motorists receiving perfect link travel time data. The total trip distances were significantly decreased on the eastbound mainline freeway (-2.5 %) and significantly increased on the eastbound ramps (+3.8 %). The total trip distance was also significantly increased on the arterial routes (+2.1 %). Overall, there was not a significant increase in the total trip distance (+0.2 %) with the major incident as compared to the equivalent non-incident scenario, which is the base run. The total trip times increased by 8.3 % on the eastbound mainline freeway and by 19.7 % on the eastbound ramps. The total trip time on the arterial routes also increased a noticeable amount (+3.4 %). These total trip time values indicate that vehicles on both the eastbound mainline freeway and ramps, and the arterial routes, are significantly impacted by a major incident on the eastbound mainline freeway. The net effect of the major incident was an increase of 3.9 % in total trip time on all links in the corridor network. The magnitude of these travel time costs is interesting since the network is loaded entirely with guided vehicles. The average speed statistics predicted for this investigation reveal the same patterns as the total trip time statistics. Summary statistics from the network indicate that average trip time per vehicle was noticeably increased (+0.29 minutes), but the average trip length per vehicle is relatively unchanged. The average network stops has increased over the base run, which indicates higher levels of congestion in the network.

Under this scenario, although the adverse effect of the major incident can be partially offset by all vehicles in the system being guided vehicles, travel times on almost all sub-portions of the corridor network were increased (comparing scenario one and three).

8.3.3 Scenario Four: Distorted Information, No Incident

Scenario four studied the impact of distorting the link travel time data that motorists use to build their minimum routing path trees. The total trip distance was reduced on the mainline freeway in both directions (a reduction of just under 1 %) and increased on both the ramps and the arterials. The most significant increases in total trip distance were on the eastbound ramps (+4.8 %) and the arterials (+3.5%). Overall, the total trip distance increased by 1 .O %. These increases in total trip distance are larger than those of scenario three, which was perfect information with a major incident. These values indicate that a reduction in the level of information quality provided to motorists results in significant increases in arterial travel and reductions in freeway travel.

Analysis of total trip times indicated significant increases in ramp trip time (+14.3 % for the eastbound ramps and +6.0 % for the westbound ramps). The total trip time on the arterial routes was also noticeably increased (+4.6 %). These values indicate that vehicles are probably being routed from the freeway to arterial routes. The net effect of distorting the link travel time data was a 3.0 % increase in total trip time in the corridor network. Analysis of average speeds revealed that the average speeds for the freeway have not varied significantly, but the average ramp speeds are much lower. The average speed on the arterial routes were also slightly reduced (-1.1 %) when compared to the base run. Thus, vehicles are being routed onto the arterial routes, which has resulted in slower arterial routes. Overall statistics indicate that the average trip time and distance per vehicle was increased. The average network stops was increased, which means higher congestion levels in this investigation as compared to the base run.

It is interesting to note that overall the total trip time in the network for this scenario, which is a non-incident situation, is almost equivalent to that of scenario three, which was a major incident situation with motorists being supplied perfect information. The increase in overall total trip distance was much larger for this scenario than for the third scenario.

Under this non-incident scenario, both total travel distances and total travel times were increased due to distorted information in the IVIS system (comparing scenario one and four). The effect of distorted information was almost as adverse as a major incident with motorists receiving perfect information (comparing scenario three and four).

8.3.4 Scenario Five: Incident Location One, Distorted Information, Minor Incident

Scenario five was an investigation at incident location one under a minor incident situation when motorists are supplied distorted travel time information upon which they build their minimum path trees. Comparing the total trip distance predicted for this scenario with that of the above scenario (distorted information under non-incident conditions) reveals that there was no significant changes in the total trip distance by the minor incident. The largest change was a 0.9 % decrease in total trip distances on the eastbound ramps. These values are very interesting, as they imply that even when motorists are supplied with distorted information, there is no significant impact of the minor incident in terms of total travel distance. Comparing this investigation to scenario one reveals that the total trip distance in the network only increased by 0.9 %.

Comparing the total travel time predicted for this scenario to the fourth scenario (distorted information under non-incident conditions) indicates that the eastbound ramp's total trip time decreased by 3.1 % and the westbound ramp's total trip time increased by 1.9 %, otherwise there were no significant changes in total trip time due to the minor incident. It is interesting to note that the ramp trip time for the eastbound freeway declined when a minor incident was modeled on the eastbound mainline freeway. The net effect of the minor incident and distorted information was an increase of 2.8 % in total trip time in the network as compared to the base run (scenario one).

The analysis of the average speeds in the network reveal similar trends to the changes in total trip time. Comparing the summary statistics predicted for this scenario to that of scenario four reveals no significant changes. Thus, the traffic performance in this scenario is very similar to that of scenario four, which was identical to scenario five except that it was under non-incident conditions.

With distorted information in the IVIS system, the added adverse effect of the minor incident was offset by the fact that all vehicles in the system were guided vehicles (comparing scenario four and five). However, providing distorted information in the IVIS system had a greater adverse effect than a minor incident (comparing scenario two and five).

