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16. Abstract

With the ever-increasing congestion facing freeways, engineers must explore every possible tool at their disposal. Freeway ramp closures present a potentially underutilized approach for mitigating freeway traffic problems. Selecting and implementing such closures, however, presents a very difficult problem. Due to the potential public outcry and traffic disturbances that can result from misused ramp closure, special care must be given to ramp closure deployment. Even when a feasible ramp closure deployment is known, numerous possible implementation possibilities exists ranging from simply signing, to manual and automatic gate operations.

This report documents the research to develop a formal laboratorial analysis procedure for evaluating peak-period ramp closure strategies, and produce guidelines for successful implementation. These guidelines consider the potential implementation solutions and their application for TxDOT, including the before-and-after-implementation system performance assessment and monitoring plans. The field implementation and evaluation of ramp closure was not a part of this research project. However, the field evaluation plan proposed in this report will provide guidance for undertaking such endeavor.

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Development and Assessment of Peak-Period Ramp Closure Strategies for Interstate Highways

By

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Technical Report 0-4764-1
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Development and Evaluation of Peak-Period Ramp Closure Strategies

Conducted for the

**Texas Department of Transportation
And the
Federal Highway Administration**

By

The Center for Transportation Infrastructure Systems
The University of Texas at El Paso
El Paso, TX 79968-0516

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ABSTRACT

With the ever-increasing congestion facing freeways, engineers must explore every possible tool at their disposal. Freeway ramp closures present a potentially underutilized approach for mitigating freeway traffic problems. Selecting and implementing such closures, however, presents a very difficult problem. Due to the potential public outcry and traffic disturbances that can result from misused ramp closure, special care must be given to ramp closure deployment. Even when a feasible ramp closure deployment is known, numerous implementation possibilities exist ranging from simply signing, to manual and automatic gate operations.

This report documents the research to develop a formal laboratorial analysis procedure for evaluating peak-period ramp closure strategies, and produce guidelines for successful implementation. These guidelines consider the potential implementation solutions and their application for TxDOT, including the before-and-after-implementation system performance assessment and monitoring plans. The field implementation and evaluation of ramp closure was not a part of this research project. However, the field evaluation plan proposed in this report will provide guidance for undertaking such endeavor.

EXECUTIVE SUMMARY

In spite of the advent of Intelligent Transportation Systems (ITS) technologies, and improved practices for traffic operation, freeway traffic management continues to be one of the most challenging tasks that engineers encounter in maintaining satisfactory mobility of Texas highway networks. Such difficulties and challenges arise from the continuing growth of passenger and goods movements along major transportation corridors, as well as evolving and intensified urban activities in metropolitan areas in Texas.

Among all the freeway management strategies, controlling freeway inflow/outflow has been a widely used approach. The type of technique, as defined by the latest Freeway Management and Operation Handbook (FHWA, 2003) and the Ramp Control and Management Handbook (2004, ongoing) include, ramp metering, entrance ramp closure, and exit ramp closure. Ramp metering is perhaps the most widely applied and fully tested technique among the three. The use of ramp metering aims to prevent freeway mainline traffic from breaking down, and maintains reasonable throughput and levels of service when the mainline traffic is onset to its capacity. However, ramp metering may not be a feasible solution under special flow/geometric restrictions.

This research explored the possibility of applying peak-hour ramp closure as a freeway flow control and queuing jumping control strategy. This research proposed a comprehensive framework from the laboratory evaluation to field operational testing and ongoing

performance monitoring and assessment. Following the proposed framework, in the ramp closure benefit and impact analysis procedure, four categories of criteria including "Freeway Level-of-Service Analysis", "Regional Surface Traffic Impacts", "Level of Closure Information Provision", and "Safety Impact (freeways and arterials)" will be used to evaluate the feasibility and operational characteristics of ramp closure. Moreover, four categories of operational strategies will be evaluated based on the above criteria to determine the optimal configuration of the integrated operational strategies in conjunction with ramp closure. The four types of operational strategies are "Closure Time and Duration", "Closure Information Provision Strategies", "ITS Strategies", and "Freeways/Arterials Control Integration". It should be noted that proper integration of possible network operational strategies provides the crucial opportunities needed to make ramp closure work. Simply executing the ramp closure without implementing a package of comprehensive and integrated traffic management strategies will reduce the likelihood of making the ramp closure a successful freeway operation strategy.

After conducting series of rigorous laboratory study, the research concludes the following findings:

- Peak-hour ramp closure has been found to be a low-cost and effective strategy for both freeway main-lane flow control and managing queue jumping applications.
- Ramp metering has been shown ineffective or unfeasible when the traffic flow in the downstream of the metered ramp is over the capacity. Metering the ramp does not improve the traffic flow conditions, it also imposes excessive queue on the ramp. In the study case, due to the short length of the ramp, the queue spills back to the upstream intersection for a significantly period of time. Closing the ramp, equivalent to zero metering rate, is more effective in preventing intersection spil!backs and minimize violations.
- Establishing a suite of traffic control and impact mitigation strategies is the key for a successful implementation of peak-hour ramp closure. These strategies include:
    - Synchronizing the adjacent intersection signal to a special timing plan, such as "all red", in conjunction with the transition of gate closure to prevent collision onset of closure.

- Information provision/advance warning are crucial to prevent last minute diversion and/or confusion at the gate. It also facilitates better traffic diversion farther upstream of the closed ramp. Mobile CMS or DMS should be used, particularly during the short-term evaluation period, to promote public awareness of the peak-hour closure.
- Continuous performance assessment and improvement should be undertaken to ensure consistent and satisfactory operating performance of both the freeways and arterials.

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1 INTRODUCTION

In spite of the advent of Intelligent Transportation Systems (ITS) technologies, and improved practices for traffic operation, freeway traffic management continues to be one of the most challenging tasks that traffic engineers encounter in maintaining satisfactory mobility of highway networks. Such difficulties and challenges arise from the continuing growth of passenger and goods movements along major transportation corridors, as well as evolving and intensified urban activities in metropolitan areas in the U.S.

Among all freeway management strategies, controlling freeway inflow/outflow has been a widely used approach. This type of technique, as defined by the latest Freeway Management and Operation Handbook (FHWA, 2003) includes, ramp metering, entrance ramp closure, and exit ramp closure. Ramp metering is perhaps the most widely applied and fully tested among the three. The use of ramp metering aims to prevent freeway mainline traffic from breaking down, and maintains reasonable throughput and levels of service when the mainline traffic is onset to its capacity.

Most of the related technical documents suggest engineers use entrance and exit ramp closures under very restrictive circumstances or only in situations where ramp metering is ineffective, because of the possibility of under-utilizing freeway capacity, over-flooding alternate routes and public concern when ramp closure is not applied carefully. The discussions of typical situations where ramp closure is recommended appear in several technical documents, including the Freeway Management and Operation Handbook (FHWA, 1997, 2003), Traffic Operations Manual (TxDOT, 1998), and Intelligent Transportation Systems (ITS) Design Manual (Wisconsin DOT, 2000). These discussions are summarized as follows:

1. The entrance ramp does not provide sufficient storage length to prevent queues of vehicles waiting to enter the freeway from interfering with surface street traffic.
2. Traffic demand on the freeway immediately upstream is at capacity, and an alternate route with adequate capacity is available.
3. Even if the upstream traffic demand is less than downstream capacity, the rate at which traffic could be allowed to enter the freeway might be so low that it would not be possible to control the entrance of ramp traffic without a large number of violations. In this case, it would be more practical to close the ramp in order to prevent congestion on the freeway.
4. Entrance ramp introduces serious weaving problems.

1.1 Lessons Learned from Previous Research

Although the idea of closing entrance ramps for optimizing freeway traffic flow first appeared nearly 30 years ago (Miesse, 1967), this strategy has never been fully investigated over the years. The Freeway Management Handbook (FHWA, 1997) briefly reports several successful implementation projects in a number of cities in the United States and Japan (e.g. Houston, Los Angeles, San Antonio, Fort Worth, Osaka and Tokyo) but no detailed technical information was described. Prevedouros (1999) conducted one of the most recent relevant studies. This study investigated the benefits and impacts of peak-hour ramp closure on the H-1 freeway in Honolulu, Hawaii. The motivations of applying ramp closure instead of ramp metering on one inner-city segment of the H-1 freeway in Honolulu during the morning peak hour were the lack of sufficient ramp storage and acceleration length, and high density of ramps. Simulations on a 10.5 km segment of the westbound H-1 freeway were conducted using KRONOS and INTEGRATION. The simulation software was able to replicate existing conditions well and identified a prime candidate on-ramp where closure or metering may produce considerable flow improvements. A two-week ramp closure experiment with traffic cones was undertaken along with extensive data collection (volumes, moving observer travel times, AUTOSCOPE-derived speeds). The simulated and actual results were compared.

The two-week experimental results did not actually meet the researchers' and the HDOT's expectations. During the experiment in which one on-ramp was closed using temporary control devices from 6 am to 10 am, the average speeds on both downstream and upstream segments of the closed ramp on the H-1 mainline were mostly worse than average speeds observed before the closure. The average speeds were worse in the outer lanes, indicating that motorists drive more cautiously with the presence of control devices (cones). The downstream average speeds improved at the end of the two-week experiment, indicating that motorists learned and adapted to ramp control configurations. In any case, the upstream average speeds were consistently lower than average speeds prior to the experiment.

The HDOT project researchers provided extensive discussions on the possible causes for the underperforming traffic conditions, and hinted that public perception was one of the major found benefits. Nonetheless, we discuss some field experiment issues that could be improved upon that particular experience.

(1) The Honolulu experiment implemented two-week advance announcement on the ramp closure to the traveling public using three portable changeable message signs (CMS) at three locations (one on mainline upstream of the closed ramp, one on upstream frontage road interchange, and the other on the downstream frontage road.) If many motorists decide to enter the freeway at upstream ramps, it will certainly increase the traffic volume at those areas. Such a phenomenon is common when motorists adjust their driving routes in response to any roadway configuration change. Such an effect may disappear

when motorists stabilize their driving pattern after a certain period of adjustment and adaptation.

(2) The Honolulu study may have overlooked the day-to-day adaptation of motorists' driving behavior and resulting traffic dynamics. Implementing a ramp closure introduces a rather drastic change in the physical freeway network connectivity. Although two-week notice was given to the traveling public, those who used to enter the freeway via the closed ramp need to modify their routes, and perhaps try other different routes around the impacted area. If the majority of the impacted drivers decide to enter the freeway at an immediately upstream ramp, worsened traffic conditions can certainly be observed there initially. However, some of them may decide to try other new routes or to select different ramps to enter the freeway, thus alter the traffic dynamic. In other words, such adjustments could continue taking place, until those affected motorists settle down to acceptable routes. Literature shows that such adjustment process could easily go beyond the two-week trial period (Srinivasan, 2000.) Data observed in this period is likely to be unrepresentative and unreliable. As such, more extended data collection period would be needed.

Nonetheless, the project concluded that motorists' perceptions toward the experiment were rather positive despite the mixed results and modest actual improvement, which was encouraging for the HDOT engineers to continue the effort of using innovative approaches in managing freeway traffic.

It can be concluded that the peak-period (variable) ramp closure techniques have not received thorough and in-depth investigation in the past. The generally restrictive use of peak-hour ramp closure can be largely attributed to the lack of understanding of its operational characteristics. This report documents an effort to unveil its potential benefit and impact through analytical and engineering approaches, aiming to obtain the FHWA's approval for operational field-testing of a peak-period ramp closure at the El Paso site discussed in the preceding section.

1.2 Federal Highway Administration Inter-State Access Application Guidelines

Every roadway improvement planning, real-time operation and evaluation of traffic impact studies on highways across the nation requires the supervision from the Federal Highway Administration (FHWA). The FHWA classifies ramp closure strategies as either temporary or permanent.[1] Recently, the closing of access ramps has been implemented to improve traffic conditions on highway main lanes and arterials in the surrounding areas. A ramp closure may also be caused by work zones in the area. According to the FHWA, ramp closure is an extreme strategy as it restrains traffic behavior that has been established over a significant period.[2] Some cases consider closing on-ramps to impede vehicle access to incorporate traffic on a highway. Other cases might consider closing exit ramps to monitor traffic on both arterials and highway.

Parallel to this effort, the FHWA TMC pooled-fund study – "Development of Ramp Management and Control Handbook"- briefly discusses the ramp closure as one type of ramp control strategy. The specific decision-making process, as illustrated in Figure 1-1, examines the ramp closure based on three categories: event related, time of day, or recurrence. The peak-hour ramp closure, focal study subject of this research, is categorized as the "time of day" type of closure. At the production of this report, the research group of the Ramp Management and Control Handbook has not provided any specific recommendation for analyzing the benefit/impact of peak-hour ramp closure.

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[1] http://tmcpfs.ops.fhwa.dot.gov/cfprojects/uploaded_files/CH%202-Final%20AOutline-v.1.1%20clean.doc U.S. Department of Transportation Federal Highway Administration. TMC Pooled-Fund Study (PFS)

[2] Freeway Management Handbook. Chapter 7. Website: http://ops.fhwa.dot.gov/Travel/traffic/freeway_management_handbook/chapter7_01.htm, U.S. Department of Transportation Federal Highway Administration. TMC Pooled-Fund Study.

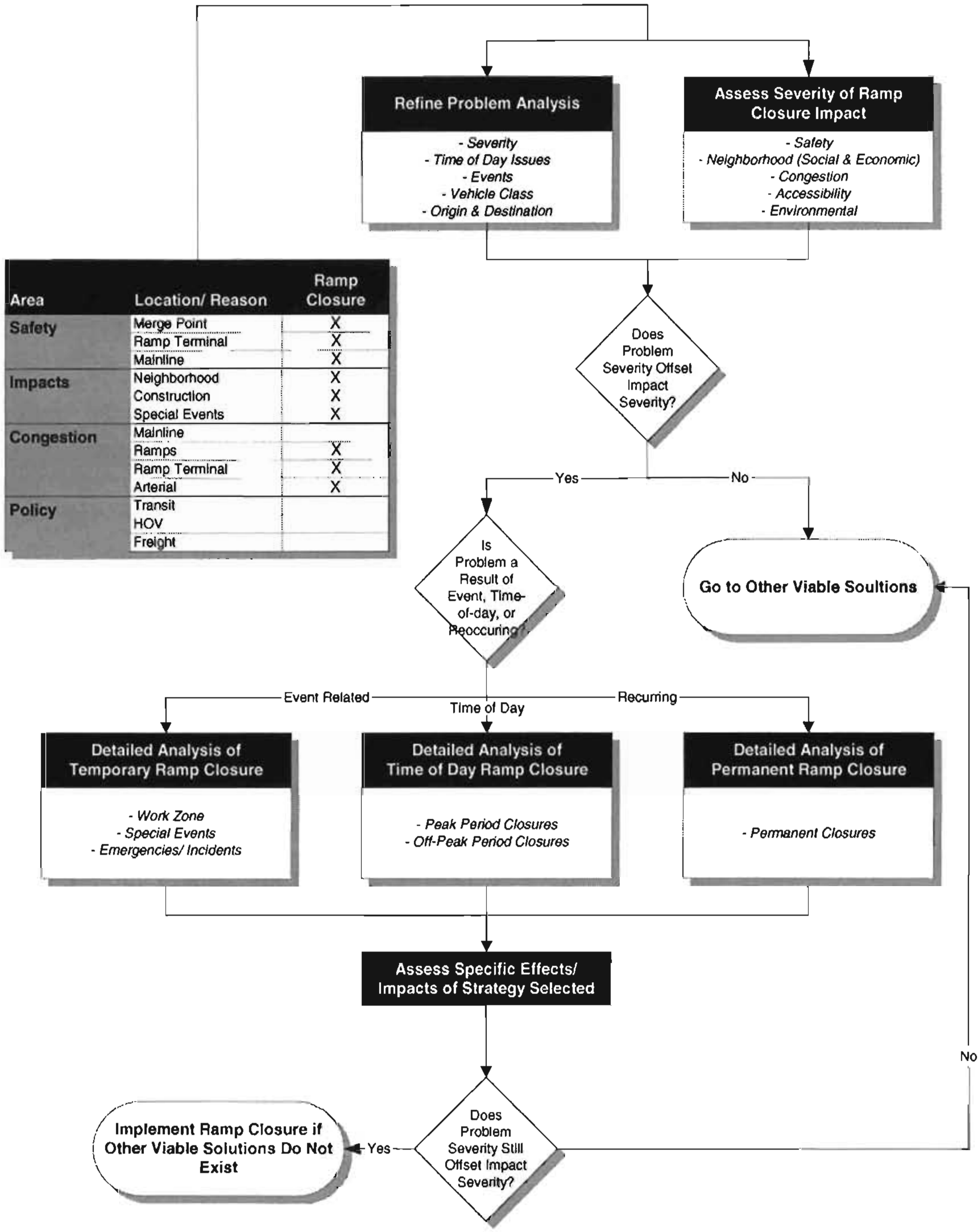

Figure 1-1 Ramp Closure Decision Tree (Ramp Management and Control Handbook, draft, 2004)

The handbook emphasizes on analyzing the traffic impact related to the closure. Closing a ramp can eliminate the need for complex traffic control that addresses both traffic entering the facility and traffic already on the facility[3] reducing motorist delay and improving safety. The FHWA also encourages traffic analysts to include public information as well as public involvement prior to implementation.[4] Although the FHWA defines a ramp closure as the simplest form of controlling traffic on-ramps, the administration advises to resort to this technique as a final alternative.

The ramp closure implementation is devised by means of automatically or manually placed either vertical or horizontal gates. Special attention must be paid to the benefit impact for each individual scenario. As previously mentioned, duration of the ramp closure must be considered. The FHWA classifies closure of a ramp to be permanent as the best approach in order to avoid driver confusion.[5] Thus, temporal closure must implement additional signs to inform upstream traffic of current conditions of the ramp.

Although no specific guidelines have been provided, special attention must be paid to the reduction of driver confusion prior to implementation. At this point, the administration may only provide recommendations for the methodology applied to the ramp closure operation. In general, the FHWA concerns main issues abutting from this control system regarding processes that are used to analyze and select ramps to be closed, strategies employed to mitigate impacts, record of negative and positive impacts, specific challenges encountered in planning and significant lessons learned for future reference.[6] Prior to any implementation of ramp closure, either classification (permanent or temporary), daily operations should be documented and related back to practices presented in a manual for each specific case. Documented

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[3] http://ops.fhwa.dot.gov/wz/resources/publications/FullClosure/CrossCutting/its.htm
[4] Turner-Fairbank Highway Research Center website: http://www.tfhrc.gov/pubrds/04may/01.htm.
[5] http://tmcpfs.ops.fhwa.dot.gov/cfprojects/uploaded_files/CH%205-Final%20Aoutline-v.1.2%20clean.doc U.S. DOT FHWA. TMC Pooled-Fund Study (PFS).
[6] http://tmcpfs.ops.fhwa.dot.gov/cfprojects/uploaded_files/CH%205-Final%20Aoutline-v.1.2%20clean.doc U.S. DOT FHWA. TMC Pooled-Fund Study (PFS).

items should include conditions such as closure and opening procedures if closure is temporal and monitoring traffic near the closure.

1.3 Research Objectives and Goals

The objective of this analysis is to study the feasibility of applying peak-hour (variable) ramp closure as a viable freeway management strategy. Central to the analysis is to investigate various traffic operation strategies in conjunction with ramp closure, and to develop recommendations for implementation, including traffic engineering, geometric consideration, and benefit evaluations. These objectives entail the following:

- To characterize conditions that warrant the application of ramp closure.
- To develop recommendations for integrated traffic operation strategies using conventional traffic engineering approaches and/or ITS technologies (if available) from traffic management and safety improvement perspectives.
- To develop an evaluation plan for continual improvement of ramp closure implementation.

2 RAMP CLOSURE GATE OPERATIONS

To implement a ramp closure scheme, gate operation issue, including the crashworthiness of the gate, traffic signal interconnection issues also need to be addressed. Generally, three types of ramp closures are considered in practice:

- **Temporary closure**: entrance ramps may be closed temporarily in response to maintenance or construction activities on either the freeway or the adjacent frontage roads or surface streets. It is common for a ramp to be closed by police during management of the downstream incident.