8.3.5 Scenario Six: Incident Location One, Distorted Information, Major Incident

The sixth and final investigation conducted at incident location one was a study of the impact of providing motorists distorted information under a major incident situation. Comparing the total trip distance of this scenario to that of scenario four (distorted information with non-incident conditions) indicates that the eastbound mainline freeway's total trip distance decreased significantly (-2.1 %) while the westbound mainline freeway's total trip distance was unchanged. The total trip distance slightly increased on the eastbound ramps (+0.9 %) and noticeably increased on the arterial routes (+2.0 %) as compared to scenario four. Thus, eastbound traffic is being routed from the eastbound mainline freeway to the arterial routes. The net effect of a major incident and distorted information was an increase of 1.1 % in total trip distance over the base run.

Analysis of Tables 8.4 through 8.7 reveals that the total trip time on the eastbound mainline freeway and eastbound ramps dramatically increased over that of scenario four (+7.6 % and +6.8 % respectively) and over that of scenario one (+7.5 % and +21.1 % respectively). In terms of total trip time, there were no significant impacts of the major incident on the westbound mainline freeway. The total trip time on the arterial routes increased by 3.0 % as compared to scenario four and 7.6 % as compared to scenario one, the base run. These values indicate that the costs of a major incident can not be overcome by a network loaded entirely with IVIS. Comparing the total trip time of this scenario to scenario four reveals that the overall total trip time increased 3.4 %, where this similar comparison with motorists being supplied perfect information revealed an increase of 3.9 % in total trip time. Average speeds in the network revealed similar patterns as the total trip time analysis.

The overall network statistics predicted for this scenario indicate that the major incident resulted in an increase in average trip time per vehicle of 0.25 minutes as compared to scenario four, which is similar to the increase observed with the non-distorted information (+0.29 minutes). The average trip length per vehicle for this scenario as compared to that of scenario four was not significantly increased by the major incident. The average percentage of network stops did increase, which was expected due to the increased congestion in the network caused by the major incident.

Under this scenario, although the adverse effect of the major incident can be partially offset by all vehicles in the system being guided vehicles, travel times on almost all sub-portions of the corridor were increased (comparing scenario four and six). However, the distorted information of the IVIS system has almost the same adverse effect as the major incident comparing scenario three and six).

8.4 INVESTIGATIONS AT INCIDENT LOCATION TWO

This section presents the two investigations which were conducted at incident location two. Recall that incident location two was at link 125 1, which is on the eastbound mainline freeway between Normandie Avenue and Vermont Avenue. This incident location, as shown in Figure 8.1, is one of the major bottleneck locations along the eastbound mainline freeway. Thus, an incident at this location will result in higher costs than a similar incident (same start time, end time and duration) at incident location one. The first investigation conducted at this location (scenario seven) was a minor incident with motorists being supplied perfect link travel time information. The second investigation conducted at this location (scenario eight) was a major incident with motorists being supplied perfect link travel time information. The traffic performance predicted for both of these runs were compared to that of scenario one (non-incident conditions with motorists receiving perfect information). Tables 8.11 through 8.20 contain the results from this comparison. These tables illustrate total trip distances (Tables 8.11, 8.12 and 8.13), total trip times (Tables 8.14, 8.15 and 8.16), average speeds (Tables 8.17, 8.18 and 8.19) and network statistics (Table 8.20).

	Scenario 1	Scenario 7	Scenario 8
Distortion Incident	0 % None	0 % Minor	0 % Major
Fre_eb	430428	423031	413476
Fre_wb	433741	433487	433378
Fre_tot	864169	856518	846854
Ramp_eb	27650	28775	29927
Ramp_wb	30768	30881	31023
Ramp_tot	58418	59656	60950
405/110	7710	7709	7711
Arterial	532446	541402	556967
Overall	1462743	1465285	1472482

Table 8.11: Incident Location Two - Total Trip Distance (veh.km)

	Scenario 1	Scenario 7	Scenario 8
Distortion Incident	0 % None	0 % Minor	0 % Major
Fre_eb	0	-7397	-16952
Fre_wb	0	-254	-363
Fre_tot	0	-7651	-17315
Ramp_eb	0	1125	2277
Ramp_wb	0	113	255
Ramp_tot	0	1238	2532
405/110	0	-1	1
Arterial	0	8956	24521
Overall	0	2542	9739

Table 8.12: Incident Location Two - Difference in Total Trip Distance (veh.km)

	Scenario 1	Scenario 7	Scenario 8
Distortion Incident	0 % None	0 % Minor	0 % Major
Fre_eb	0.0	-1.7	-3.9
Fre_wb	0.0	-0.1	-0.1
Fre_tot	0.0	-0.9	-2.0
Ramp_eb	0.0	4.1	8.2
Ramp_wb	0.0	0.4	0.8
Ramp_tot	0.0	2.1	4.3
405/110	0.0	0.0	0.0
Arterial	0.0	1.7	4.6
Overall	0.0	0.2	0.7

Table 8.13: Incident Location Two - Percent Change in Total Trip Distance

	Scenario 1	Scenario 7	Scenario 8
Distortion	0 %	0 %	0 %
Incident	None	Minor	Major
Fre_eb	5771	6235	6524
Fre_wb	5274	5257	5264
Fre_tot	11045	11492	11788
Ramp-eb	544	647	831
Ramp_wb	621	631	654
Ramp-tot	1165	1278	1485
405/1 10	156	156	156
Arterial	18898	19500	20576
Overall	3 1264	32426	3 4 0 0 5

Table 8.14: Incident Location Two - Total Trip Time (veh.hrs)

	Scenario 1	Scenario 7	Scenario 8
Distortion	**0 %**	**0 %**	0 %
Incident	None	Minor	Major
Fre_eb	0	464	753
Fre_wb	0	-17	-10
Fre_tot	0	447	743
Ramp-eb	0	103	287
Ramp-wb	0	10	33
Ramp-tot	0	113	320
405/1 10	0	0	0
Arterial	0	602	1678
Overall	**0**	1162	2741

Table 8.15: Incident Location Two - Difference in Total Trip Time (veh.hrs)

	Scenario 1	Scenario 7	Scenario 8
Distortion	**0 %**	**0%**	0 %
Incident	None	Minor	Major
Fre_eb	0.0	8.0	13.0
Fre_wb	0.0	-0.3	-0.2
Fre_tot	0.0	4.0	6.7
Ramp_eb	0.0	18.9	52.8
Ramp_wb	0.0	1.6	5.3
Ramp-tot	0.0	9.7	27.5
405/110	0.0	0.0	0.0
Arterial	**0.0**	3.2	8.9
Overall	0.0	3.7	8.8

Table 8.16: Incident Location Two - Percent Change in Total Trip Time

	Scenario 1	Scenario 7	Scenario 8
Distortion	0 %	0 %	0 %
Incident	None	Minor	Major
Fre_eb	74.6	67.9	63.4
Fre_wb	82.2	82.5	82.3
Fre_tot	78.2	74.5	71.8
Ramp-eb	50.9	44.5	36.0
Ramp-wb	49.6	48.9	47.5
Ramp-tot	50.2	46.7	41.1
405/110	49.3	49.3	49.4
Arterial	28.2	27.8	27.1
Overall	46.8	45.2	43.3

Table 8.17: Incident Location Two - Average Speed (km/h)

	Scenario 1	Scenario 7	Scenario 8
Distortion	**0%**	0 %	0 %
Incident	None	Minor	Major
Fre_eb	0	-6.7	-11.2
Fre_wb	0	0.3	0.1
Fre_tot	0	-3.7	-6.4
Ramp-eb	0	-6.4	-14.9
Ramp_wb	0	-0.7	-2.1
Ramp-tot	0	-3.5	-9.1
405/1 10	0	0.0	0.1
Arterial	0	-0.4	-1.1
Overall	**0**	-1.6	-3.5

Table 8.18: Incident Location Two - Difference in Average Speed (km/h)

	Scenario 1	Scenario 7	Scenario 8
Distortion	0 %	0 %	0 %
Incident	None	Minor	Major
Fre_eb	0.0	-9.0	-15.0
Fre_wb	0.0	0.4	0.1
Fre_tot	0.0	-4.7	-8.2
Ramp-eb	0.0	-12.6	-29.3
Ramp_wb	0.0	-1.4	-4.2
Ramp-tot	0.0	-7.0	-18.1
405/110	0.0	0.0	0.2
Arterial	0.0	-1.4	-3.9
Overall	0.0	-3.4	-7.5

Table 8.19: Incident Location Two - Percent Change in Average Speed

	Scenario 1	Scenario 7	Scenario 8
Distortion Incident	0 % None	**0 %** Minor	**0 %** Major
Avg. trip time per veh. (min.)	7.48	7.76	8.13
Avg. trip length per veh. (km)	5.83	5.84	5.87
Avg. network stops (%)	34.7	35.6	36.6

Table 8.20: Incident Location Two - Summary Statistics

An analysis of the speed contour maps for both the eastbound and westbound mainline freeway was completed for scenarios seven and eight. Both the minor and major incident at incident location two resulted in heavier congestion levels on the eastbound mainline freeway than the congestion levels observed for the minor and major incident at incident location one with motorists receiving perfect information. Comparison of the westbound mainline freeway's speed contour maps for scenario two and three (minor and major incident at incident location one with perfect information provided to motorists) to those for scenario seven and eight did not reveal any significant changes.

The congestion patterns for scenarios seven and eight did not extend beyond the west side of the eastbound mainline freeway or the east side of the westbound mainline freeway or into the eighth time slice. Thus, the summary statistics contained in Tables 8.11 through 8.20 capture the full impact of the variation in traffic performance that resulted from the investigations.

8.4.1 Scenario Seven: Incident Location Two, Perfect Information, Minor Incident

The seventh scenario investigated a minor incident at incident location two with motorists being supplied perfect link travel time information. Analysis of Tables 8.11 through 8.13 reveals that the total trip distance on the freeway mainline eastbound noticeably decreased (-1.7 %) and increased on the eastbound ramps (+4.1 %). The total trip distance for the westbound mainline freeway and ramps was not significantly impacted. The total trip distance on the arterial routes was also increased (+1.7 %). Considering the entire network, there was not a significant increase in the total trip distance as a result of this minor incident (+0.2 %).

Analysis of the total trip time reveals a significant increase in the total trip time on the eastbound mainline freeway (+8.0 %), eastbound ramps (+18.9 %) and arterial routes (+3.2 %). The total trip time on the westbound mainline freeway and ramps was not significantly impacted by the major incident. Considering the entire network, there was a significant increase in the total trip time of 3.7 %. Recall that the analysis of scenario two indicated that the minor incident at incident location one with motorists receiving perfect information did not result in significant changes in total trip time. Analysis of average speeds indicates similar patterns to that of the total trip time.