- **Variable schedule**: because of extreme recurring downstream capacity deficiencies, ramps may be closed during certain peak periods and open at off-peak times. Automated gate operations are recommended from the standpoint of operation efficiency and cost. This type of ramp closure is the most relevant to freeway traffic management, and is of the greatest interest to this research project.

- **Permanent closure**: a ramp may be closed on a permanent basis due to changes in the freeway systems or demand patterns. Concrete barriers or other physical constraints are recommended.

Methods of entrance ramp closure that have been used in current systems include manual barriers, automated gates, and signs. Experience in Detroit and Los Angeles has indicated that signs alone cannot affect an entrance ramp closure (Wattleworth et al. 1968; Newman et al. 1969.) Automated barriers enable an entrance to be closed and opened automatically, which tend to increase the flexibility of closure as a means of control. Since manual placement of barriers is labor intensive, this approach is best suited for short-term or trial control projects. The three types of ramp closure-control methods are briefly discussed as follows.

- Manual placed barrier such as barricades, barrels, or cones.

  As shown in Figure 2-1 to Figure 2-3, this type of ramp control requires manual placement of the control device between storage and deployment.

Figure 2-1 Cone gate

Figure 2-2 Type III Barricade Traffic Gate (Stored Position)

Figure 2-3 Type III Barricade Traffic Gate (Deployed)

- Automated barriers such as gates used at railroad crossing.

  This type of gate could be operated on a manual, pre-timed or traffic responsive mode. The swing arms as shown in Figure 2-4 to Figure 2-6, could function in a vertical swing or horizontal swing mode. The horizontal swing gate is commonly used in lane changing traffic control applications, such as High Occupancy Vehicle (HOV) lane control. The vertical swing gate has wider applications ranging from toll plaza to weather-related freeway control. Both types of gates can be controlled from an on-site cabinet or remotely from a traffic management center.

Figure 2-4 Horizontal swing gate(Chicago)

Figure 2-5 Vertical Swing Arm Traffic Gate (Open Position), ITS Design Manual, Wisconsin DOT

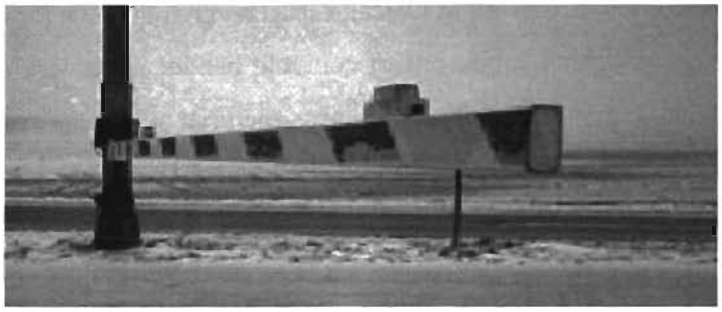


Figure 2-6 Vertical Swing Arm Traffic Gate (Closed Position)

- Signing

Using only signs or signals to control ramp closure is less common because of the difficulty in managing violations. However, it has been applied by the Minnesota DOT to control variable ramp closure, as shown in Figure 2-7. This ramp originally had a gate arm, along with the lights and signs, but MnDOT decided the maintenance did not warrant keeping the gate operational and the arm was removed. An average of 30-40 daily violations was observed.

Figure 2-7 Variable ramp closure using signing in St. Paul, Minnesota

2.1.1 *Chicago Reversible Lanes*

The Kennedy Expressway in Chicago uses reversible lanes for controlling congestion. Swing Gates, which rotate out of concrete barrier walls, are used to redirect traffic away from entry ramps. The gate arm material used is aluminum. Rotating drum message signs are used to indicate if the reversible lane is open or closed. Fiber optic auxiliary signs are used to warn the motorists that the gates are closing. Restraining barriers are deployed across each ramp to safely stop errant motorists. Video cameras are used to identify errant vehicles. The gates used were purchased from B&B Electrical which were approved by the FHWA. The gates were therefore not crash tested by the Illinois DOT.

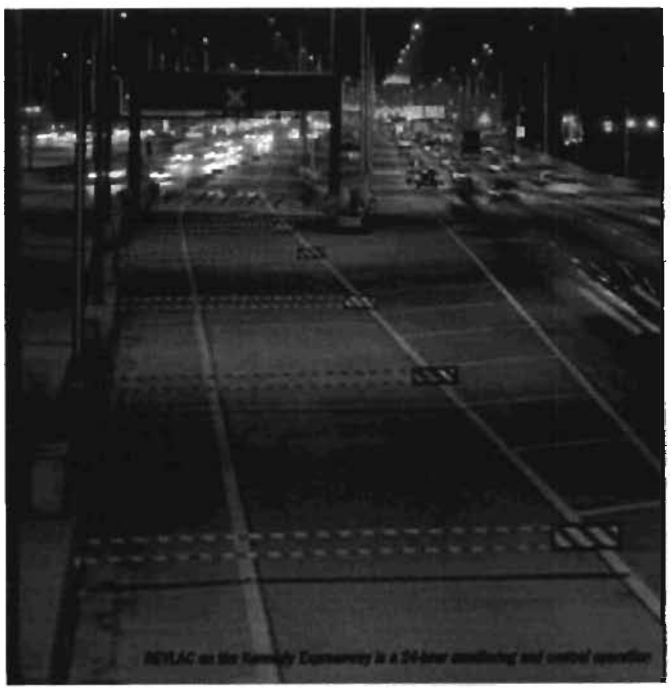


Figure 2-8 Chicago reversible lanes

2.1.2 *Minnesota*

In Minnesota, automated gates have been used in the IH-90 and US 71 interchange. These gates were originally used to stop traffic from entering the roadway during poor weather conditions. However, when the automated gates were used during periods of severe congestion or incidents they were found to be extremely effective in prevention of large traffic build up. The automated gates are initiated and remotely controlled from the office. Cameras located at the site allow the gates to be remotely controlled depending on prevailing traffic conditions. The gate was purchased in component form and then assembled by their partner, Greg Thomson of ThomTech. The gate arm was made of aluminum. The gate used is very similar to the one used on railroads. The gate components were purchased from Traffic Safety Corporation out of Kentucky. The cost of the gate was $8,200 without labor. The cost of fixing the damaged gate arms due to an accident was found to be around $1,200 without labor. No tests for crashworthiness were conducted by the Minnesota DOT. However, the manufacturer previously crash tested the gates.. FHWA approval was not obtained in closing the ramps. However, the closure was conducted according to the Minnesota DOT policy for road closure, which had been approved by the FHWA. A representative from the FHWA helped in drafting up the guidelines for implementation of the closures.

Figure 2-9 Minnesota IH-90 ramp closure gate

2.1.3 *Wyoming*

In Wyoming, gates are being used for road closure under severe weather conditions. Automatic gates have been installed at one location and the gates in all the other locations are in the process of being automated. The automatic gates being used are retrofit of the existing manually operated gates. The manually operating gates have been FHWA approved and crash tested according to the NCHRP 350 specifications (Mak et. al., 1996). A number of remote control gates are currently being tested.

2.1.4 *Other Relevant Studies*

The other states actively considering the usage of automated gates are Nebraska and South Dakota. Nebraska is currently studying the possibility of automating the gates in an integrated ITS framework. The primary usage of the gates is for road closure during severe weather conditions. The gates developed by Wyoming are being considered for usage. South Dakota has also conducted a technical study with a view of automating all manual gates used for road closure under severe weather conditions

3 RESEARCH FRAMEWORK AND APPROACH

In addressing the above objectives, the research approach proposed by TxDOT reflects several important considerations that focus on characterizing the feasibility of ramp closure, and developing an integrated traffic management plan in conjunction with ramp closure, in order to maximize the benefit of ramp closure, while minimizing or mitigating potential impacts on the network.

Figure 3-1 describes the general framework for conducting the feasibility study and implementation of a peak-period ramp closure. First, an application and an entrance ramp that is considered the candidate location for applying ramp closure are identified. The pattern and intensity of traffic congestion must be characterized according to defined performance indicators. Basic qualification procedures are then applied to examine if the ramp satisfies basic requirements. The qualification criteria will be primarily the freeway geometric features such as ramp spacing, main-lane and ramp traffic volumes, ramp storage, availability of alternate routes, etc.

Next, the feasibility of ramp metering should be studied. According to the FHWA guidelines, recommendation is made to consider the ramp metering strategy prior to adopting permanent or temporary closure of a ramp. If ramp metering is found to be desirable, then ramp metering is recommended, otherwise, benefit and impact analysis for a ramp closure option is recommended.

In the ramp closure benefit and impact analysis procedure, four categories of criteria including "Freeway Level-of-Service Analysis," "Regional Surface Traffic Impacts," "Level of Closure Information Provision", and "Safety Impact (freeways and arterials)" will be used to evaluate the feasibility and operational characteristics of ramp closure. Moreover, four categories of operational strategies will be evaluated based on the above criteria to determine the optimal configuration of the integrated operational strategies in conjunction with ramp closure. The four categories of operational strategies are "Closure Time and Duration", "Closure Information Provision Strategies", "ITS Strategies", and "Freeways/Arterials Control Integration". It should be noted that proper integration of possible network operational strategies provides the crucial opportunity needed to make ramp closure work. Simply executing the ramp closure without implementing a package of comprehensive and integrated traffic management strategies will reduce the likelihood of making the ramp closure a successful freeway operation strategy.

The above analyses are conducted in a laboratory environment using analytical/simulation models. If ramp closure is found to be a feasible and desirable option, the analysis results will be submitted to the FHWA for considering the approval of the field-testing project.

A set of traffic operation strategies in conjunction with the ramp closure have been identified. To prepare the ramp closure for deployment, two tasks need to be undertaken. One is the design of gate operation scheme, and the other is the traffic control plan for evaluating and improving the ramp closure practice after the closure is in place. The former issue concerns the gate operation, whether the closure is implemented with automated or manual approach. If automated gate operation is selected, which types of gate should be considered, and what are the pros and cons of using different types of gate. Criteria used for this consideration should include the crashworthiness of the gate, installation, operations and maintenance costs, as well as driver safety.

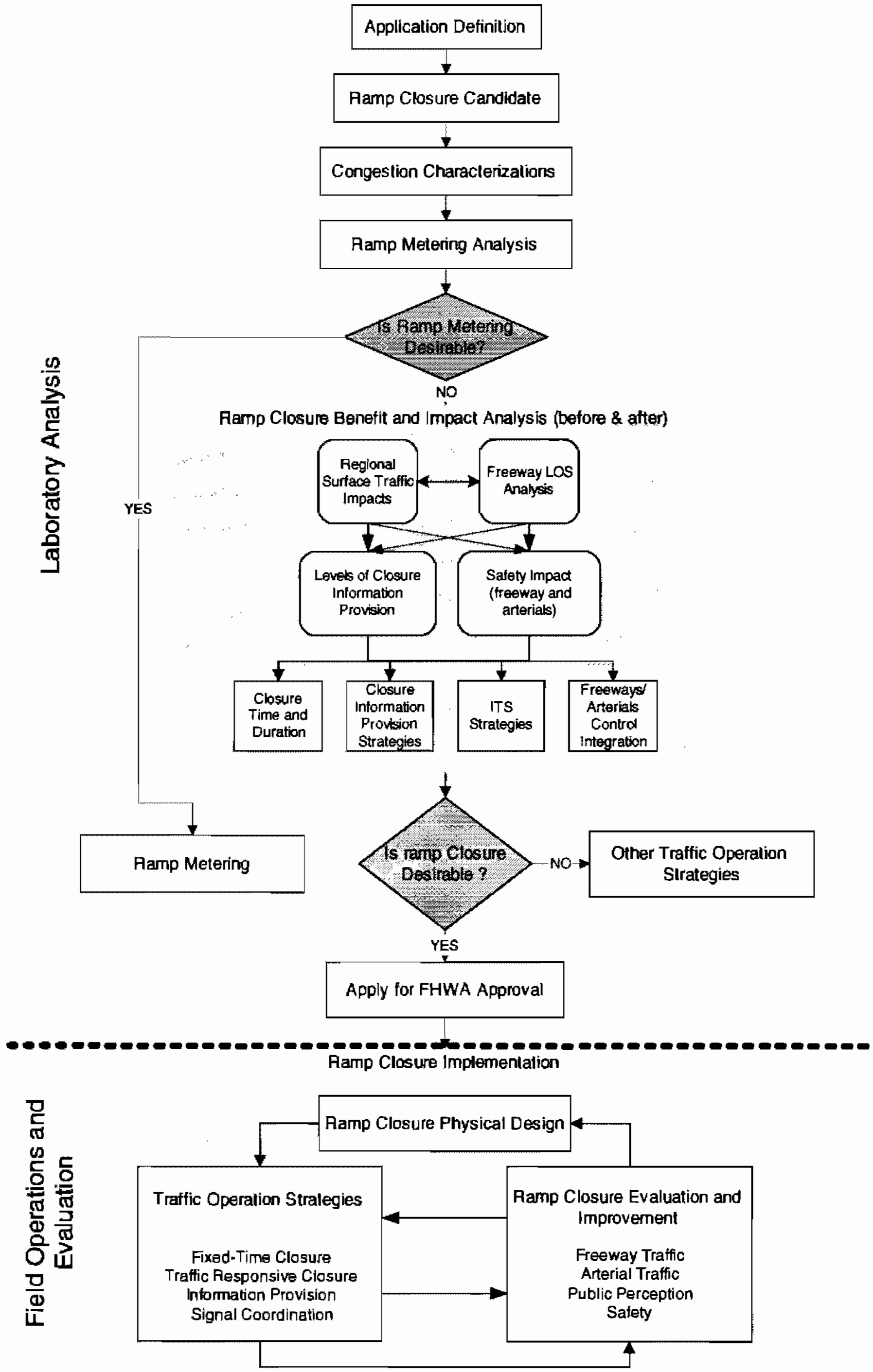


Figure 3-1 Research Framework

The before-and-after evaluation of ramp closure helps engineers identify issues and improve deployment and operation of ramp closure. The evaluation plan presented in this report encompasses a set of performance indicators to be included in evaluation, procedures for before-and-after data collections, and recommendations for interpreting evaluation results. Here, the research preliminarily defines four types of performance indicators for this purpose. They are "Freeway Traffic Impacts," "Arterial Traffic Impacts", "Public Perception," and "Safety Impacts". Prevedouros (1999) conducted a two-week experiment of ramp closure, and reported that drivers were generally surprised about the closure. The freeway performance did not reach expectation even after trying a variety of control device configurations. There is an important and well-documented phenomenon in that motorists constantly adjust driving behavior for a period in response to traffic operational strategies (Srinivasan and Mahmassani 2000; Srinivasan, 2000). Any traffic patterns observed before the equilibrium of driver-system interaction is reached may not be well representative of the true impacts of the closure strategy. In conducting the evaluation of peak-period ramp closure one should consider such a behavior equilibration process, and collect data over a sufficient period. Six to eight weeks of continuous monitoring and evaluation is recommended.

4 PEAK-PERIOD RAMP CLOSURE CASE STUDIES

Traffic impact studies require an in-depth analysis of traffic demand and basic traffic behavior. Choosing the right traffic model to simulate real-time traffic is one of the main responsibilities in research studies. Thus, it becomes a challenge for a traffic engineer to obtain sufficient information to foresee future problems in any physical change of the current infrastructure system. Computer animated traffic simulations present the opportunity to capture restricted traffic impact. Traffic simulation is a conventional approach to control traffic; it has existed for decades and includes strategies such as fixed timing plans, actuated signal control, and semi-actuated control. However, computer simulations are not capable of responding effectively to short-term changes in traffic demand. Thus, maintenance of such systems is resource intensive, i.e. there will always be future innovations. The FHWA along with many researchers from academia and the private sector have directed efforts to improve computer traffic simulations in order to rely on simulation output prior to any plan implementation.

The FHWA's Traffic Software Integrated System (TSIS) is now used across the nation as it has evolved into a sophisticated toolkit throughout the years. TSIS is comprised of CORSIM, TRAFVU, TSHEL and TRAFED, each with different software capabilities to simulate a microscopic traffic simulation, process animated output, and graphic input editors on both highways and arterials, respectively. CORSIM may be used for Ramp Closure Strategy and it is a good example of how current conditions must be replaced by updated traffic demand on our roads.

This project furthers decision-making utilizing a complex microscopic simulation using TSIS. CORSIM model was built in order to model the impact of peak-period ramp closure at the existing case studies. Moreover, route choices were determined from existing infrastructure in the model in order to create a path-based simulation. TSIS 5.0 allows users to input traffic generated routes. This new feature enables the capability to produce traffic that have a propensity to take a single route instead of the commonality random turns in previous versions or other simulation tools. Figure 4-2 illustrates the TRAFVU file in TSIS. A close-up for the Paisano on-ramp is provided.

4.1 El Paso, Texas Case Study

4.1.1 Congestion Characterization

El Paso, situated at the west-most tip of Texas (Figure 4-1), was considered to be a potential representative location for the assessment of implementing a peak-period ramp closure due to unique traffic congestion issues on the Interstate Highway 10 (IH-10) in the vicinity of IH-10/US-54 interchange – also known as the "Spaghetti Bowl" area (Figure 4-2). This segment of IH-10 has long been observed to have both high traffic volumes, including high truck traffic, as well as high weaving intensity because many trucks use the US-54 southbound exit ramp to reach the Mexico-bound commercial Port-of-Entry (POE) – Bridge of the Americas (BOTA). As shown in **Figure 4-3**, three on-ramps (Geronimo, Trowbridge, and Paisano) and three off-ramps (Paisano, Reynolds, and US-54 S/N) are placed within the 6,200 foot-long freeway segment. The spacing between any two-ramp junctions is less than 2,000 ft. The shortest spacing lies between the Paisano on-ramp and Reynolds off-ramp with merely 731 ft, as denoted as Segment 4 in **Figure 4-3**. No auxiliary lane is placed in the Segment 4, therefore, the Segment 4 is defined as the most restricted Type "A" weaving segment by the Highway Capacity Manual (HCM). Most Mexico-bound trucks prepare for exit by merging into the outer most lanes starting as early as in Segment 4, combined with high main-lane and Paisano on-ramp volume, making the study site one of the most congested segments along the IH-10 corridor in El Paso. Traffic accident data collected from 2000-2003 confirms high accident occurrence rate at the study site (see **Figure 4-4**, milepost 23).

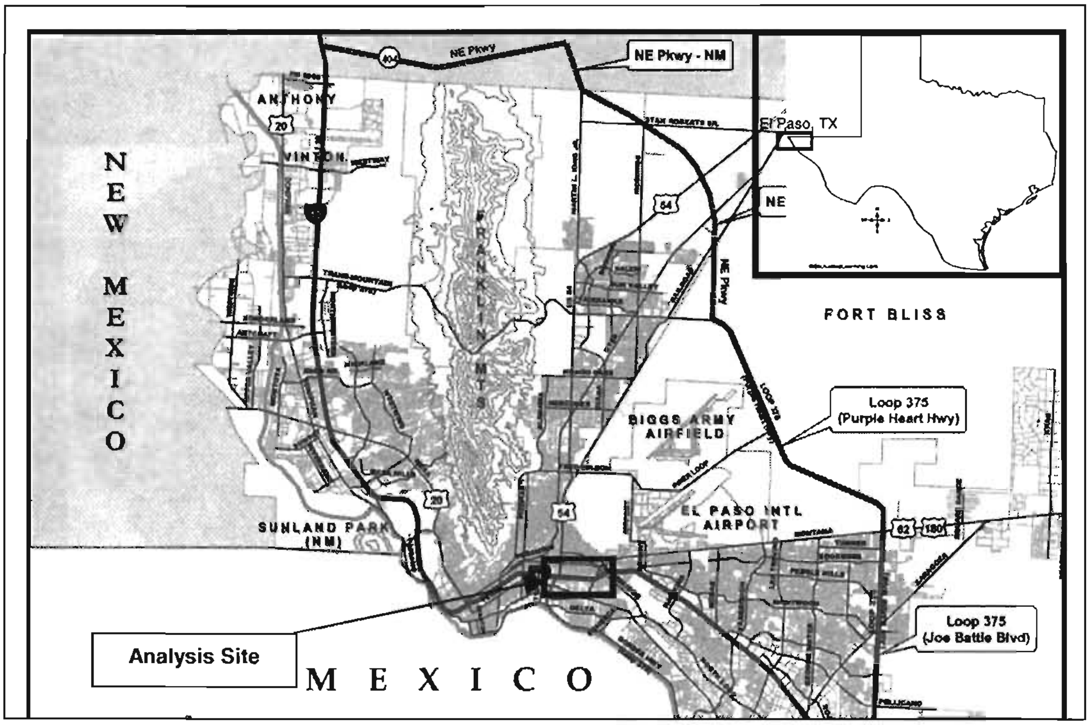


Figure 4-1 Peak-hour ramp closure analysis site (El Paso MPO, TxDOT, 2004)

Paisano on-ramp was once temporarily closed due to a construction project from June 1999 to January 2000. During the ramp closure period, the traffic condition in the study site (IH-10 main-lane Segment 1 to 5) was observed to have significantly improved. This observation motivates the further investigation of possible ramp controls to improve IH-10 traffic operations.