The network statistics predicted for the scenario seven investigation indicate that the average trip time per vehicle was increased by 0.28 minutes, but there were no significant changes in the average trip length per vehicle. The average network stops increased, which is due to the higher levels of congestion along the eastbound mainline freeway, eastbound ramps and arterial routes.

Recall that the adverse effect of the minor incident at incident location one with motorists receiving perfect information (scenario two) was offset by the fact that all vehicles in the network were guided vehicles. However, under this scenario the adverse effect of the minor incident can only be partially offset by all vehicles in the system being guided vehicles (comparing scenario one and seven). Adverse effects were observed on the eastbound mainline fi-eeway, eastbound ramps and arterial routes for this scenario. It should be noted that the adverse effect of the minor incident at incident location two were slightly less than the adverse effect of the major incident at incident location one (comparing scenario three and seven).

8.4.2 Scenario Eight: Incident Location Two, Perfect Information, Major Incident

Scenario eight was an investigation of the impact of a major incident at incident location two when motorists were provided perfect information. Analysis of Tables 8.11 through 8.13 indicates that the total trip distance on the eastbound mainline freeway was significantly reduced (-3.9 %), whereas the total trip distance on the eastbound ramps was significantly increased (+8.2 %). The total trip distance on the westbound mainline freeway and ramps was not significantly impacted by the major incident. For the arterial routes, the total trip distance was significantly increased (+4.6 %). Thus, a significant volume of traffic is being routed from the eastbound mainline freeway to the arterial routes. The net effect of the major incident was an increase in total trip distance of 0.7 %.

Analysis of Tables 8.14 through 8.16 shows that the total trip time for scenario eight along the eastbound mainline freeway and ramps was significantly increased (+13.0 % and +52.8 % respectively). The westbound mainline freeway's total trip time was not impacted by the major incident, but the westbound ramps were significantly impacted (+5.3 %). The total trip time on the arterial routes was also significantly increased (+8.9 %). Overall, the total trip time in the network was increased by 8.8 %. Thus, the major incident resulted in substantial increases in the total trip times for most areas of the corridor. The magnitude of these trip time costs was fairly surprising given that the entire network is loaded with guided vehicles receiving perfect link travel time data.

The average speeds predicted for the various components of the corridor followed the same pattern as the total trip times. It is interesting to note that for the major incident situation at incident location two, the average speeds on the eastbound mainline freeway were reduced by 15.0 %, whereas the major incident at incident location one (scenario three) resulted in only a 9.9 % reduction in average eastbound mainline freeway speeds. For scenario eight, the average speed for the entire network was reduced by 7.5 %.

The network summary statistics for scenario eight, as compared to scenario one (perfect information under non-incident conditions), revealed that the average trip time per vehicle increased by 0.65 minutes. The average trip time per vehicle for scenario eight was greater than that for scenarios one through seven. For scenario eight, the average trip length per vehicle was only slightly increased. The average network stops was greater than both scenarios one and seven, which was expected with the higher congestion levels in the network under the major incident situation.

Under this scenario, although the adverse effect of the major incident can be partially offset by all vehicles in the system being guided vehicles, travel times on almost all sub-portions of the corridor network were substantially increased (comparing scenario one and eight).

8.5 CONCLUSIONS

The initial investigation plan was to study the impact of various ATIS and ATMS control strategies in the morning peak period, midday period and the evening peak period. However, due to time and fund restrictions, coupled with limitations of the INTEGRATION model, the investigation stage of the project was restricted to limited ATIS studies with a modified morning peak period.

Investigations of ATIS control strategies were severely limited since attempts to establish an INTEGRATION run with a network loaded entirely with unguided vehicles were unsuccessful. Thus, the base run for the ATIS investigations was with a network loaded entirely with guided vehicles. Investigations to study the impact of supplying motorists two different levels of information quality were conducted. The INTEGRATION model was also capable of simulating variations in the number of surveillance links in the network, CMS and HAR. However, due to extremely limited project time tests to validate these features of the INTEGRATION model were not conducted by the research team. Tests to validate these features could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Thus, investigations to study these untested control strategies were not considered for this research effort.

The INTEGRATION model was only capable of limited investigations of ATMS control strategies. The model was not able to simulate optimized ramp meter timing plans or optimized signal coordination along an arterial route. The optimization of individual signals required implementation in the calibration stage of the project, thus this feature was not available to be incorporated into the final design of experiment. The INTEGRATION model had proven to be robust in simulating HOV lanes, but these control strategies were classified as lower priority investigations and were not studied during this research effort.

Investigations with supply coding modifications were possible and were conducted at two different incident locations with both a minor incident and major incident. Investigations with demand coding modifications were not possible, due to problems encountered in calibrating time slices with heavy demand patterns. The investigations conducted for this stage of the project studied the impact of IVIS with different levels of information quality under both non-incident and incident situations in a network loaded entirely with guided vehicles.

The demand pattern for the morning peak period was modified to reduce the level of congestion in the network due to problems encountered in the calibration of time slices with heavy traffic demands. The modified morning peak period was an eight time slice run (including the warm-up time slice) that was created from the origin/destination matrices for the first three time slices of the morning peak period, which were all reasonably calibrated during the calibration stage of the project. These time slices were combined to form a demand pattern of 1-1-2-3-2-1-1-1.

The investigations were conducted at two incident locations. The first incident location was on the eastbound mainline freeway upstream of the major bottlenecks which were predicted along the eastbound mainline freeway in the base run. Six investigations were conducted at this location; two levels of information quality (perfect information and distorted information) and three incident situations (no incident and a minor and major incident) were all studied. These investigations revealed that for a minor incident, a 100 percent IVIS system can offset the adverse effect of the minor incident. However, the adverse effect of the major incident along the eastbound mainline freeway in the Santa Monica Freeway corridor, can only be partially offset by all vehicles in the system being guided vehicles. Under the major incident situation, travel times on almost all sub-portions of the corridor network were in increased.

The investigations with the distorted information indicated that a reduction in the level of information quality provided to motorists results in significant increases in total arterial trip distances and reductions in total freeway trip distances. The overall total trip time in the corridor when vehicles were provided distorted information was almost equivalent to that when vehicles were provided perfect information under the major incident situation. Thus, in terms of total trip times the effect of distorted information was almost as adverse as a major incident with motorists receiving perfect information.

The second incident location was on the eastbound mainline freeway at a major bottleneck location. The minor incident at this location resulted in an adverse effect to vehicles on the eastbound mainline freeway and ramps, and to vehicles traveling along the arterial routes. The major incident at this second location had a larger impact on eastbound vehicles and the arterial routes than the minor incident. Thus, the costs of both the minor and major incident at a bottleneck location on the eastbound mainline freeway could only be partially offset by all vehicles in the system being guided vehicles.

Chapter 9: SUMMARY

The purpose of this final chapter of the report was to highlight some of the major findings and conclusions of this three year, multi-person research effort in applying the INTEGRATION model to the Santa Monica Freeway corridor in Los Angeles. This simulation study, like most simulation studies, consisted of the following four major activities:

- Selecting simulation model and site for application,
- Assembling and coding model input data,
- Testing and calibrating the model,
- Undertaking investigations with the model.

The major findings of this study are presented under each of these major activities in the following sections, and the conclusions are presented at the end of the chapter.

9.1 SELECTING SIMULATION MODEL AND SITE FOR APPLICATION

The research team had previously assessed the available simulation models which could be employed in investigating ATMS and ATIS strategies in a freeway corridor. The key required features of the models included origin/destination demand estimation, traffic assignment coupled with traffic simulation, and the ability to simulate ATIS and ATMS strategies in a freeway corridor. Three models, CONTRAM, INTEGRATION and SATURN, were found to be most applicable in these earlier studies. The research team gained experience in using the CONTRAM and INTEGRATION models. While recognizing that all models had some deficiencies, the INTEGRATION model was determined to be the most appropriate for this current study.

While many difficulties were encountered with applying the INTEGRATION model in this study, it is still considered to be the model most suitable for studies similar to this one. Two new model developments, CORFLO/CORSIM and DYNASMART, are currently underway and should also be considered for future studies of this nature.

The previous experience with models was covered in Chapter 2 and an overview of the INTEGRATION model was presented in Chapter 3. This documentation of the INTEGRATION model in Chapter 3, is considered to be one of the most comprehensive available today.

The Santa Monica Freeway corridor was selected as the site for this current study because of the previous experience of the research team in working in this corridor, and the cooperative arrangements which developed with the California Department of Transportation and the City of Los Angeles. While an excellent site for considering ATMS and ATIS implementations, the physical size, time duration of the study, and complexity of the freeway corridor provided significant challenges to the research team.

The freeway portion of the freeway corridor included over twenty. directional freeway miles with great variability in geometric features. There were thirty on-ramps and thirty off-ramps along the freeway with varying merge and diverge design configurations, and varying intersection designs where the ramps connected to the arterial street system.

The arterial portion of the freeway corridor included over 200 direction arterial street miles and over 500 intersections. The intersections included yield sign controlled, stop sign controlled, and signal controlled intersections. The 3 12 signalized intersections included a wide variety of multi-lane configurations and multi-phase signal timing plans.

The study duration extended from 6:00 A.M. to 8:00 P.M., a total of fourteen hours which was divided into twenty-eight 30 minute time slices. The target was to obtain traffic counts on over 3200 links for each of the twenty-eight time slices which represented almost 100,000 link flows.

Previous simulation modeling experience in the Santa Monica Freeway corridor was described in Chapter 2. The general characteristics of the freeway and arterial street portions of the corridor are described near the beginning of Chapter 4.

9.2 ASSEMBLING AND CODING MODEL INPUT DATA

The flow chart of project activities contained in Figure 1.1 in Chapter 1 pictorially illustrates that the assembly and coding of model input data included freeway data and arterial data which in turn included in each supply, demand, and control sub-data sets. The effort to assemble and code input data required over a year's efforts with two to three members of the research team involved. Once the sub-data sets were assembled, data was coded and inputted into the INTEGRATION model. Data collection and preparation of model input was covered in considerable detail in Chapters 4 and 5.

The freeway and arterial street data were assembled separately, and because of the delay in obtaining the arterial data, work on the freeway-only portion of the corridor proceeded first. Delays in the arterial data collection stage of the project were due to two factors. First, the data collection efforts were hampered by delays in the installation of loop detectors along the arterial streets. Secondly, the Northridge earthquake occurred in the middle of the project which further delayed data collection efforts. Only through the cooperation of the California Department of Transportation and the City of Los Angeles could such a gigantic data set be assembled.