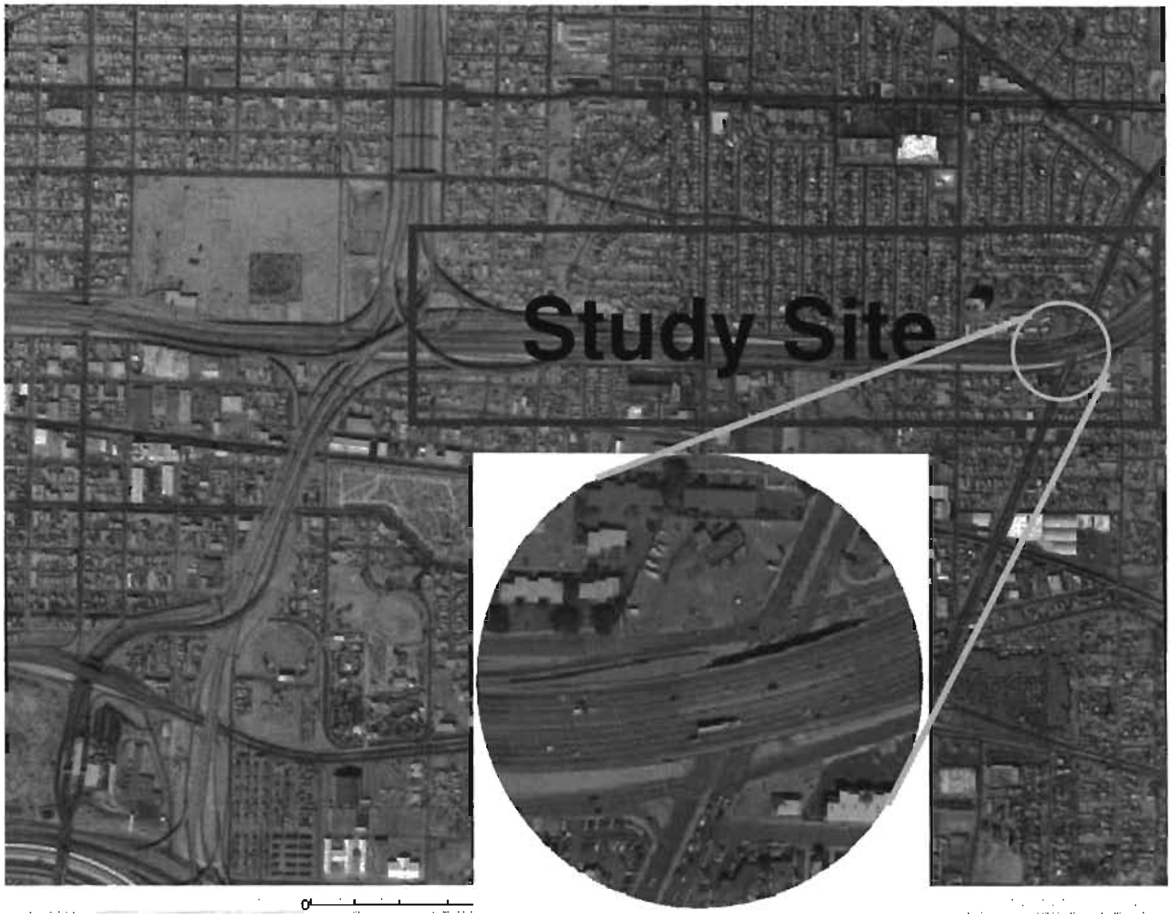


Figure 4-2 Analysis site and candidate ramp closure site – Paisano on-ramp

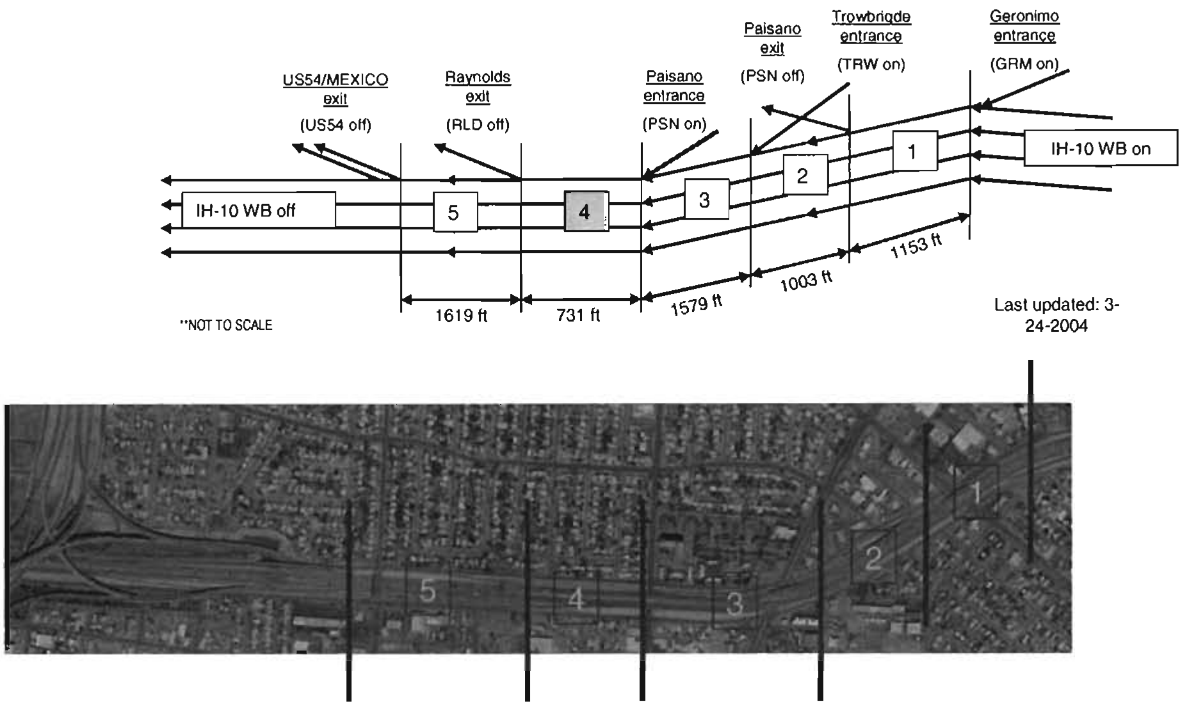


Figure 4-3 Case study site geometry and analysis representation

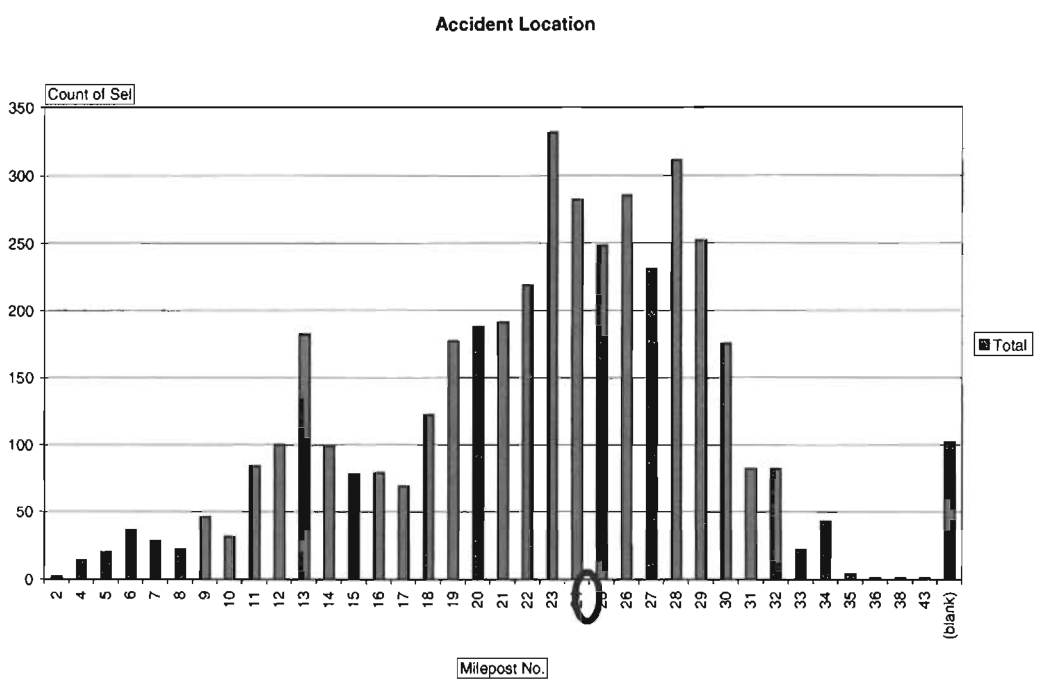


Figure 4-4 Accident rate on the project site (2000-2003, TxDOT)

4.1.2 Data Collection

4.1.2.1 Traffic Data

The first task before the traffic analysis is to identify the study site. The identified study site, as illustrated in Figure 4-2 and **Figure 4-3** reflects the deliberate considerations that once the Paisano on-ramp is closed during peak-hour, traffic that used to enter IH-10 through the Paisano ramp would be redirected to other downstream or upstream ramps. Such a flow redistribution may introduce various degrees of impact in other adjacent ramp/weaving segments. Therefore, incorporating all the six on/off-ramps near the Paisano ramp was deemed necessary from a modeling standpoint. The study site consists of the following: H-10 Westbound Traffic, Reynolds off-ramp, Geronimo on-ramp, Paisano off-ramp, Trowbridge on-ramp, US-54/Mexico off-ramp, and Paisano on-ramp.

Real-time traffic data was unavailable from the TransVista, El Paso TMC due to maintenance at the time of data collection. Therefore, efforts have been placed to identify proper videotaping locations within the study site and to record traffic video clips. Manual traffic count reading from these video clips was conducted. Video recordings took place on: February 18, 2004 from 7:00 am to 9:00 am and February 19, 2004 from 4:00 pm to 6:00 pm .

The study site is an eight-lane highway, four lanes in each direction. **Table 4-1** shows the traffic volumes at respective segments and ramps in the study site as previously defined. During morning peak hour 7:00 to 8:00 AM, the IH-10 main-lanes exhibit about 6,200 – 7,700 Passenger-Car Equivalent (PCE) (Segment 4) per hour (pcph). The equivalent flow rates are about 1,550 - 1925 PCE per hour per lane (pcphpl). Note that the highest flow rate was observed in Segment 4. The figures for 8:00-9:00 AM are about 6,100 – 7,200 pcph, equivalent to 1,525 – 1,800 pcphpl. The afternoon peak hours appear to be more congested than the morning peak hours. The flow-rate from 4:00-5:00 PM is between 1,250 – 1,925 pcphpl among defined segments. The 5:00-6:00 PM hour exhibits 1,375 – 2050 pcphpl.

It is clear that Segment 4 – immediate downstream of Paisano on-ramp is at capacity. The Paisano on-ramp constantly feeds about 500 pcph into IH-10. Comparing all the off-ramp volumes indicates that the US54 off-ramp experiences significantly high volumes in the afternoon peak hours, increased by three folds from about 850 pcphpl to about 2,400 pcphpl. This result clearly indicates that not only the Segment 4 has reached capacity, but also sustained high weaving intensity since many vehicles are getting off IH-10 and merging into the US54 off-ramp.

If the Paisano on-ramp traffic is blocked and redistributed to other ramps during peak hours, not only the inflow reduction will make the Segment 4 traffic volume well below capacity, but also turn the Segment 4 from a weaving segment to a ramp junction, which may significantly improve the operational speed and density in that segment.

Table 4-1- Current Traffic Demand - Traffic flow in pcph.

AM Traffic Flow - 7:00am

LOCATION	VEH	TRUCKS		PCE
IH-10 WB on	6135	226	3%	6700
GRM on	698	7	1%	716
1	6833	233	3%	7416
PSN off	464	10	2%	489
2	6369	223	3%	6927
TRW on	336	8	2%	356
3	6705	231	3%	7283
PSN on	366	20	5%	416
4	7071	251	3%	7699
RLD off	502	2	0%	507
5	6569	249	3%	7192
US54 off	962	14	1%	997
IH-10 WB off	5607	235	4%	6195

PM Traffic Flow - 4:00pm

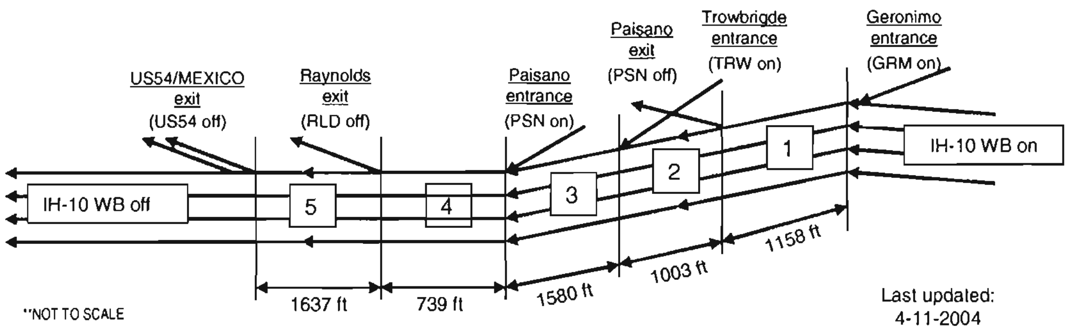
LOCATION	VEH	TRUCKS		PCE
IH-10 WB on	5620	295	5%	6358
GRM on	795	3	0%	803
1	6415	298	4%	7160
PSN off	498	16	3%	538
2	5917	282	4%	6622
TRW on	501	15	3%	539
3	6418	297	4%	7161
PSN on	492	22	4%	547
4	6910	319	4%	7708
RLD off	266	7	2%	284
5	6644	312	4%	7424
US54 off	2170	69	3%	2343
IH-10 WB off	4474	243	5%	5082

AM Traffic Flow - 8:00am

LOCATION	VEH	TRUCKS		PCE
IH-10 WB on	5401	287	5%	6119
GRM on	510	10	2%	535
1	5911	297	4%	6654
PSN off	430	6	1%	445
2	5481	291	5%	6209
TRW on	440	22	4%	495
3	5921	313	5%	6704
PSN on	462	33	6%	545
4	6383	346	5%	7248
RLD off	277	18	6%	322
5	6106	328	5%	6926
US54 off	725	30	4%	800
IH-10 WB off	5381	298	5%	6126

PM Traffic Flow - 5:00pm

LOCATION	VEH	TRUCKS		PCE
IH-10 WB on	6073	286	4%	6788
GRM on	784	1	0%	787
1	6857	287	4%	7575
PSN off	474	7	1%	492
2	6383	280	4%	7083
TRW on	481	17	3%	524
3	6864	297	4%	7607
PSN on	552	18	3%	597
4	7416	315	4%	8204
RLD off	199	11	5%	227
5	7217	304	4%	7977
US54 off	2299	72	3%	2479
IH-10 WB off	4918	232	4%	5498



		Mainlane	GRM on	1	PSN off	2	TRW on	3	PSN on	4	RLD off	5	US54 off	Mainlane
AM Peak-Hour Traffic 7:15 - 8:15	VEH	6289	709	6996	519	6479	371	6850	419	7269	447	6822	893	5929
	TRUCKS	247	8	255	6	249	11	260	26	286	2	284	17	267
	% Trucks	6%	2%	5%	2%	5%	4%	5%	9%	6%	1%	6%	3%	6%
	TOTAL	6536	717	7253	525	6728	382	7110	445	7555	449	7106	910	6196
	PHF	0.91	0.84		0.77		0.75		0.86		0.82		0.87	

		Mainlane	GRM on	1	PSN off	2	TRW on	3	PSN on	4	RLD off	5	US54 off	Mainlane
PM Peak-Hour Traffic 4:45 - 5:45	VEH	6077	826	6903	487	6416	512	6928	556	7484	242	7242	2268	4974
	TRUCKS	286	4	290	10	280	22	302	14	316	13	303	63	240
	% Trucks	7%	1%	6%	3%	6%	6%	6%	4%	6%	7%	6%	4%	7%
	TOTAL	6363	830	7193	497	6696	534	7230	570	7800	255	7545	2331	5214
	PHF	0.96	0.88		0.87		0.88		0.87		0.72		0.98	

Figure 4-5 Study site peak-hour traffic data summary

4.1.3 *Highway Geometric Data*

The Level of Service (LOS) of each respective segment has been further studied by both HCM and simulation approaches, and the results are further discussed in the following sections. HCM approach requires first to partition the entire study site into segments to be one of the three types of highway elements – basic segment, ramp junction, or weaving segment. Each segment's LOS will be analyzed based on different speed and flow-rate prediction methods. The LOS is usually defined based on the density in the segment predicted by the models.

4.2 Freeway Traffic Improvement Analysis (Existing Condition) - Highway Capacity Analysis (HCM)

Careful investigation of geometric layouts of ramps within the study site led to five segments as shown in **Table 4-5**. The beginning and the end of each segment are either an on-ramp, an off-ramp or both. As such, Segments 1 and 4 were defined as weaving segments, noted is that the two segments are shorter than 2,500 ft in length and no auxiliary lanes exist. As such, they were defined as weaving segments instead of ramp junctions. Segments 3 and 5 were defined as ramp junctions and Segment 2 was defined as a basic segment.

After obtaining geometric and traffic volume data, the HCM LOS analysis was conducted using the HCS2000 software. The LOS analysis was preformed not only on the existing situation but also on several ramp closure induced flow distribution scenarios. The HCM analysis results are documented in this section, and the results of two other flow distribution scenarios are presented in the next section.

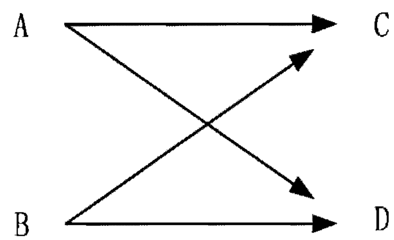


Figure 4-6 Schematic of the flow distribution in a weaving segment

As previously discussed, the segments 1 and 4 in the study site were defined to be freeway-weaving segments. Figure 4-7 illustrates the HCM methodology in obtaining the LOS of a freeway-weaving segment. In the current situation, the geometric data includes the length of the weaving segment (measured from the gores of the on and off-ramps), the weaving type (both are Type A weaving), the terrain type (study site is the level terrain) and the number of lanes (4 lanes). The free flow speed is set to be 65 mph. Another important input is the traffic volumes for weaving and non-weaving traffic. Before no weaving traffic was collected in the data collection tasks, one has to infer the weaving/non-weaving traffic volumes for each movement direction shown in Figure 4-6 and their corresponding peak hour factors, their percentage of trucks and buses as well as recreational vehicles. It is assumed that the driver population factor in all our case is equal to one. Estimating the unknown weaving/non-weaving volumes (V_{AD} , V_{AC} , V_{BC} , V_{BD}) start from the known flows $V_A = V_{AC} + V_{AD}$,

$V_B = V_{BC} + V_{BD}$, $V_D = V_{AD} + V_{BD}$, $V_D = V_{AC} + V_{BC}$. By solving the four equations, the four unknown flows can be computed.