An important contribution of the research team efforts was the compilation of all available traffic count information for all portions of the Santa Monica Freeway by half-hour time periods from 6:00 A.M. to 8:00 P.M. These counts included all previous traffic counts over the past five years as well as comprehensive data sets collected by the research team.

Although causing a delay in the project schedule, the City of Los Angeles collected an excellent comprehensive traffic count data set for most of the arterial streets in the corridor using their recently operational ATSAC system. Unfortunately turning movements were only available at a few intersections. Another contribution of the research team efforts was the compilation of these traffic counts in a systematically developed set of spreadsheets.

The final step in this phase of the project was inputting the assembled supply, demand, and control data into the INTEGRATION model. The final coding of the freeway corridor included 111 origin and destination nodes, 1747 nodes, and 3286 links. The capacities were estimated for each link and appropriate speed-flow curves were also developed for each link. A plot of the final freeway corridor configuration was shown in Figure 5.11.

The time-of-day intersection (312) and ramp (30) traffic signal plans were coded into the INTEGRATION model. Because of the complex signalized intersections and the heavy traffic demands, a unique expanded intersection coding scheme was developed as shown in Figure 5.5.

A demand estimation process was developed as shown in Figure 5.17 which connected interactively the QUEENSOD program with the main program, INTEGRATION. In addition, the research team developed a large number of auxiliary programs which were used to enhance the input coding process, provide for the connection between QUEENSOD and INTEGRATION models, and later for checking and summarizing the model outputs.

The final coding for the INTEGRATION model included the following four data files: node descriptor, link descriptor, signal timing plans, and origin/destination demand matrices.

It should be noted that the effort required to collect and code data is directly proportional to the size and complexity of the network. A network as large and complex as the Santa Monica Freeway corridor required over a year's efforts to collect and code the model input data. Even with a data collection effort this large, problems were still encountered in the calibration efforts due to the need for an even more extensive and accurate data base of demand data (i.e. link flows).

9.3 TESTING AND CALIBRATING THE MODEL

Due to the continuous updating of the INTEGRATION model and limited number of real-world applications of the INTEGRATION model, thorough testing of the model was required. The effort completed by the research team in the testing of the INTEGRATION model was detailed in Chapter 6. The testing of the model analyzed the INTEGRATION model's ability to model both freeway and arterial links, however the freeway portion of the corridor was emphasized due to its importance in the corridor simulation.

INTEGRATION model tests with two simple freeway networks indicated that the INTEGRATION versions 1.5d and 1.5e were predicting similar traffic performance and that these predictions were similar to traffic performance estimates from a well established simulation model (FREQ) and analytical solutions. Tests with a simple arterial network indicated that INTEGRATION versions 1.5d and 1.5e were reasonably modeling arterial links.

To test the INTEGRATION model on a larger scale the freeway-only portion of the Santa Monica Freeway corridor was coded and calibrated for both INTEGRATION versions 1.5d and 1.5e. In order to judge the performance of the INTEGRATION model, the freeway-only portion of the Santa Monica Freeway corridor was accurately coded and calibrated with the FREQ model, which is a well established freeway simulation model.

With some modifications to the coding of freeway off-ramps, the INTEGRATION 1.5e modeling of the eastbound mainline freeway was fairly reasonable. However, there were still some questions about the performance of the INTEGRATION model in simulating the freeway-only portion of the corridor. The INTEGRATION model was predicting higher congestion levels on the eastbound mainline freeway during the morning peak period. Comparison of the link flows predicted by INTEGRATION 1.5e and FREQ revealed differences on the order of plus or minus 500 vehicles per hour.

The testing of the INTEGRATION model's ability to model HOV lanes determined that the INTEGRATION model is very robust in its abilities to simulate a wide range of HOV facilities.

Future simulation projects utilizing the INTEGRATION model should attempt to obtain much more detailed documentation on the INTEGRATION model and many of its subsidiary programs than was available for this research effort. Detailed documentation on the methodologies programmed into the INTEGRATION model is crucial.

Various features of the INTEGRATION model were tested and modified as a result of this research effort. Future simulation projects should also obtain detailed reports on the testing and validation of the various features of the INTEGRATION model.

While not completely satisfied with the results of the mode1 testing effort, a decision was made to move ahead into the calibration effort with the anticipation that these remaining problems could be overcome in the calibration effort or if not, the calibration effort and the later planned investigations would be redesigned considering these limitations.

The calibration stage of the research project was one of the most extensive stages of the project, involving over one year's efforts. Due to time and fund restrictions, coupled with problems encountered in the use of the INTEGRATION model, the calibration stage of the project only attempted the calibration of fourteen of the twenty-eight time slices to be simulated. Only six of these fourteen time slices were classified as calibrated at the conclusion of the calibration effort. The calibration effort conducted by the research team was detailed in Chapter 7.

The corridor calibration was difficult because a network of the size and complexity of the Santa Monica Freeway corridor had never been successfully calibrated using the INTEGRATION model nor an equivalent model. The calibration effort focused initially on a single expanded intersection and then on calibrating the entire Santa Monica Freeway corridor. A huge effort was undertaken during the calibration of the first time slice to refine the demand estimation process that will be utilized during the calibration of all subsequent time slices. Once the problems associated with the calibration of the first time slice were resolved the calibration of the morning peak period and midday period were undertaken.

The attempts to calibrate the INTEGRATION model for the first early-morning uncongested time slice, which included work to refine the demand estimation process, required three months of effort on the part of the research team. A major accomplishment was the identification of problems in the modeling effort, and incorporating refinements in the demand estimation and traffic simulation process in an attempt to overcome or minimize these problems. The research team received guidance from the INTEGRATION model developer's during this stage of the calibration effort and both participated in the model refinements.

An incremental approach was followed during the calibration of the morning peak period (eight time slices) and midday period (ten time slices). The effort to calibrate the morning peak period resulted in a reasonable calibration of time slices one, two and three. However, after an extensive effort, both at U.C. Berkeley and Queen's University, a reasonable calibration of time slices four, five and six was not attained. The calibration of the last two time slices of the morning peak period was not attempted. It should be noted that the model developer was not under contractual obligations to the research team, but did provide guidance and recommendations throughout the calibration of the morning peak period and did take the opportunity of attempting to calibrate the morning peak period.

The calibration effort for the midday period achieved a marginally reasonable calibration for time slices nine, ten and eleven, but not for time slices twelve through sixteen. The calibration of the last two time slices of the midday period was not attempted.

Future research should further improve the demand estimation process. Improvements in the demand estimation process would lead to the generation of more accurate origin/destination matrices, which would produce a more reasonable calibration. Further work to determine the optimal number of minimum path trees and optimal generation time of those minimum path trees, and to determine the optimal arterial link flow reliability factors, should be undertaken.

The calibration effort undertaken for this research project highlights the high correlation between network size and complexity, and the calibration effort required. A more successful calibration would surely have resulted if a more comprehensive and more accurate demand data base (i.e. observed link flows) was available. Future data collection efforts should attempt to obtain traffic volumes of all turning movements at all intersections. Observed turning movements will improve the modeling of traffic performance at signalized intersections. Since unrealistic delay at many signalized intersections in the corridor was one of the main reasons why the time slices with heavy demands could not be calibrated, improved modeling of traffic at intersections will allow a more successful calibration effort.

It should be noted that the research team devoted over a years' efforts to the data collection effort. This effort did result in one of the most comprehensive and complete corridor data bases ever assembled.

9.4 UNDERTAKING INVESTIGATIONS WITH THE MODEL

The investigations conducted by this research effort primarily focused on the impact of in-vehicle information systems (IVIS) with different levels of information quality under non-incident and various incident conditions. An evaluation of the INTEGRATION model's potential ability to model ATIS and ATMS control strategies and the results of the investigations conducted was contained in Chapter 8.

Investigations of ATIS control strategies were severely limited since attempts to establish an INTEGRATION run with a network loaded entirely with unguided vehicles were unsuccessful. Thus, the base run for the ATIS investigations was with a network loaded entirely with guided vehicles. Investigations to study the impact of supplying motorists two different levels of information quality were conducted. The INTEGRATION model was also capable of simulating variations in the number of surveillance links in the network, CMS and HAR. However, due to extremely limited project time tests to validate these features of the INTEGRATION model were not conducted by the research team. Tests to validate these features could not be found in any of the literature on the previous applications of the INTEGRATION model available to the research team. Thus, investigations to study these untested control strategies were not considered for this research effort.

The INTEGRATION model was only capable of limited investigations of ATMS control strategies. The model was not able to siiulate optimized ramp meter timing plans or optimized signal coordination along an arterial route. The optimization of individual signals required implementation in the calibration stage of the project, thus this feature was not available to be incorporated into the final design of experiment. The INTEGRATION model had proven to be robust in simulating HOV lanes, but these control strategies were classified as lower priority investigations and were not studied during this research effort.

Investigations with supply coding modifications were possible and were conducted at two different incident locations with both a minor incident and major incident. Investigations with demand coding modifications were not possible, due to problems encountered in calibrating time slices with heavy demand patterns. The investigations conducted for this stage of the project studied the impact of IVIS with different levels of information quality under both non-incident and incident situations in a network loaded entirely with guided vehicles.

The demand pattern for the morning peak period was modified to reduce the level of congestion in the network due to problems encountered in the calibration of time slices with heavy traffic demands. The modified morning peak period was an eight time slice run (including the warm-up time slice) that was created from the origin/destination matrices for the first three time slices of the morning peak period, which were all reasonably calibrated during the calibration stage of the project.

The investigations were conducted at two incident locations. The first incident location was on the eastbound mainline fi-eeway upstream of the major bottlenecks which were predicted along the eastbound mainline freeway in the base run. Six investigations were conducted at this location; two levels of information quality (perfect information and distorted information) and three incident situations (no incident and a minor and major incident) were all studied. These investigations revealed that for a minor incident, a 100 percent MS system can offset the adverse effect of the minor incident. However, the adverse effect of the major incident along the eastbound mainline freeway in the Santa Monica Freeway corridor, can only be partially offset by all vehicles in the system being guided vehicles. Under the major incident situation, travel times on almost all sub-portions of the corridor network were increased.