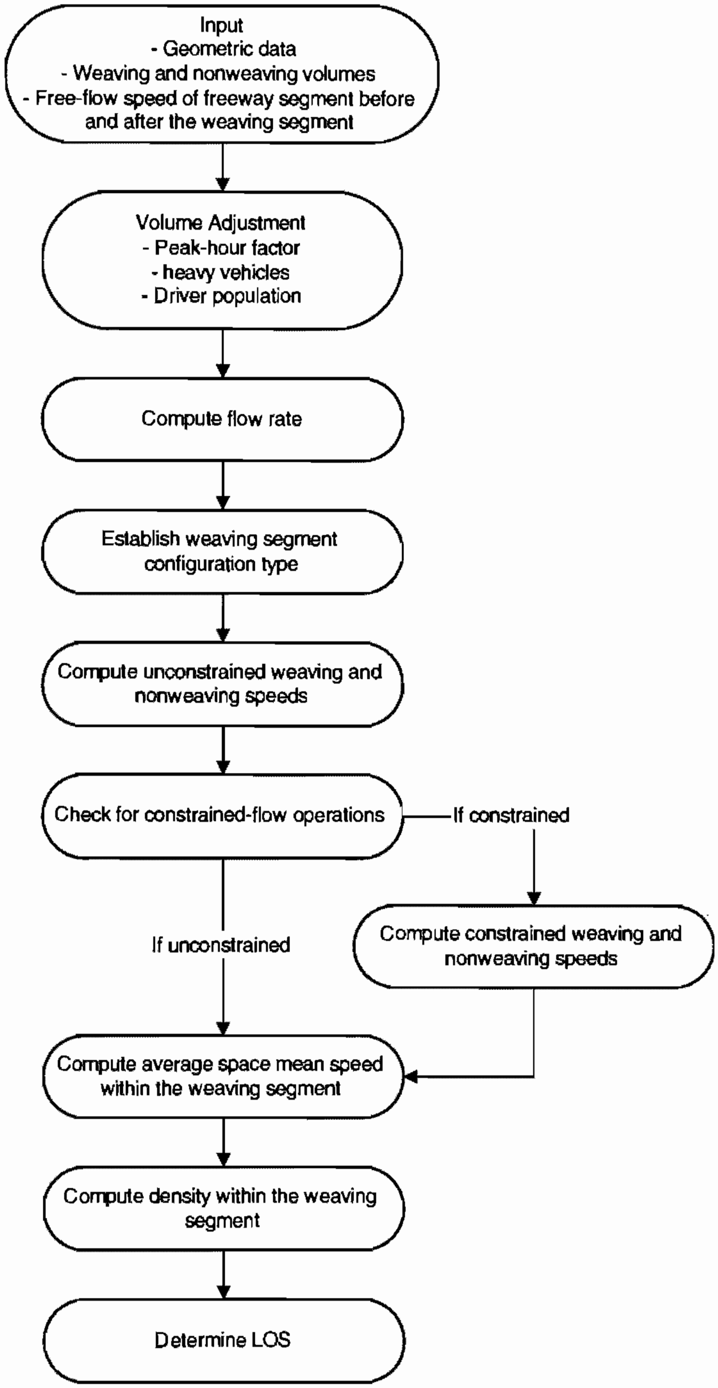


Figure 4-7 Freeway-weaving segment methodology (HCM 2000)

The LOS results for each segment of the study site are illustrated in Figure 4-8.

The HCM analysis reveals that the Segment 4 is of LOS F, which means this segment of highway is operating at undesirable traffic condition. The Segment 1, another weaving segment from Geronimo to Trowbridge is perating at LOS E. The rest of the segments are of LOS C or D. This result is consistent with the observed traffic data in that the Segment 4 not only has the highest traffic volumes among the five segments because of the three consecutive on-ramps in the upstream, but also pertains to the most restricted weaving type—Type A. Furthermore, the length of Segment 4 is only 731 feet, which intensifies the conflicts between interweaving vehicles due to limited weaving length.

Segment	5	4	3	2	1
Avg. speed	----	44.89	----	66.7	46.85
Weaving speed Sw	----	36.25	----	----	40.21
Non-weaving speed Snw	----	46.14	----	----	48.56
LOS	**D**	**F**	**C**	**D**	**E**

Figure 4-8 LOS of all the segments in the current situation

4.3 Freeway Traffic Improvement Analysis (Ramp Closure Scenarios) - Highway Capacity Analysis (HCM)

One of the main objectives in this research is to demonstrate the effectiveness of the peak-hour ramp closure strategy. In the case study, based on the current traffic situation analysis, it is intuitive to choose to close the Paisano on-ramp to alleviate the traffic congestion. If the Paisano on-ramp is closed during peak hours, those who usually enter IH-10 via the Paisano on-ramp will have to choose other ramps or even local streets. Whether the flow re-distribution causes other bottleneck at other locations or not is of particular concern. Intuitively, short after the ramp closure is implemented, commuters will continue with their original routes until seeing signs at proximity of the ramp. The detour hence occurs near the Paisano on-ramp. In this case, commuter will utilize more of the IH-10 frontage roads to enter the IH-10 in the next on-ramp, or use Paisano south bound if the destination is the downtown area. However, in the end, most commuters will adapt to the ramp closure and may choose to take other on-ramps or even an entirely different route. Such an adaptation process is usually termed "equilibrium." The equilibrium process could take from several weeks to months to complete. In other words, continuously monitoring the traffic pattern resulted from the closure is the key to draw objective conclusions on the benefit and impact of the closure strategy.

To capture the flow redistribution due to ramp closure, one needs a good dynamic traffic assignment model. DYNASMART-P (Mahmassani, et al. 2001) has been proven a promising tool for this purpose. Due to time and resource requirements for establishing and calibrating such a model, and limited research time frame available for this project, it was decided that a simpler method (HCM + microscopic simulation) be applied to approximate possible flow re-distribution and resulting highway LOS. In the applied HCM method, engineering judgment has been employed to distribute Paisano traffic to adjacent on-ramps. In the case study, two possible scenarios were examined. In Scenario 1, the Paisano on-ramp is closed and the volume is evenly distributed on the Trowbridge on-ramp and Geronimo on-ramp. In Scenario 2, the Paisano on-ramp is closed and the volume is evenly distributed on the Reynolds on-ramp and Trowbridge on-ramp. Note that only two scenarios were examined using the HCM approach. A wider array of flow distribution scenarios were proposed and examined using the simulation-based approach. These results are presented in section 4.4.

4.3.1 *Scenario 1*

In this scenario, the Segment 1 is a freeway-weaving segment and the Segment 2 is a basic freeway segment. The traffic volumes in the two segments will be higher than the current situation on the account that half of the redistributed traffic volume is going to be distributed to these two segments.

Compared to the current situation, the traffic condition in the Segment 4 in the Scenario 1 has been significantly improved from LOS F to D, which is attributed to the closure of the Paisano on-ramp. However, it is also found that the LOS in the Segment 1 has been worsened from LOS E to F under the assumed flow distribution scenario.

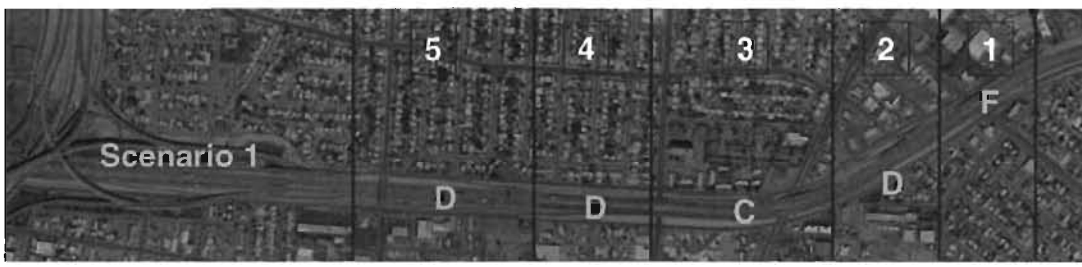


Figure 4-9 The LOS of all the segments in Scenario 1

4.3.2 *Scenario 2*

In Scenario 2, the Paisano on-ramp is closed and the volume is evenly distributed to the Reynolds on-ramp and Trowbridge on-ramp. The major difference between the Scenario 2 and the Scenario 1 is that half of the redistributed volume will get into IH-10 at the downstream of Paisano on-ramp.

The overall LOS for the study site is shown in Figure 4-10. From this figure, one can see that the closure of the Paisano on-ramp has improved the LOS in Segment 4 without worsening the LOS at other segments.

Figure 4-10 The LOS of all the segments in Scenario 2

In summary, HCM LOS analysis has drawn findings and conclusions as follows (see **Figure 4-11**):

1. The flow redistribution could affect the LOS in other upstream or downstream segments, primarily because the study site is operating at near-capacity condition.
2. The Scenario 2 represents a more plausible flow redistribution than Scenario 1 since those vehicles used to use the Paisano on-ramp are less likely to go all the way upstream to enter the freeway, particularly most of commuters are aware of the congested traffic conditions in the study sites. A less number of commuters may choose to enter IH-10 at upstream ramps and then traverse through the congested study site. It may be more reasonable to assume that a significant portion of the Paisano traffic will be distributed to the Paisano downstream ramp, such as the Reynolds on-ramp.
3. Comparing model outputs and observed data indicates that the HCM approach generally over-estimates speed for all highway segments, primarily the weaving segment. Because of congested traffic conditions and high access intensity, conducting the LOS analysis based on microscopic simulation becomes important.

As such, the microscopic simulation approach is further employed to carry out an extensive set of experiments. The next section highlights the process taken and resulted obtained from the microscopic traffic simulation approach.

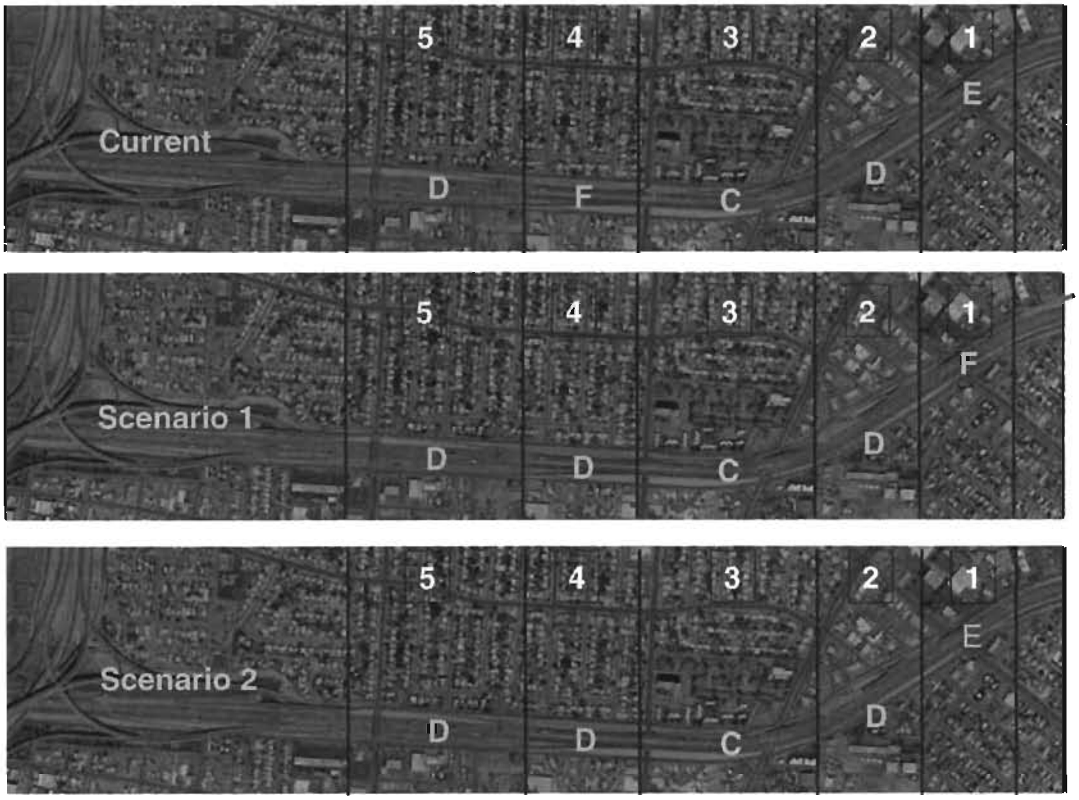


Figure 4-11 LOS analysis results for current and different ramp closure flow scenarios

4.4 Freeway Traffic Improvement Analysis (Ramp Closure Scenarios) - Simulation-Based Analysis (TSIS)

The simulated network in TSIS consists of a 2-mile eight-lane IH-10 freeway segment (**Figure 4-12** to **Figure 4-14**). It contains all ramps and main-lane segments. In order to obtain realistic simulation outputs, effort was placed to acquire key model geometric inputs. There inputs include grades along the study site, lengths of auxiliary lanes, distance between gores of ramps, and curvature of horizontal curves along the main lanes. The simulation time was specified to be the same as data collection periods. A vehicle equipped with a GPS unit was also used to obtain accurate longitude and latitude data in order to supplement the schematics and satellite image obtained from TxDOT. Some arterial segments were included in the simulation model but specific details were geared towards Paisano Dr. The network was set up using the TRAFEd front-end graphical process in TSIS, the coded network can be seen in **Figure 4-12**.

Figure 4-12 Coded study site in TRAFEd, TSIS

The network entry volumes were specified using the collected traffic volumes as shown in **Table 4-1**. The turning percentages at the off ramps are specified using the calculation done for the HCM analysis. The turn percentages at the Paisano Dr./IH-10 intersection were specified using the collected data, and the signal timing was specified using the timing plan provided by the City of El Paso Traffic Operation Department.

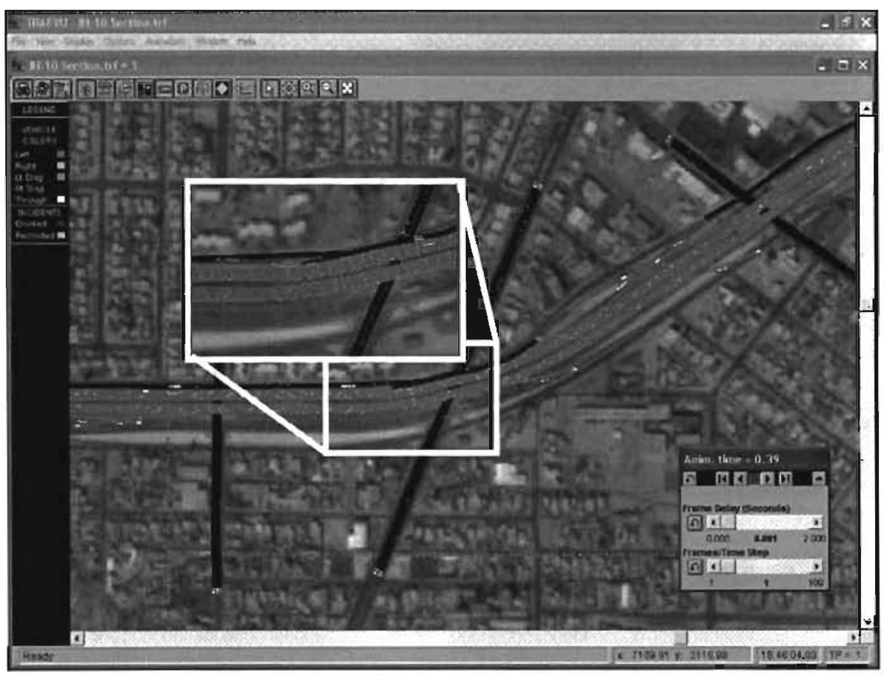


Figure 4-13 Traffic Software Integrated System Simulation (El Paso Case Study)

Figure 4-14 Snapshot of traffic simulation of the study site in CORSIM

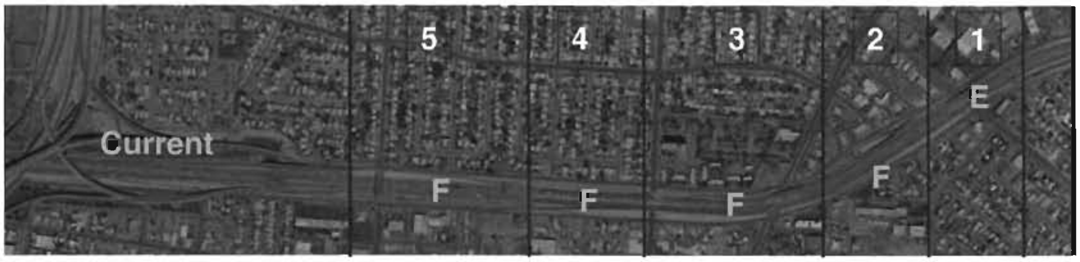
In the following sections, highway LOS analysis results are presented

4.4.1 Existing condition

Determining the LOS based on a simulation approach is more straightforward than the HCM approach in the sense that the LOS is determined by density and the density information can be directly found in the simulation outputs. **Table 4-2** summarizes the speed, density and LOS for each highway segment. The estimated speed for the Segment 4 is about 20 mph. The rest of the segments have speed ranging from 20-40 mph and flow ranging from 1600 to 1800 pcphpl. Note that due to dense ramp coverage, the flow rate is lower than the typical capacity at 2000 – 2200 pcphpl. The results are consistent with field observations.

Table 4-2 Study site current traffic LOS (CORSIM)

Segment	5	4	3	2	1
Speed (mph)	25.3	20.5	28.8	38.1	45.0
Volume (vphpl)	1800	1540	1750	1680	1557
Density (vpmpl)	54.6	56.9	61.6	44.2	36.3
LOS	**F**	**F**	**F**	**F**	**E**



4.4.2 *Pre-specified Flow Distribution*

The two Paisano flow distribution scenarios were also examined using simulation. The LOS results are summarized in **Figure 4-15**. The simulation results indicate that Segment 4 is current operating at LOS F. Closing the Paisano on-ramp will improve the LOS from F to E. Distributing Paisano flow to various different on ramps causes the change of LOS in these respective areas. For example, the LOS in Segment 2 varies between F and D. The LOS for Segment 1 varies between E and D. The results also indicate that the LOS in that study site could be sensitive to how the Paisano flows are re-distributed since the entire freeway segment is operating at capacity. Any change of inflow could cause direct change of LOS.

In order to capture realistically the possible actual LOS, Paisano flows are randomly distributed. Thirty randomized flow distributions were specified into thirty simulation runs. The details are discussed in the following section.

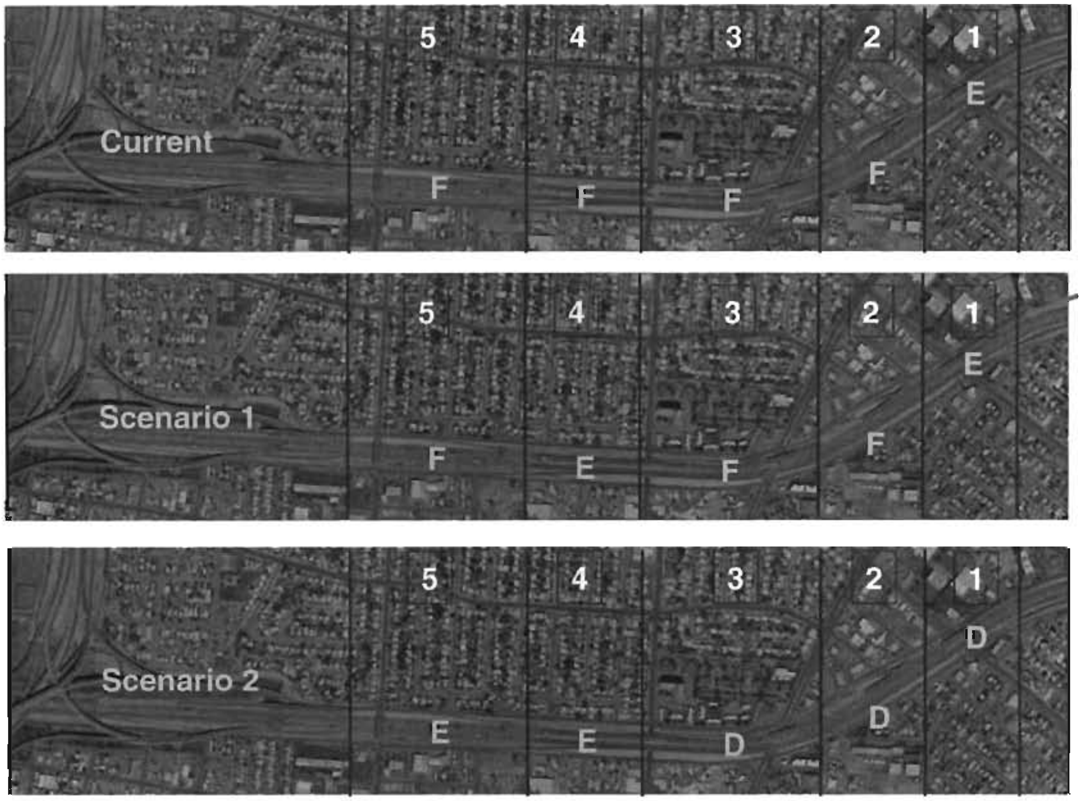


Figure 4-15 Study site LOS based on pre-specified flow distribution

4.4.3 *Random Flow Distribution*

Since actual flow distribution is unknown, this section presents a technique that randomly distributes the Paisano flow to various on/off ramps. Simulation runs are conducted for each flow distribution scenario. The next step is to examine the possible range of LOS for each segment across all the scenarios. If a segment maintains similar LOS across all the scenarios, then we have the confidence that this is the actual LOS in reality. This method introduces higher robustness in determining the actual LOS for each segment than the manual ad hoc flow assignment.

In the scenario design, Trowbridge, Geronimo and Reynolds are considered the only three ramps that receive significant diverted Paisano traffic. In the total 30 simulation runs, all the Paisano flows are randomly distributed to these ramps, adding to existing flows, become the new flows for these ramps (see Table 4-3).

Table 4-3 Access Ramp Random Demand Distribution

Scenario	Diverted Demand	Trowbridge			Geronimo			Raynolds		
		%	vph	demand	%	vph	demand	%	vph	demand
1		34	203	775	11	66	926	55	328	678
2		32	191	763	53	316	1176	15	89	439
3		18	107	679	24	143	1003	58	346	696
4		51	304	876	7	42	902	42	250	600
5		0	0	572	93	554	1414	7	42	392
6		17	101	673	82	489	1349	1	6	356
7		12	72	644	29	173	1033	59	352	702
8		12	72	644	7	42	902	81	483	833
9		20	119	691	58	346	1206	22	131	481
10		31	185	757	41	244	1104	28	167	517
11		76	453	1025	16	95	955	8	48	398
12		34	203	775	9	54	914	57	340	690
13		18	107	679	10	60	920	72	429	779
14		24	143	715	19	113	973	57	340	690
15	596	25	149	721	43	256	1116	32	191	541
16		47	280	852	22	131	991	31	185	535
17		63	375	947	32	191	1051	5	30	380
18		3	18	590	94	560	1420	3	18	368
19		28	167	739	44	262	1122	28	167	517
20		1	6	578	85	507	1367	14	83	433
21		31	185	757	57	340	1200	12	72	422
22		30	179	751	8	48	908	62	370	720
23		38	226	798	0	0	860	62	370	720
24		0	0	572	73	435	1295	27	161	511
25		3	18	590	60	358	1218	37	221	571
26		11	66	638	58	346	1206	31	185	535
27		21	125	697	77	459	1319	2	12	362
28		0	0	572	85	507	1367	15	89	439
29		74	441	1013	1	6	866	25	149	499
30		16	95	667	10	60	920	74	441	791

The speed, density and flow for each segment in each run were recorded. After all 30 runs were completed, aggregated statistics were computed and summarized in **Table 4-4, Figure 4-16** to **Figure 4-20**. From **Table 4-4**, one can find that the average LOS for each segment is D, E, E, D, and D in sequence of segment number. Also examining the LOS distribution figures, one can find that the LOS for segments 4 and 5 are almost invariably at D, indicating obvious LOS improvement in both segments. Segment 1 improves from E to D; Section 2 improves from F to E and Section 3 improves from F to E. Such a universal improvement can be attributed to the improvement of Section 4. Ramp closure induced higher speed and throughput directly help improve the traffic condition upstream, which in turn results in the improvement of traffic conditions in Sections 1 – 3.

Table 4-4 Average density and LOS for highway segment

	SECTION									
	1		2		3		4		5	
	K	LOS	K	LOS	K	LOS	K	LOS	K	LOS
MEAN	34.5	D	36.4	E	41.9	E	30.2	D	32.4	D
MEDIAN	33.7	D	35.7	E	42.4	E	30.0	D	32.6	D
MODE	32.0	D	33.3	D	39.0	E	32.2	D	32.9	D
MIN	26.9	D	31.0	D	34.4	D	26.7	D	30.4	D
MAX	45.7	F	52.3	F	50.8	F	36.4	E	34.9	D
VARIANCE	11.968		19.315		18.362		5.016		1.512	
ST. DEV.	3.460		4.395		4.285		2.240		1.230	

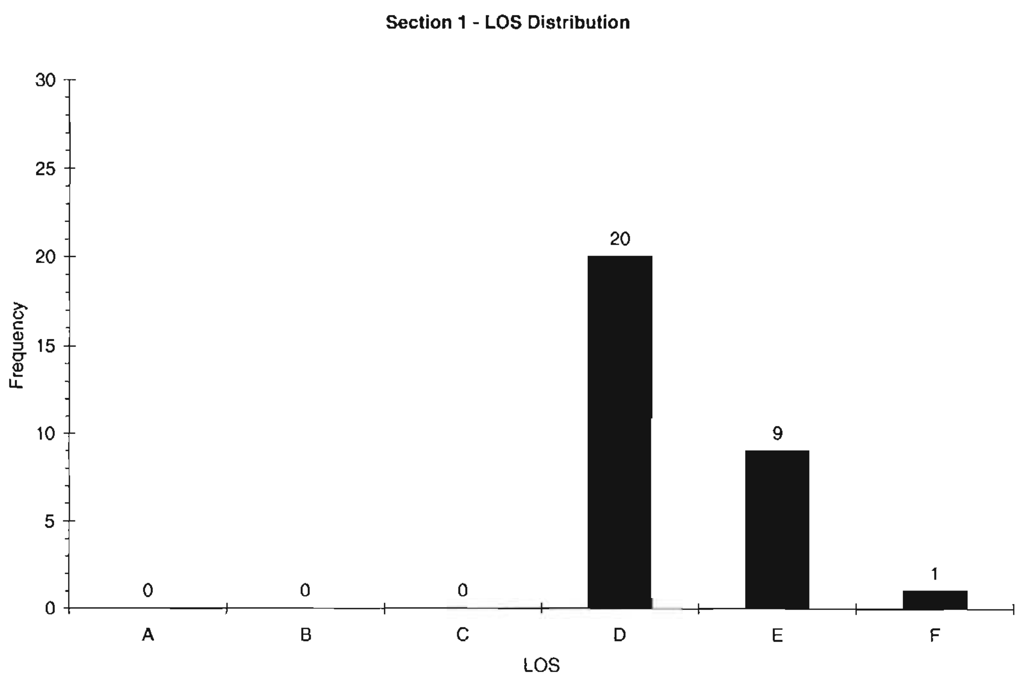


Figure 4-16 LOS distribution for Segment 1

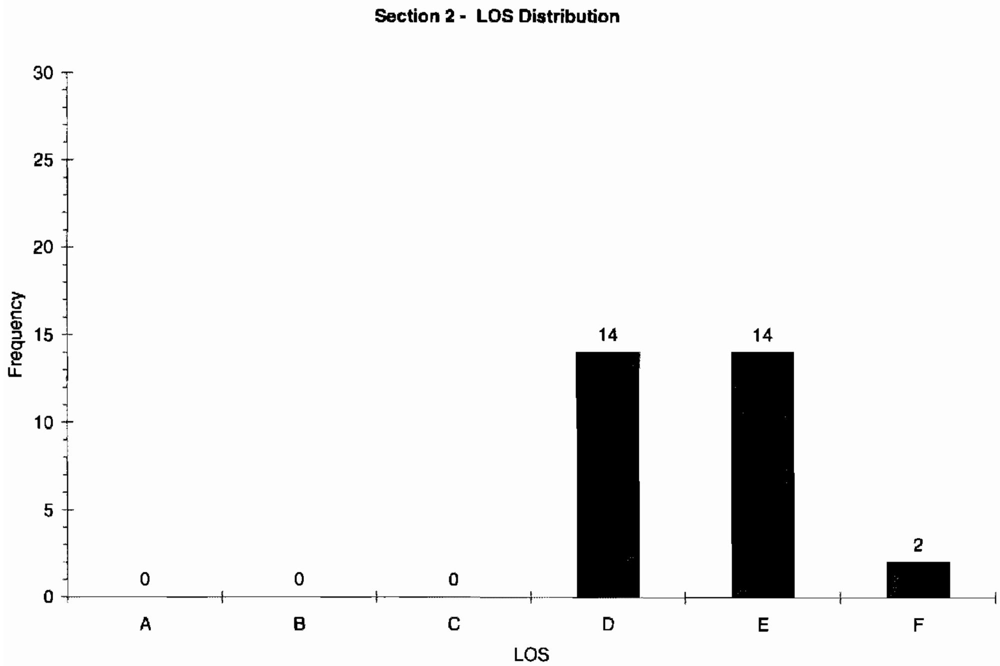


Figure 4-17 LOS distribution for Segment 2

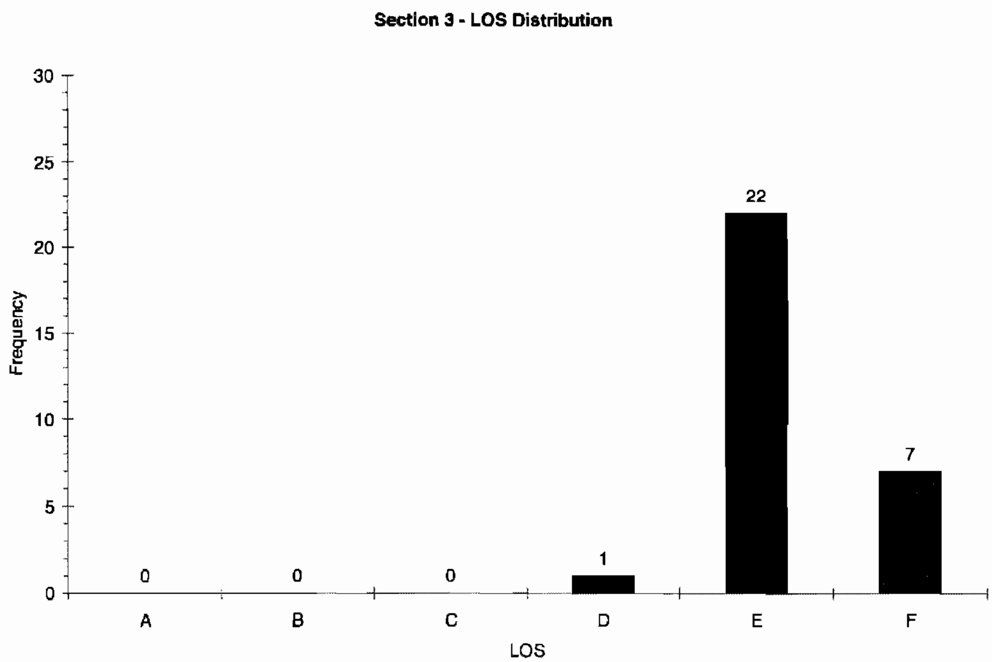


Figure 4-18 LOS distribution for Segment 3

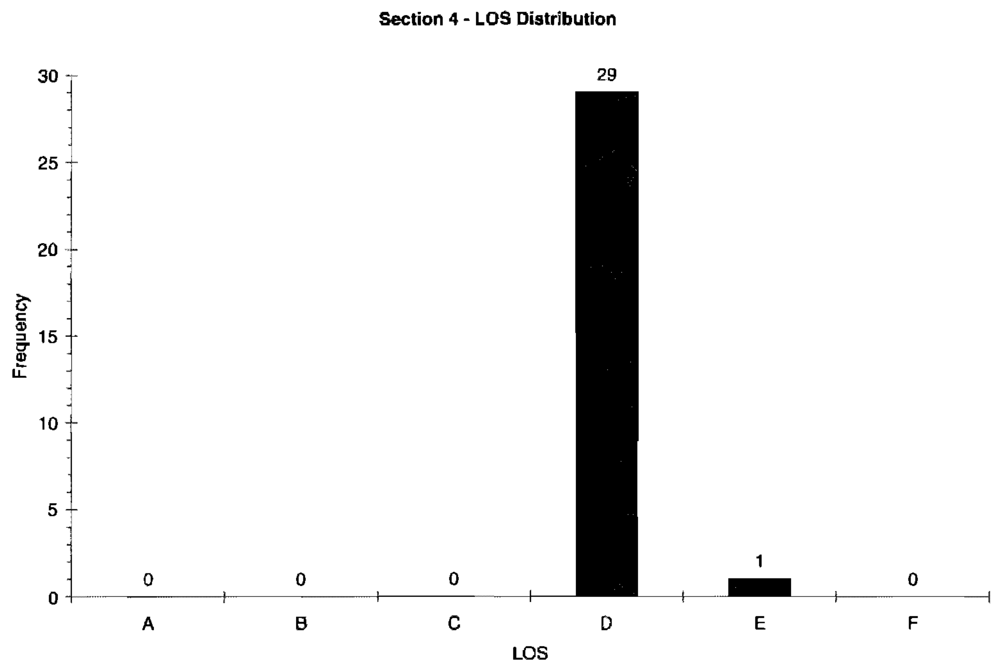


Figure 4-19 LOS distribution for Segment 4

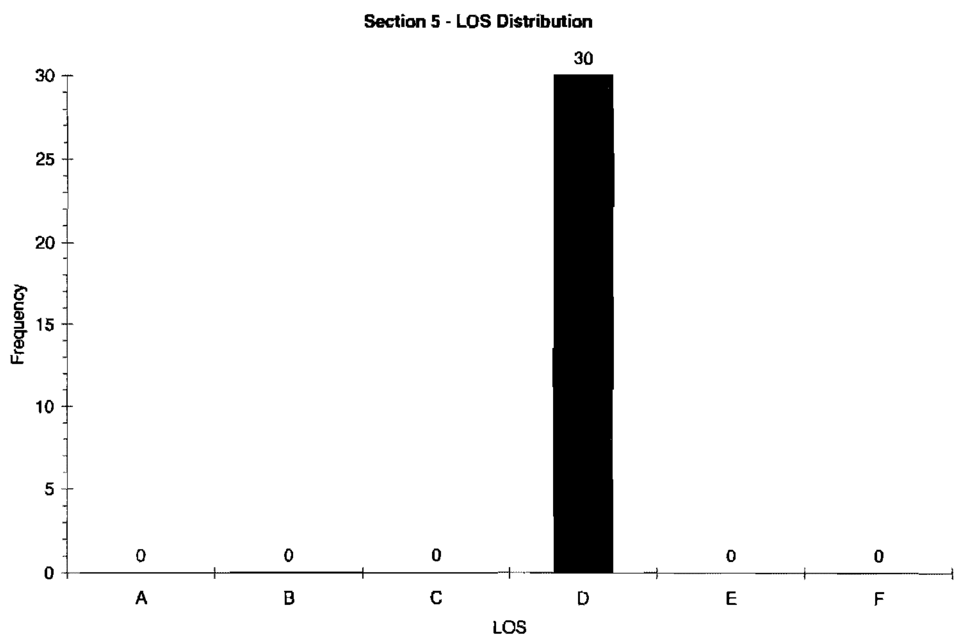


Figure 4-20 LOS distribution for Segment 5

4.5 Ramp Metering Feasibility Analysis

The feasibility of ramp metering was evaluated in the simulation environment. The specified ramp-metering scenario assumes the installation of an ARENA type of ramp metering device on the Paisano ramp, aiming to alleviate congestion on main lanes in the Segment 4. Simulation results showed that the LOS for Segment 4 would be improved to LOS E from F. However, severe queue spillback (about 1000 ft long) occurred for about 30 minutes at the Paisano/Gateway W intersections due to low metering rate resulted from high volume at the Segment 4.

The geometric characteristics that make the Paisano on-ramp not suitable for this type of control include (1) Proximity to arterial intersection. The Paisano/Gateway W. intersection is within 200 ft. (2) Number of ramp lanes. The Paisano ramp is a one-lane on-ramp. (3) Lack of storage. The length of Paisano on-ramp is less than 300 ft, which is considered relatively short for storing queue vehicles. (4) Lack of acceleration lane. There is no acceleration lane for the Paisano ramp. Vehicles have difficulties to speed up and merge into the main lanes once passing the ramp signal.

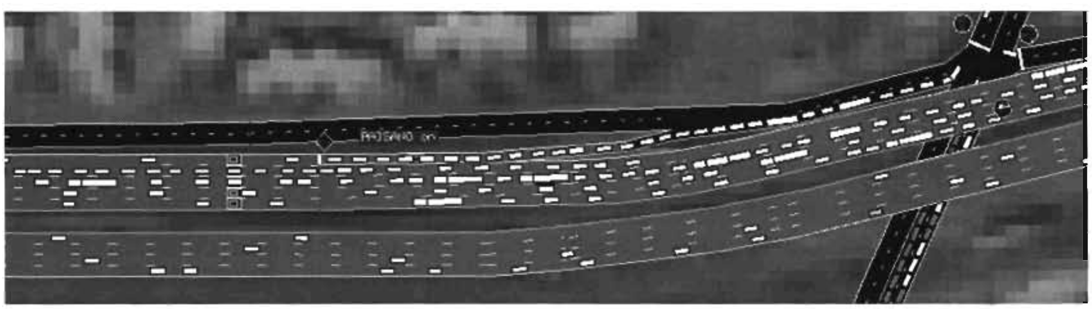


Figure 4-21 Ramp Metering Device on Ramp

Overall, the analysis has found that the ramp metering strategy would introduce marginal benefit to IH-10 main lane traffic at the expense of sever spillback and congestion at the nearby Paisano/Gateway W intersection.

4.6 Austin, Texas Case Study

In Austin, the freeway segment under consideration is the northbound section of the Interstate 35 from the Yager exit to the Wells Branch Parkway exit. A schematic representation of the network is shown in the figure. The northbound Interstate has been a highly congested section with a very low LOS, which has been complicated by drivers using the frontage road for freeway queue jumping during peak periods. The impact of closing the entrance ramp from Yager Lane to the northbound section considering the impact of queue jumping on IH 35 during peak period is studied.

4.6.1 Highway LOS Analysis

Two types of Level of Service analysis were conducted. The first method uses the procedures presented in the Highway Capacity Manual.

4.6.1.1 Highway Capacity Manual (HCM) Analysis

According to the HCM 2000, the freeway section being studied can be divided into 8 different segments as shown in the figure below.