The investigations with the distorted information indicated that a reduction in the level of information quality provided to motorists results in significant increases in total arterial trip distances and reductions in total freeway trip distances. The overall total trip time in the corridor when vehicles were provided distorted information under the non-incident situation was almost equivalent to that when vehicles were provided perfect information under the major incident situation. Thus, in terms of total trip times the effect of distorted information was almost as adverse as a major incident with motorists receiving perfect information.

The second incident location was on the eastbound mainline freeway at a major bottleneck location. The minor incident at this location resulted in an adverse effect to vehicles on the eastbound mainline freeway and ramps, and to vehicles traveling along the arterial routes. The major incident at this second location had a larger impact on eastbound vehicles and the arterial routes than the minor incident. Thus, the costs of both the minor and major incident at a bottleneck location on the eastbound mainline freeway could only be partially offset by all vehicles in the system being guided vehicles.

9.5 CONCLUDING OBSERVATIONS AND RECOMMENDATIONS

The concluding observations and recommendations from this research report summarize the observations and discuss the potential enhancements in the data collection effort and in the INTEGRATION model's performance. Some general remarks on computer simulation modeling are also presented.

The data collection effort included the assembly and coding of both freeway and arterial data. During the model calibration for both the freeway-only simulation and the freeway corridor simulation the importance of complete and accurate freeway on-ramp and off-ramp volumes and freeway-to-freeway connector volumes was realized. During the freeway calibration work the need for accurate and proper freeway performance measures, such as temporal and spatial velocity patterns and freeway end-to-end travel times, was reconfirmed. The need to collect turning movement volumes at all signalized intersections was also realized during the calibration stage of the project. During the calibration of time slices with heavy demands unrealistic traffic performance was being simulated at a number of intersections which resulted in queue spill-back problems in the arterial network. A more successful calibration would surely have resulted if all turning movement volumes at all signalized intersections were available.

Thus, it is critical for future freeway data collection efforts to collect complete and accurate freeway on-ramp and off-ramp volumes and freeway-to-freeway volumes. It is also important that freeway performance measures be collected during the freeway data collection efforts. For detailed simulation modeling, it is critical that accurate and complete signalized intersection turning movement volumes be included in the arterial data collection efforts.

The data collection effort for this research project, which collected intersection approach volumes and only a few intersection turning movement volumes, required over a year's efforts with two to three members of the research team involved to collect and code data. It should be noted that the effort required to collect and code data is directly proportional to the size and complexity of the network.

The link flow data was used to derive a synthetic origin-destination demand matrix. This research project highlighted the difficulties these models have in determining the actual origin-destination demand patterns. Future freeway corridor simulation efforts should try to obtain actual origin-destination demand patterns from driver survey studies or from other sources. Since these studies are very time consuming and expensive, the use of a travel demand model should be considered to generate a more accurate origin-destination demand matrix.

The performance of the INTEGRATION model was determined during the model testing, model calibration and investigation stages of the research project. The five potential areas for improvements in the INTEGRATION model that were identified during this research were;

- Further testing and enhancements of the opposing link feature of the INTEGRATION model are required. The initial calibration efforts revealed that detailed simulation of a signalized intersection in INTEGRATION required an eight node/twelve link configuration, which would allow for the coding of left-turning movements which are delayed by an opposing vehicle flow. The opposing link feature of INTEGRATION should be enhanced to calculate the amount of delay imposed on left-turning movements as a function of the geometry of the signalized intersection.

- The INTEGRATION model's capability to simulate ATMS control strategies needs to be expanded. The generation and simulation of optimal signal timing plans along an arterial route and optimal freeway ramp metering plans should be incorporated into the INTEGRATION model.

- An improvement is required in the process in which a base run with a network loaded entirely with unguided vehicles for ATIS investigations is established. The ability to establish an INTEGRATION simulation of a network loaded entirely with unguided vehicles, which uses the travel time information or minimum path trees generated from a run with a network loaded entirely with guided vehicles, that reasonably matches the simulation run with the network loaded entirely with guided vehicles needs to be improved.

- The output generated by the INTEGRATION model should be enhanced in order to ease the analysis of the simulation output. The user should be able to obtain speed, density and flow contour maps for a set of user-specified links.

- A reduction in the run time of the INTEGRATION model is required. Extremely slow run times of several days hampered calibration and investigation efforts for this project.

While many difficulties were encountered with applying the INTEGRATION model in this study, it is still considered to be the model most suitable for studies similar to this one.

The effort and time required to complete a detailed and accurate fourteen hour simulation of the entire Santa Monica Freeway corridor was underestimated by the research team. Over a year's efforts were spent to assemble a comprehensive input data set, which resulted in the best available single source of these data collected to date for the Santa Monica Freeway corridor. The data requirements for a more successful calibration effort were much greater than the project was able to collect, The model testing and calibration stages of the project, which also required over a year's efforts, developed and refined a systematic procedure for the calibration process and demand estimation process. These stages also identified and corrected and/or minimized some of the problems with the INTEGRATION model, with help from the model developer. Some of the improvements still required in the INTEGRATION model are identified above. Given the time, budget and talent available for this project, only a partial calibration was achieved and only limited investigations were conducted. Future simulation projects should be very conservative in their estimation of the effort required for data collection and coding, model testing and calibration, and investigations. The effort required to simulate a network is directly proportional to the size (both temporal and spatial) and complexity of the network.

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