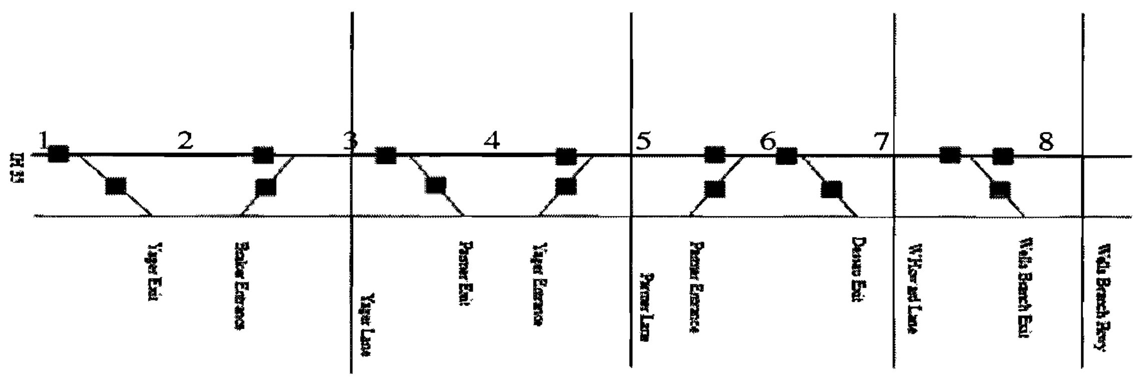


Figure 4-22 IH-35 Segment from Yager Exit to Wells Branch Exit

Table 4-5 IH 35 Segment – Segment Number

Section	Sec Number
Before Yager Exit	1
Yager Exit - Braker Entrance	2
Braker Entrance - Parmer Exit	3
Parmer Exit - Yager Entrance	4
Yager Entrance - Parmer Entrance	5
Parmer Entrance - Dessau Exit	6
Dessau Exit - Wells Branch Exit	7
After Wells Branch Exit	8

Level of Service analysis is carried on each of the different segments of the IH-35 network after the Yager entrance ramp is closed. The analysis for LOS on the freeway segment is carried on in accordance with the methodology given in HCM 2000.

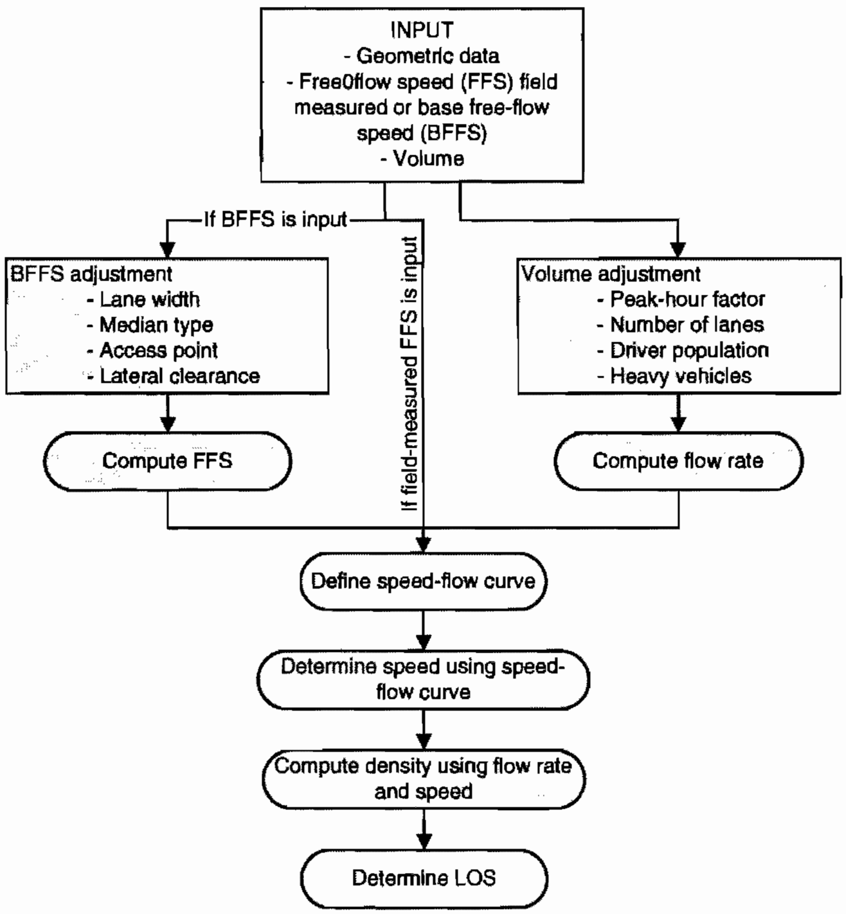


Figure 4-23 Methodology for LOS analysis for Basic Freeway Segment (HCM, 2000)

The traffic data for the analysis was obtained from a study conducted by TX DOT in 1999 investigating the performance of the I-35.

Table 4-6 Mainline count at Braker Lane Bridge

Time	Cars (15 min)	Trucks	Total (15 min)	Total Hourly	% Trucks
4:30 – 4:45	1309	67	1376	5502	4.87
4:45 – 5:00	1416	69	1485	5936	4.62
5:00 – 5:15	1472	43	1515	6060	2.84
5:15 – 5:30	1386	42	1428	5710	2.94
5:30 – 5:45	1326	50	1376	5502	3.64
5:45 – 6:00	1223	56	1279	5112	4.34
6:00 – 615	1215	49	1264	5056	3.88
6:15 – 6:30	1162	55	1217	4868	4.52

Table 4-7 Input Data at Braker Entrance

Time	Through (15 min)	Through (Hourly)	Braker Entrance (15 min)	Braker Entrance (Hourly)	Through (15 min)
4:30 – 4:45	1282	5128	135	538	1417
4:45 – 5:00	1329	5316	122	486	1451
5:00 – 5:15	1295	5178	100	400	1395
5:15 – 5:30	1115	4458	79	316	1194
5:30 – 5:45	1067	4266	45	178	1112
5:45 – 6:00	995	3980	49	196	1044
6:00 – 615	1025	4100	59	236	1084
6:15 – 6:30	1068	4270	64	254	1131

Table 4-8 Input Data at Parmer Exit

Time	Through (15 min)	Through (Hourly)	Parmer Exit (15 min)	Parmer Exit (Hourly)	Through (15 min)
4:30 – 4:45	1417	5666	130	520	1287
4:45 – 5:00	1451	5802	136	542	1315
5:00 – 5:15	1395	5578	137	546	1258
5:15 – 5:30	1194	4776	155	618	1039
5:30 – 5:45	1112	4444	162	648	949
5:45 – 6:00	1044	4176	153	610	892
6:00 – 615	1084	4336	148	590	937
6:15 – 6:30	1131	4524	149	594	983

Table 4-9 Input Data at Yager Entrance

Time	Through (15 min)	Through (Hourly)	Yager Entrance (15 min)	Yager Entrance (Hourly)	Through (15 min)
4:30 – 4:45	1287	5146	78	312	1365
4:45 – 5:00	1315	5260	154	616	1469
5:00 – 5:15	1258	5032	200	800	1458
5:15 – 5:30	1039	4156	230	920	1269
5:30 – 5:45	949	3796	240	960	1189
5:45 – 6:00	892	3566	230	920	1122
6:00 – 615	937	3746	210	840	1147
6:15 – 6:30	983	3930	175	700	1158

Table 4-10 Input Data at Parmer Entrance

Time	Through (15 min)	Through (Hourly)	Parmer Entrance (15 min)	Parmer Entrance (Hourly)	Through (15 min)
4:30 – 4:45	1365	5460	251	1004	1616
4:45 – 5:00	1469	5876	283	1132	1752
5:00 – 5:15	1458	5832	308	1232	1766
5:15 – 5:30	1269	5076	401	1604	1670
5:30 – 5:45	1189	4756	406	1624	1595
5:45 – 6:00	1122	4488	384	1536	1506
6:00 – 615	1147	4588	301	1204	1448
6:15 – 6:30	1158	4632	267	1068	1425

Table 4-11 Input Data at Dessau Exit

Time	Through (15 min)	Through (Hourly)	Dessau Exit (15 min)	Dessau Exit (Hourly)	Through (15 min)
4:30 – 4:45	1616	6464	79	316	1537
4:45 – 5:00	1752	7008	66	264	1686
5:00 – 5:15	1766	7064	62	248	1704
5:15 – 5:30	1670	6680	47	188	1623
5:30 – 5:45	1595	6380	38	152	1557
5:45 – 6:00	1506	6024	37	148	1469
6:00 – 615	1448	5792	58	232	1390
6:15 – 6:30	1425	5700	63	252	1362

Table 4-12 Input Data at Wells Branch Exit

Time	Through (15 min)	Through (Hourly)	Wells Branch Exit (15 min)	Wells Branch Exit (Hourly)	Through (15 min)
4:30 – 4:45	1537	6148	459	1836	1078
4:45 – 5:00	1686	6744	415	1660	1271
5:00 – 5:15	1704	6816	437	1748	1267
5:15 – 5:30	1623	6492	419	1676	1204
5:30 – 5:45	1557	6228	436	1744	1121
5:45 – 6:00	1469	5876	437	1748	1032
6:00 – 615	1390	5560	425	1700	965
6:15 – 6:30	1362	5448	440	1760	922

Once the entrance ramp is closed, the flow in the area will redistribute itself over a long period. It will be difficult to estimate the flows on the links after closure unless one has an idea of the origin destination pattern of the various vehicles that use the freeway. Another way to estimate the flows is to conduct a scenario-based analysis that would give a conservative estimate of the benefits. Once the Yager Entrance ramp is closed, the vehicles that would have entered the freeway through this entrance ramp could: (a) travel along the frontage road further north and enter the freeway through the Parmer Entrance, (b) travel south along the frontage road and enter the freeway through the Braker entrance, (c) do not enter the freeway at all. Due to lack of data available, it is assumed that all the vehicles that would have entered the freeway through the Yager entrance ramp still enter the freeway through the Braker or Parmer Entrance. This is the worst case possible for the freeway and under normal conditions, some of the vehicles may not enter the freeway at all thereby providing a conservative analysis. The various flow split scenarios considered were 5:95, 10:90 up to 50:50. A flow split of 5:95 means that 5% of the vehicles that would have entered the freeway through the Yager Entrance ramp backtrack and enter I-35 through the Braker entrance and 95% of the vehicles enter the I-35 through the Parmer entrance ramp downstream.

Another phenomena modeled is the problem of queue jumping. Since the section of the I-35 being studied is extremely congested when queues are being formed, some vehicles might get on to the frontage road using Yager or Parmer Exit and then enter I-35 using the Yager entrance ramp. Queue jumpers decrease the throughput of the freeway section and increase the propensity of incidents in the area by increasing the amount of weaving in the freeway sections. Closure of the Yager Entrance ramp is expected to discourage the queue jumpers as they loose one way of getting on to the freeway. The different queue jumping scenarios considered were 25, 50, 75, 100, 125 and 150 queue jumpers.

As we can observe from the figure, only the conditions on segments 3, 4, and 5 will change with the closure of the Yager ramp. The LOS obtained for the various scenarios modeled are shown in the table.

The trends observed are as follows:

- LOS for link 1 is E, and for links 2, 6, 7, and 8 is D for every scenario.
- The density of link 4-5 changed however, the LOS remained the same at D for every split ratio and all level of queue jumpings considered.
- LOS for link 3 for the split ratio of 5:95 and 10:90 is D for all levels of queue jumping considered and is E for all the other split ratio considered.
- LOS for the entire stretch is observed to be at D for all the scenarios except for two scenarios where the LOS for the segment between Braker entrance and Parmer Exit is observed to be E.
- LOS for the stretch of the freeway from Parmer exit is constant at D for all the scenarios.
- LOS between Braker entrance and Parker exit is D for all the scenarios except for 2 split ratios where the LOS is observed to be E for all levels of queue jumping studied.

Table 4-13 LOS Base Case

Link No.	LOS
1	E
2	E
3	F
4	E
5	F
6	F
7	D
8	D

The Level of Service for links 1,2,6,7 and 8 is shown below in **Table 4-14**.

Table 4-14 LOS for all scenarios

Link	LOS
1	E
2	D
6	D
7	D
8	D

After the Yager ramp is closed, link 4 and link 5 will form a single segment and therefore it will be referred here as link 4-5. LOS for links 3 and 4-5 for various scenarios are given below.

Table 4-15 Queue Jumping = 0

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-16 Queue Jumping = 25

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-17 Queue Jumping = 50

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-18 Queue Jumping = 75

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-19 Queue Jumping = 100

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-20 Queue Jumpers = 125

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

Table 4-21 Queue Jumpers = 150

Split ratio	LOS: Link 3	LOS: Link 4-5
"5:95"	D	D
"10:90"	D	D
"15:85"	E	D
"20:80"	E	D
"25:75"	E	D
"30:70"	E	D
"35:65"	E	D
"40:60"	E	D
"45:55"	E	D
"50:50"	E	D

4.6.1.2 CORSIM Analysis

Level of Service analysis of the freeway section under entrance ramp closure was also conducted by simulating the traffic using CORSIM for various scenarios. The geometric details of the sections of the I-35 were obtained from aerial photographs of the study area. The traffic flow data contained evening peak-traffic flow volumes for the main line section of I-35 and all the entrance/exit ramps. Once the Yager entrance ramp is closed, it is difficult to predict the traffic flow volumes on the freeways and entrance/exit ramps accurately. Therefore, multiple scenarios are considered regarding the flow redistribution and queue jumping to determine the flows on the mainline interstate sections.

The level of service obtained for the various scenarios modeled are shown in the tables. The overall trends observed in level of service for various scenarios are described below.

- The level of service of the section of I-35 from Yager Exit to Braker Entrance decreased to E or F under some scenarios.
- By closing the Yager entrance, the performance of the section of the freeway from Parmer Exit to Parmer Entrance improved significantly. The level of service of that section improved to D (and in some cases C) for most of the scenarios.

- The level of service of the section of I-35 from Dessau exit to Wells Branch was observed to improve to C for some of the scenarios. The level of service remained the same at D for the other scenarios.
- There was no consistent increase in the level of service observed for the remaining sections of the freeway.

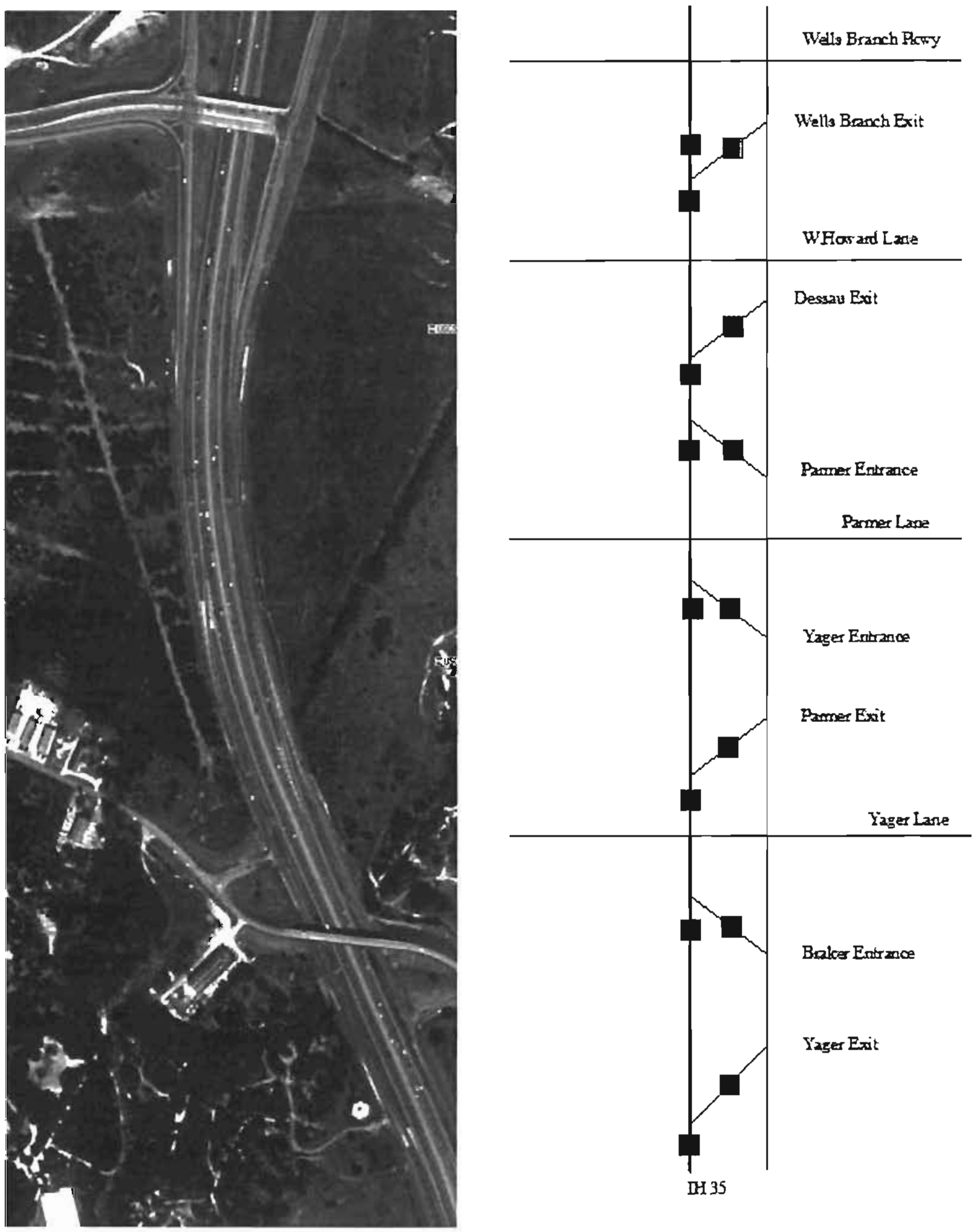


Figure 4-24 An aerial view of the freeway section between Yager Lane and Parmer Lane containing the Yager Entrance ramp (left), and A schematic representation of the freeway section (right)

Overall, the closure appears beneficial even under conservative system estimates/assumptions. In this study it is assumed that all the vehicles that currently use the Yager Entrance ramp would get on to the freeway using some other freeway section. This is the worst case possible. However, this may not be true as on closure some vehicles may choose to not to use the interstate. In such a scenario, the performance of the freeway is expected to be better than the results obtained from the CORSIM analysis. The analysis conducted is for all possible scenarios that may occur once the ramp is closed. If refined estimates of the flow splits and the number of queue jumpers are obtained, then analysis that is more precise can be conducted.

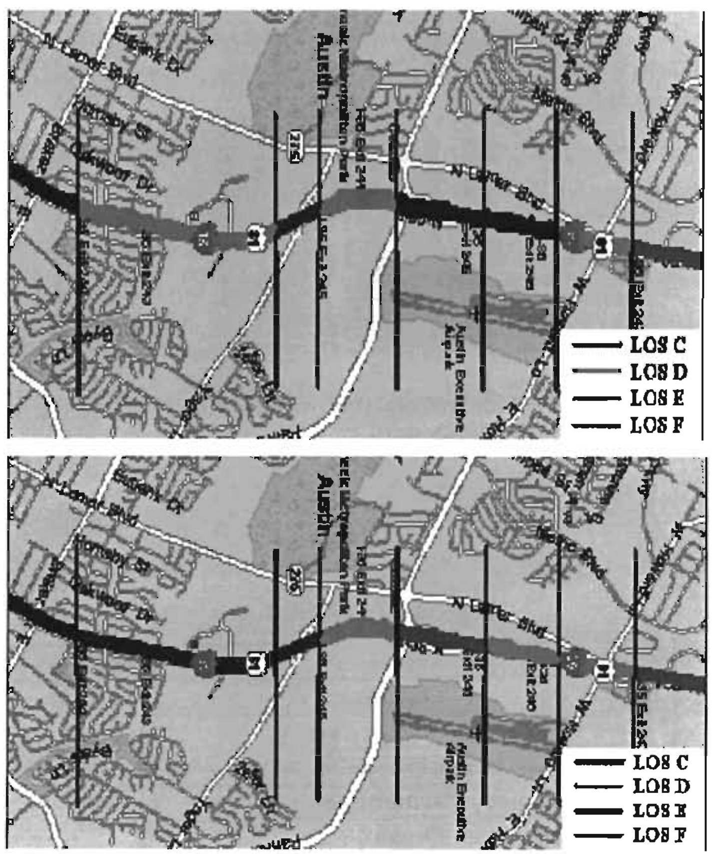


Figure 4-25 LOS of the I 35 sections (before and after) closing the Yager Entrance Ramp. Flow Split 5:95 and 50 queue jumpers

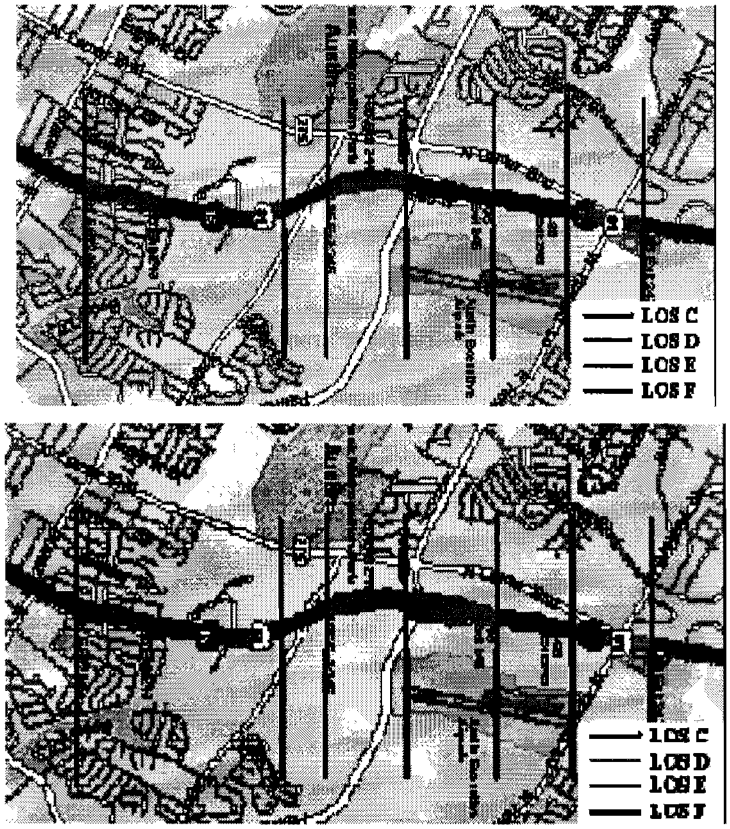


Figure 4-26 LOS of the I 35 sections (before and after) closing the Yager Entrance Ramp . Flow Split 5:95 and 25 queue jumpers

Table 4-22 CORSIM Analysis Base Case

Base Case	Section	LOS
Yager Exit - Braker Entrance	2	D
Braker Entrance - Parmer Exit	3	F
Parmer Exit - Yager Entrance	4	D
Yager Entrance - Parmer Entrance	5	E
Parmer Entrance - Dessau Exit	6	E
Dessau Exit - Wells Branch	7	D

Table 4-23 CORSIM Analysis Queue Jumpers = 0

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	D	E	E	E	F	F	E	F	E	F
3	F	F	F	F	F	F	F	F	F	F
4	D	D	C	D	D	C	C	D	C	D
5	E	D	C	D	D	C	C	C	C	C
6	F	E	D	E	D	D	E	D	D	D
7	D	D	C	C	C	C	C	C	C	C

Table 4-24 CORSIM Analysis Queue Jumpers = 25

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	D	D	E	E	E	F	F	F	F	F
3	F	F	F	F	F	F	F	E	F	F
4	D	D	D	D	D	D	D	F	D	D
5	D	D	D	D	C	C	C	D	C	D
6	E	F	E	E	D	D	D	D	D	D
7	D	D	C	D	C	C	C	D	C	C

Table 4-25 CORSIM Analysis Queue Jumpers = 50

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	E	F	E	D	F	E	E	F	F	F
3	F	F	F	F	F	F	F	F	F	F
4	D	D	D	D	C	D	D	D	D	D
5	C	D	D	D	C	D	D	D	C	D
6	D	F	E	E	D	D	E	E	D	D
7	D	D	D	D	C	C	C	D	C	C

Table 4-26 CORSIM Analysis Queue Jumpers = 75

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	D	E	D	F	D	D	F	E	F	F
3	F	F	F	F	F	F	F	F	F	F
4	D	D	D	D	D	D	D	D	D	D
5	D	D	D	D	D	D	D	D	D	D
6	F	F	F	F	F	E	E	E	D	D
7	D	D	D	D	D	C	C	D	C	C

Table 4-27 CORSIM Analysis Queue Jumpers = 100

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	E	E	F	E	D	E	F	F	E	F
3	E	F	F	F	F	F	F	F	F	F
4	C	D	D	D	D	C	D	D	D	D
5	C	D	D	D	D	C	D	D	D	D
6	D	E	F	D	E	D	D	D	F	D
7	C	D	D	D	C	C	C	D	D	C

Table 4-28 CORSIM Analysis Queue Jumpers =125

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	D	E	E	E	F	F	D	F	E	F
3	F	F	F	F	F	F	F	F	F	F
4	D	D	D	D	D	D	D	D	D	D
5	D	D	D	D	D	D	D	D	D	D
6	F	E	E	D	D	E	E	D	D	E
7	D	D	D	C	C	D	C	C	C	C

Table 4-29 CORSIM Analysis Queue Jumpers = 150

Sec	"5:95"	"10:90"	"15:85"	"20:80"	"25:75"	"30:70"	"35:65"	"40:60"	"45:55"	"50:50"
2	D	D	E	F	E	D	E	F	E	F
3	F	F	F	F	F	E	F	F	F	F
4	D	D	D	D	D	D	D	D	D	D
5	D	D	D	D	D	D	D	D	D	C
6	E	F	D	F	E	F	D	E	E	D
7	C	D	D	D	D	D	D	C	D	C

Some of the salient results of the CORSIM analysis for select scenarios are provided in the tables below.

Table 4-30 CORSIM Output , Queue Jumping 100 , Split 5:95

SEC	VEHICLES		LANE	CURR	AVG	VEH-	VEH-	TOTAL	MOVE	DELAY	VOL	DEN	SPEED
DATA	IN	OUT	CHNG	CONT	CONT	MILES	MIN	TIME	TIME	TIME	V/LN/HR	V/LN-M	M/HR
2	1429	1383	1110	159	127.1	1611.9	1907	80.9	63.3	17.6	1885.1	37.2	50.72
3	1491	1478	682	45	37.3	401.4	559.9	22.6	15	7.6	1894.2	44	43.01
4	1236	1222	307	52	38.7	578.5	580.2	28.3	26.1	2.3	1639	27.4	59.83
5	1222	1224	375	35	39.6	599.4	594.7	29.2	27.2	2	1629.2	26.9	60.47
6	1697	1695	1175	61	58.4	767.4	876	30.9	25	5.8	1702.5	32.4	52.56
7	1616	1612	915	57	49.5	742.6	742.1	27.6	25.6	2	1613.5	26.9	60.04

Table 4-31 CORSIM Output , Queue Jumping 100 , Split 10:90

SEC	VEHICLES		LANE	CURR	AVG	VEH-	VEH-	TOTAL	MOVE	DELAY	VOL	DEN	SPEED
DATA	IN	OUT	CHNG	CONT	CONT	MILES	MIN	TIME	TIME	TIME	V/LN/HR	V/LN-M	M/HR
2	1428	1376	1079	160	134.3	1613.8	2014	85.4	63.3	22.1	1887.2	39.3	48.08
3	1493	1497	810	40	46.9	401	704.1	28.4	15	13.4	1892.4	55.4	34.17
4	1259	1258	414	43	42	593.1	630.7	30	26.1	3.9	1680.2	29.8	56.42
5	1258	1254	478	44	41.7	617.7	625.9	29.8	27.2	2.6	1679	28.4	59.22
6	1717	1714	1228	72	76.5	775.6	1148	40	25.1	14.9	1720.6	42.4	40.55
7	1634	1642	942	40	52.7	755.2	790.2	28.9	25.6	3.3	1640.9	28.6	57.34

Table 4-32 CORSIM Output , Queue Jumping 100 , Split 15:85

SEC	VEHICLES		LANE	CURR	AVG	VEH-	VEH-	TOTAL	MOVE	DELAY	VOL	DEN	SPEED
DATA	IN	OUT	CHNG	CONT	CONT	MILES	MIN	TIME	TIME	TIME	V/LN/HR	V/LN-M	M/HR
2	1413	1400	1143	163	159	1622.5	2385	100.6	63.1	37.5	1897.4	46.5	40.81
3	1527	1530	787	39	40.5	411.6	607.2	23.9	15	8.9	1942.7	47.8	40.67
4	1277	1275	340	41	41.4	601.8	621.7	29.2	26.1	3.1	1704.8	29.4	58.07
5	1275	1265	532	49	45.3	626.8	679.3	31.9	27.2	4.7	1703.6	30.8	55.36
6	1719	1690	1387	101	87.7	765.1	1316	46.5	25.1	21.5	1897.3	48.7	34.88
7	1603	1598	1088	56	53.9	735.8	807.9	30.3	25.6	4.7	1598.4	29.3	54.63

Table 4-33 CORSIM Output , Queue Jumping 100 , Split 20:80

SEC	VEHICLES		LANE	CURR	AVG	VEH-	VEH-	TOTAL	MOVE	DELAY	VOL	DEN	SPEED
DATA	IN	OUT	CHNG	CONT	CONT	MILES	MIN	TIME	TIME	TIME	V/LN/HR	V/LN-M	M/HR
2	1445	1396	1049	148	126.5	1635.2	1898	79.4	63.3	16.1	1912.2	37	51.69
3	1532	1527	780	37	42.9	412.5	643.8	25.3	15	10.3	1946.6	50.6	38.44
4	1302	1293	320	50	41.8	612.5	826.9	28.9	26.1	2.8	1735.3	29.6	56.63
5	1293	1298	366	41	43	638	644.4	29.7	27.2	2.5	1734.2	29.2	59.4
6	1742	1692	1220	101	62.8	774.4	941.4	32.9	25.1	7.8	1717.9	34.8	49.36
7	1618	1627	968	58	51.6	745.4	777.3	28.6	25.6	3.2	1819.5	26.1	57.53

4.6.2 *Information Provision*

During the period of ramp closure, it is advisable to place three Changeable Message Signs - one on the frontage road (1000 ft south of the intersection), two on Yager Lanes on both sides 1000 ft east and 1000 ft west of the intersection. Further, the Changeable Message Sign should be activated approximately two minutes before the closure.

Flashing beacons should be placed on state information signs warning people of closed Yager entrance. In addition, the state warning on fines for drivers who violate should be present. One of the signs should be placed at the ramp entrance and the other CORSIM Analysis placed 100 feet from the ramp entrance. The beacons should be activated 45 seconds before the ramp is closed.

The ramp closure should not affect the traffic at the Yager Lane – frontage road intersection significantly. The volume of vehicles turning right may decrease slightly as there may be vehicles that may take alternate routes realizing that the ramp is closed. Therefore, no change in signalization is needed, as ramp entrance closure will not significantly affect the turning movements in the Yager Lane – frontage road intersection. The users of the Yager entrance are expected to travel north along the frontage road and use the next entrance to the interstate - the Parmer entrance. Therefore, at the Parmer Lane – frontage road intersection the number of northbound vehicles will increase if Yager entrance is closed. Therefore, during the period of the ramp closure, the green time for the north bound traffic on the Parmer lane - frontage road intersection should be increased.

4.6.3 *Summaries and Recommendations*

A study of the section of the northbound I-35 section in Austin was conducted. The impact of closing the Yager entrance ramp was studied. A scenario based Level of Service analysis was conducted using the methodology presented in the HCM and by simulation using CORSIM. The summary of the results of the analysis is presented below:

4.6.3.1 Summary of HCM analysis

- The LOS of the section from Parmer exit to Parmer entrance improves significantly to D under all scenarios and the LOS of the section of the freeway from Braker entrance to Parmer exit was found to improve to E.
- Ramp Closure is found not to decrease the performance of any section of the freeway for all the scenarios.

4.6.3.2 Summary of the CORSIM analysis

- Ramp Closure decreases the LOS of the section of I-35 from Yager exit to Braker entrance under some scenarios.
- Ramp Closure improved the performance of the section of the freeway from Parmer exit to Parmer entrance to D for most scenarios and the performance of the section of the freeway from Dessau exit to Wells Branch to C for some of the scenarios.
- No consistent increase/decrease in LOS was observed in other sections except the initial section where the LOS decreased for almost all scenarios.

4.6.3.3 Recommendations

From the scenario-based analysis, Yager entrance ramp closure is found to improve the performance of the freeway. A preliminary deployment plan has also been presented. However, before implementation a much more detailed study based on the guidelines presented in this report has to be conducted for the freeway section in Austin.

Variable schedule ramp closure in which the Yager entrance ramp is closed during peak hours and opened during non-peak hours is recommended. Hard Closure using automatic swing gates are recommended for usage. The material of the gate should preferably be aluminum, which is relatively cost effective and has sufficient resistance to low impact crashes. The components of the closure system must be easily replaceable when damaged due to crashes. The gates used must be FHWA approved or must be crash-tested as per the specifications provided in NCHRP report 350. Since crash testing a gate is a costly and cumbersome process, it is recommended that the gate used be the same used in Minnesota or Chicago. These gates are used in closing high volume, high speed roads like freeway ramps.

5 PLANNING AND IMPLEMENTATION CONSIDERATIONS

5.1 Crashworthiness

Conducting crash tests on new gate design is a costly and cumbersome process. Therefore, the research team recommends the usage of gates that have been already crash tested and approved by the FHWA. Examples of such gates are the gates used by Wyoming and Minnesota . Details of the design standards for both the gates can be found in the I 90 gate operations system research report (2001). For example, the gates used by Minnesota, Wyoming and Chicago are in traffic conditions similar to that of the recommended sites in El Paso and Austin. These gates are placed on ramps exiting/entering high-speed freeway sections. A surveyed list from interviewed DOT engineers of companies manufacturing such gates includes:

(i) B&B Electrical – used in Chicago
(ii) Thomtech Engineering Design – Minnesota
(iii) Winter Alpine Engineering Corporation – Wyoming
(iv) Safetran Systems –South Dakota
(v) Hy-Security Gate systems – South Dakota

5.2 Life-Cycle Cost Estimates

The cost of the gate alone is expected to be around $8200 without labor. Minnesota DOT experienced two accidents in the year of 2002, and the gate arm was damaged. The cost of integrating the gate with the ITS facility is expected to be around $60,000 - 100,000, varyingly depending on the scope and level of integration.

5.3 Public Awareness

One of the key steps in the ramp planning and operation closure process is to keep the public well aware of the planning and operation status. Once the candidate closure ramp is ready to be implemented, sufficient publicity has to be provided through mass media. The local press should be involved in the process. Since the local press plays an important role in molding the public opinion, significant effort and care should be placed on conveying to the local press the benefits of the system. Local legislative, law making and enforcing bodies could also be involved in the process. This is because the success of the ramp closure will depend on reducing the number of violators. Publicity also ensures that people directly affected by the closure of the ramp are aware of information like scheduling and hence can plan their routes accordingly. The public should be made aware of the tangible system benefits that will be obtained out of closing the ramps. If the benefits of the closure are exaggerated when presented to the public, it will lead to disillusionment when the system is in place leading to negative public perception.

5.4 Integrating ITS Technologies

The ramp closure should also be coordinated with the traffic signals and other ITS devices used for freeway management near the closed ramp. If there is any ramp metering done on the other ramps near the closed ramp then care should be taken to ensure that the metered flow volume is high. The ramp meters on the entrances in the immediate vicinity of the closed ramp must be shut down completely. This is because due to the closure of the ramp the ramp volumes on the other ramps in the vicinity will increase. Excessive metering in the ramps might lead to queue formation on the other ramps thus increasing the delay. Queues formed can extend to the arterials and the frontage roads thus leading to significant deterioration of the system performance. The ramp volume on the closed ramp will divert to alternate routes thus changing the traffic flow pattern on the surface streets. This will result in an increase/decrease in the volumes of various movements in the arterials or the surface streets. Thus, the actual green times of the various movements will change and additional green times will have to be provided wherever necessary. The possible paths/route taken by vehicles that would have used the closed ramps must be identified by a simple O-D trip analyses or by using Dynamic Traffic Assignment. The green times must be increased on all such movements. The green times on all possible routes leading to the closed ramp must be decreased.

The information about closure must be displayed on all Dynamic Message Signs near the ramp. Dynamic Message Signs must be placed on all inbound arterials. These signs must be placed at a distance of 1000 feet from the ramp. Dynamic Message Signs must be activated 2-5 minutes before the ramp closure. Warning signs combined with yellow flashing beacons must be placed on all inbound arterials. The beacons must be activated 45-90 seconds before the ramp closure and must be placed at a distance of 100-500 feet from the ramp. Warning sings should contain information about possible fines for all violators. The numbers recommended by this research team are obtained by a synthesis of the all the studies conducted for ramp closure.

Cameras are recommended to be placed on all gates. This helps in easier monitoring of the traffic conditions near the ramp. These cameras also aid in identifying errant drivers. They provide video evidence against motorists who crashed into the gates, and discouraged them from suing the DOT.

6 SYSTEM PERFORMANCE ASSESSMENT AND MONITORING

Although peak-hour ramp closure is not a usual freeway traffic control practice, it has been shown by this analysis to be a potential effective strategy when other ramp control strategies like ramp metering is not feasible. Because only limited prior experience has been documented, it is important to perform a thorough before-and-after-closure assessment with particular emphasis on the direct/indirect benefit/cost and safety impact on freeways and arterials. This chapter discusses a general procedure recommended for the short-term and long-term performance monitoring and assessment so that the effectiveness of peak-hour ramp closure operation can be constantly maintained.

Three-stage planning and operations tasks are defined for the peak-hour ramp closure implementation. They are discussed as follows.

- **Pre-implementation planning**

  Before the peak-hour ramp closure is implemented, efforts need to be made to undertake the following tasks, which include:

  (1) *Defining performance indicators*

  The performance indicators can be classified into three groups:

a. Freeway LOS performance

This group of indicators includes average speed, average density, average flow rate. For the study site, the scope of the assessed highway segment includes the entire segment shown in Figure 1-1.

b. Arterial LOS performance

This group of indicators includes average link speed, density, and flow rate, as well as intersection delays. The recommended intersections to be evaluated are indicated in circles in Figure 6-1.

c. Safety Impact

Number of accidents on both IH-10 main lanes and Paisano/Gateway W intersection needs to be collected. Number of gate collisions and close calls also need to be collected, documented and analyzed.

Figure 6-1 Recommended freeway performance assessment segment

(2) Setting up data collection plan

Data collection will primarily utilize the existing traffic detection system. Detectors data, deployed on main lanes and on/off ramps along IH-10, will be collected. Additional supplementary data can be sought through video surveillance or probe vehicles. For example, probe vehicles equipped with a GPS system can be dispatched during the time of interest in order to collect actual speed and travel time information. A video camera to monitor continuously the gate operation during the short-term testing phase is recommended. This is to help understand motorists' reaction and behavior before, during and after the closure.

(3) Collecting traffic and accident data on both freeways and arterials

Once the data collection scope and mechanism are defined, the data collection is recommended to start from at least 1-2 weeks before the scheduled start of ramp closure, continuing to a scheduled end of date.

(4) Inter-connecting traffic control devices and coordinating with other agencies (optional)

If the gate is connected with other control devices, (e.g. nearby flashers, dynamic message signs, changeable message signs, and/or intersection signal, etc.) the necessary connections need to be completed at this stage. It is recommended that the signal phasing at the Paisano/Gateway W intersection be set to "All Red" phase starting from 5-10 second before gate closure in motion until the completion of the gate closure. To ensure the gate and signals are properly synchronized, the inter-connection between them is recommended. There are several different ways for timing synchronization. Both TxDOT and the City of El Paso will need to agree on an inter-connection approach at the pre-implementation planning stage.

(5) Plan and deploy traffic control devices (i.e. where, when and how to deploy permanent or temporary traffic control devices)

In addition to the gate/intersection signal synchronization, other necessary traffic control devices need to be planned and deploy at this stage. The deployment of traffic control devices follows the recommended traffic control plan, which is briefly described as follows. It is noted that actual deployment of the traffic control plan may vary depending on other practical considerations at time of deployment.

a. Two flashers with warning messages (and lane assignment message with an arrow, such as "Use frontage road when ramp is closed") are recommended to be installed at the entrance of the Paisano ramp. It is also recommended that at least one additional flasher with warning messages to be installed at 150-200 feet[7] before the stop line of each inbound approach (Paisano north-/southbound, Gateway East westbound.) of the intersection (see Figure 6-2). All four flashers are activated at 5-10 seconds prior to the gate closure until the completion of gate closure.

Figure 6-2 Flasher locations

b. The Paisano / Gateway East intersection signal runs at a special "All Red" phase starting from about 5-10 seconds prior to the gate closure until the completion of gate closure.

c. Four portable changeable message signs are placed at major inbound approaches at least 500-1000 ft upstream of the intersection. The signs display messages indicating the time of

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[7] Based on 2.5-5.0 second of reaction time at the speed of 30 mph before motorists approach the intersection

the day of Paisano ramp closure. The messages signs are recommended to be deployed at least two weeks prior to the date of ramp closure deployment. The possible changeable message sign locations are shown in Figure 6-3

Figure 6-3 Locations for changeable message signs

d. Ramp closure information (starting date, and time of day, etc.) can be displayed on the dynamic message signs along IH-10 during peak hours, provided the ramp closure information does not preempt other incident/traffic/amber alert type of information. It is recommended that such messages be displayed two weeks prior to the deployment until a defined date.

e. The same information can also be displayed on the TransVista website following the same defined period as used by the dynamic message signs.

(6) Notifying the public of the upcoming ramp closure

Public notification mechanisms are recommended to disseminate the ramp closure information to the public prior to the closure. Press release can be sent to major newspapers and/or TV stations to increase the public awareness of the ramp closure event.

Also shown in Figure 6-4, the pre-implementation planning is recommended to start 1-2 months prior to the deployment – depending on the scope of work – to ensure that most likely scenarios and outcomes are anticipated and control measures are provided.

- **Short-term monitoring and evaluation**

  Days to weeks after the closure deployment is perhaps the most critical period in which traffic disturbance on arterials are likely to occur. During this period, motorists will start to encounter the closure on-site (if they are not aware of the closure prior to the closure) and try to adjust to different routes. Traffic patterns on both the IH-10 and the vicinity of ramp terminal on the arterials are likely to fluctuate during this period. Effort needs to be made to monitor continuously the motorists' behavior near the gate, to determine if hazardous traffic condition or driving behavior arises. At the end of this period, assessment and further improvement decisions may be made to improve the operation.

  It is recommended that such a short-term evaluation be performed at the end of first month of operations so that conclusive observations can be drawn and additional remedial measures can be put in place.

- **Long-term monitoring and evaluation**

  Long-term monitoring and evaluation is recommended in order to capture the equilibrated traffic dynamics. As previously discussed, traffic disturbance or motorist adaptations require a significant period to settle down to an equilibrium state. Six to twelve months is recommended as the minimal long-term monitoring and evaluation period. Over this period, TxDOT engineers can more realistically estimate (1) cost of maintenance or repair of the gate, (2) increase or decrease of incidents comparing to pre-implementation conditions, (3) traffic condition changes on freeways and adjacent arterials, (4) public perceptions/opinion.

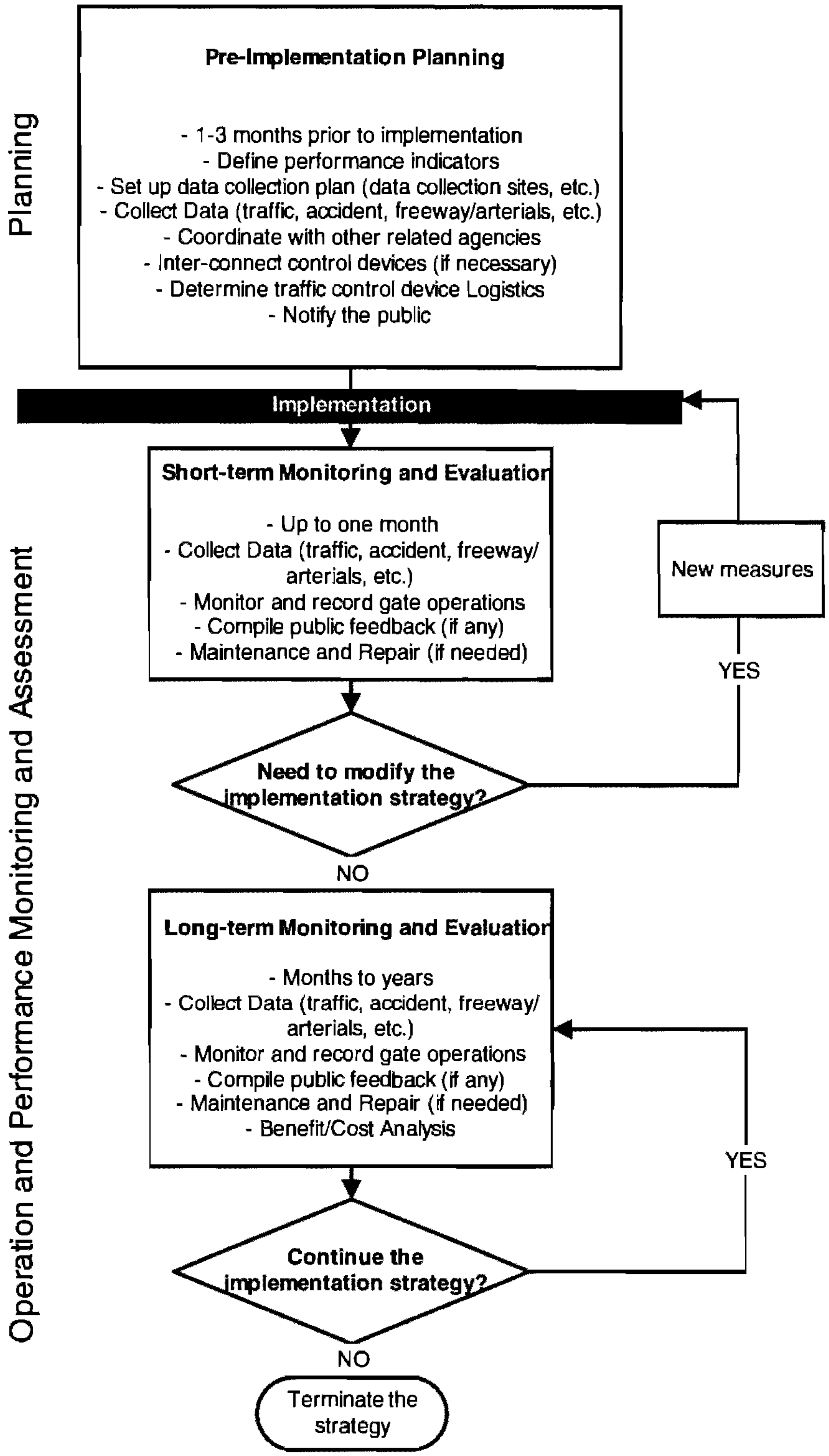

Figure 6-4 Framework for Implementation planning, performance monitoring and assessment (short-term and long-term)

7 RECOMMENDATIONS AND CONCLUDING REMARKS

After conducting series of rigorous laboratory study, the research concludes the following findings:

- Peak-hour ramp closure has been found to be a low-cost and effective strategy for both freeway main-lane flow control and managing queue jumping applications.
- Ramp metering has been shown not to be effective or feasible when the traffic flows in the downstream of the metered ramp is over the capacity. Metering the ramp does not improve the traffic flow conditions. It also imposes excessive queue on the ramp. In the study case, due to the short length of the ramp, the queue spills back to the upstream intersection for a significant period. Closing the ramp, equivalent to zero metering rate, is more effective in preventing intersection spillbacks and minimizing the violations.
- Establishing a suite of traffic control and impact mitigation strategies is the key for a successful implementation of peak-hour ramp closure. These strategies include:

- Synchronizing the adjacent intersection signal to ALL RED in conjunction with the transition of gate closure to prevent collision onset of closure
- Information provision/advance warning is crucial to prevent last minute diversion and/or confusion at the gate. It also facilitates better traffic diversion farther upstream of the closed ramp. Usage of mobile CMS or DMS is recommended, particularly during the short-term evaluation period, to promote public awareness of the peak-hour closure.

- Continuous performance assessment and improvement is recommended to ensure consistent and satisfactory operating performance of both the freeways and arterials.
- For the Austin case study, variable schedule ramp closure in which the Yager entrance ramp is closed during peak hours and opened during non-peak hours is recommended. Use of Hard Closure automatic swing gates is recommended. Aluminum is the recommended material, aluminum is relatively cost effective and has sufficient resistance to low impact crashes. The components of the closure system must be easily replaceable when damaged due to crashes. The gates used must be FHWA approved or must be crash- tested as per the specifications provided in NCHRP report 350. Since crash testing a gate is a costly and cumbersome process, it is recommended that the gate used be the same used in Minnesota or Chicago. These gates are used in closing high volume and high speed roads like freeway ramps.

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9 APPENDIX

Table 9-1 LOS worksheet for Section 1 in the existing situation

```
______________________________Inputs________________________________

Freeway free-flow speed, SFF                  65             mph
Weaving number of lanes, N                    4
Weaving segment length, L                     1153           ft
Terrain type                                  Level
    Grade                                                    %
    Length                                                   mi
Weaving type                                  A              Multilane or C-D
Volume ratio, VR                              0.18
Weaving ratio, R                              0.35

_____________Conversion to pc/h Under Base Conditions______________

                                         Non-Weaving      Weaving
                                         V       V        V       V
                                          A-C     B-D      A-D     B-C
Volume, V                                5918    52       445     778      veh/h
Peak-hour factor, PHF                    0.96    0.88     0.96    0.88
Peak 15-min volume, v15                  1541    15       116     221      v
Trucks and buses                         7       1        7       1        %
Recreational vehicles                    0       0        0       0        %
Trucks and buses PCE, ET                 1.5     1.5      1.5     1.5
Recreational vehicle PCE, ER             1.2     1.2      1.2     1.2
Heavy vehicle adjustment, fHV            0.966   0.995    0.966   0.995
Driver population adjustment, fP         1.00    1.00     1.00    1.00
Flow rate, v                             6380    59       479     888      pc/h

________________Weaving and Non-Weaving Speeds___________________

                                         Weaving      Non-Weaving
Weaving intensity factor, Wi             1.18         0.64
Weaving and non-weaving speeds, Si       40.21        48.56
Number of lanes required for
unconstrained operation, Nw (Exhibit 24-7)            1.13
Maximum number of lanes, Nw (max) (Exhibit 24-7)      1.40
Type of operation is                                  Unconstrained
```

____Weaving Segment Speed, Density, Level of Service and Capacity____

Weaving segment speed, S	46.85	mph
Weaving segment density, D	41.65	pc/mi/ln
Level of service, LOS	E	
Capacity for base condition, cb	7979	pc/h

________________Limitations on Weaving Segments__________________

	Analyzed	If Max Exceeded See Note Maximum	Note
Weaving flow rate, Vw	1367	2800	a
Average flow rate (pcphpl)	1951	2350	b
Volume ratio, VR	0.18	0.35	c
Weaving ratio, R	0.35	N/A	d
Weaving length (ft)	1153	2500	e

Table 9-2 LOS worksheet for the Section 4 in the existing situation

Inputs

Freeway free-flow speed, SFF	65	mph
Weaving number of lanes, N	4	
Weaving segment length, L	731	ft
Terrain type	Level	
Grade		%
Length		mi
Weaving type	A	Multilane or C-D
Volume ratio, VR	0.10	
Weaving ratio, R	0.27	

Conversion to pc/h Under Base Conditions

	Non-Weaving V A-C	Non-Weaving V B-D	Weaving V A-D	Weaving V B-C	
Volume, V	7013	38	217	532	veh/h
Peak-hour factor, PHF	0.96	0.87	0.96	0.87	
Peak 15-min volume, v15	1826	11	57	153	v
Trucks and buses	6	4	6	4	%
Recreational vehicles	0	0	0	0	%
Trucks and buses PCE, ET	1.5	1.5	1.5	1.5	
Recreational vehicle PCE, ER	1.2	1.2	1.2	1.2	
Heavy vehicle adjustment, fHV	0.971	0.980	0.971	0.980	
Driver population adjustment, fP	1.00	1.00	1.00	1.00	
Flow rate, v	7524	44	232	623	pc/h

Weaving and Non-Weaving Speeds

	Weaving	Non-Weaving
Weaving intensity factor, Wi	1.59	0.77
Weaving and non-weaving speeds, Si	36.25	46.14
Number of lanes required for unconstrained operation, Nw (Exhibit 24-7)		0.78
Maximum number of lanes, Nw (max) (Exhibit 24-7)		1.40
Type of operation is		Unconstrained

Weaving Segment Speed, Density, Level of Service and Capacity

Weaving segment speed, S	44.89	mph
Weaving segment density, D	46.91	pc/mi/ln
Level of service, LOS	F	
Capacity for base condition, cb	7827	pc/h

Limitations on Weaving Segments

	Analyzed	If Max Exceeded See Note: Maximum	Note
Weaving flow rate, Vw	855	2800	a
Average flow rate (pcphpl)	2105	2350	b
Volume ratio, VR	0.10	0.35	c
Weaving ratio, R	0.27	N/A	d
Weaving length (ft)	731	2500	e

Section 2 is defined to be a freeway basic segment. In HCS2000, the following information is required: traffic volume, peak hour factors, the percentage of trucks and buses and recreational vehicles, terrain types, driver population factors, number of lanes, lane width, lateral clearance, interchange density and free flow speed. All the inputs and outputs of this segment in HCS2000 are listed below:

Table 9-3 LOS worksheet for the Section 2 in the existing situation

Flow Inputs and Adjustments		
Volume, V	6696	veh/h
Peak-hour factor, PHF	0.96	
Peak 15-min volume, v15	1744	v
Trucks and buses	6	%
Recreational vehicles	0	%
Terrain type:	Level	
Grade	0.00	%
Segment length	0.00	mi
Trucks and buses PCE, ET	1.5	
Recreational vehicle PCE, ER	1.2	
Heavy vehicle adjustment, fHV	0.971	
Driver population factor, fp	1.00	
Flow rate, vp	1796	pc/h/ln
Speed Inputs and Adjustments		
Lane width	12.0	ft
Right-shoulder lateral clearance	6.0	ft
Interchange density	0.50	interchange/mi
Number of lanes, N	4	
Free-flow speed:	Ideal	
FFS or BFFS	70.0	mi/h
Lane width adjustment, fLW	0.0	mi/h
Lateral clearance adjustment, fLC	0.0	mi/h
Interchange density adjustment, fID	0.0	mi/h
Number of lanes adjustment, fN	1.5	mi/h
Free-flow speed, FFS	68.5	mi/h
	Urban Freeway	
LOS and Performance Measures		
Flow rate, vp	1796	pc/h/ln
Free-flow speed, FFS	68.5	mi/h
Average passenger-car speed, S	66.7	mi/h
Number of lanes, N	4	
Density, D	26.9	pc/mi/ln
Level of service, LOS	D	

The Sections 3 and 5 are defined to be ramp junction segments. More specifically, the former is an on-ramp junction segment (merge influence) and the latter is an off-ramp junction (diverge influence). In HCS2000, the following data is required: number of lanes on-ramp and freeway, free flow speed on-ramp and freeway, side of freeway ramp connection, length of first acceleration/deceleration lane, the adjacent ramp data, traffic volume composition and terrain. All the specific inputs and outputs of these two segments in HCS2000 are listed below:

Table 9-4 LOS worksheet for Section 3 in the existing situation

```
_____________________________Freeway Data___________________________

Type of analysis                            Merge
Number of lanes in freeway                  4
Free-flow speed on freeway                  55.0          mph
Volume on freeway                           7230          vph

_____________________________On-ramp Data___________________________

Side of freeway                             Right
Number of lanes in ramp                     1
Free-flow speed on-ramp                     35.0          mph
Volume on-ramp                              534           vph
Length of first accel/decel lane            590           ft
Length of second accel/decel lane                         ft

___________________Adjacent Ramp Data (if one exists)______________

Does adjacent ramp exist?                   Yes
Volume on adjacent Ramp                     570           vph
Position of adjacent Ramp                   Downstream
Type of adjacent Ramp                       On
Distance to adjacent Ramp                   1579          ft

_______________Conversion to pc/h Under Base Conditions____________

Junction Components                    Freeway     Ramp        Adjacent
                                                               Ramp
Volume, V (vph)                        7230        534         570        vph
Peak-hour factor, PHF                  0.96        0.88        0.87
Peak 15-min volume, v15                1883        152         164        v
Trucks and buses                       6           6           4          %
Recreational vehicles                  0           0           0          %
Terrain type:                          Level       Level       Level
     Grade                                     %           %           %
     Length                                    mi          mi          mi
Trucks and buses PCE, ET               1.5         1.5         1.5
Recreational vehicle PCE, ER           1.2         1.2         1.2
Heavy vehicle adjustment, fHV          0.971       0.971       0.980
Driver population factor, fP           1.00        1.00        1.00
Flow rate, vp                          7757        625         668        pcph

__________________Estimation of V12 Merge Areas____________________

                  L   =    0.00    (Equation 25-2 or 25-3)
                   EQ
                  P   =    0.328   Using Equation  4
                   FM
                  v   = v  (P  ) =  2541   pc/h
                   12    F   FM

________________________Capacity Checks____________________________

                           Actual        Maximum         LOS F?
     v                     8382          9000            No
      FO
     v                     3166          4600            No
      R12

__________Level of Service Determination (if not F)______________

Density, D = 5.475 + 0.00734 v  + 0.0078 v   - 0.00627 L  =   26.2    pc/mi/ln
          R                    R          12             A
Level of service for ramp-freeway junction areas of influence  C
```

Speed Estimation			
Intermediate speed variable,	M_S	= 0.372	
Space mean speed in ramp influence area,	S_R	= 50.2	mph
Space mean speed in outer lanes,	S_0	= 46.6	mph
Space mean speed for all vehicles,	S	= 47.9	mph

Table 9-5 LOS worksheet for Section 5 in the existing situation

```
____________________________Freeway Data______________________________

Type of analysis                            Diverge
Number of lanes in freeway                  4
Free-flow speed on freeway                  55.0           mph
Volume on freeway                           7545           vph

______________________________Off-ramp Data___________________________

Side of freeway                             Right
Number of lanes in ramp                     2
Free-Flow speed on-ramp                     35.0           mph
Volume on-ramp                              2331           vph
Length of first accel/decel lane            140            ft
Length of second accel/decel lane           140            ft

________________Adjacent Ramp Data (if one exists)____________________

Does adjacent ramp exist?                   Yes
Volume on adjacent ramp                     255            vph
Position of adjacent ramp                   Upstream
Type of adjacent ramp                       Off
Distance to adjacent ramp                   1619           ft

_____________Conversion to pc/h Under Base Conditions_________________

Junction Components                    Freeway     Ramp        Adjacent
                                                               Ramp
Volume, V (vph)                        7545        2331        255       vph
Peak-hour factor, PHF                  0.96        0.98        0.72
Peak 15-min volume, v15                1965        595         89        v
Trucks and buses                       6           4           7         %
Recreational vehicles                  0           0           0         %
Terrain type:                          Level       Level       Level
     Grade                             0.00    %   0.00    %   0.00    %
     Length                            0.00    mi  0.00    mi  0.00    mi
Trucks and buses PCE, ET               1.5         1.5         1.5
Recreational vehicle PCE, ER           1.2         1.2         1.2
Heavy vehicle adjustment, fHV          0.971       0.980       0.966
Driver population factor, fP           1.00        1.00        1.00
Flow rate, vp                          8095        2426        367       pcph

___________________Estimation of V12 Diverge Areas___________________

                  L  =    0.00    (Equation 25-8 or 25-9)
                   EQ
                  P  =    0.260   Using Equation  0
                   FD
                  v  = v  + (v - v ) P  =  3900   pc/h
                   12   R     F   R   FD

___________________________Capacity Checks___________________________

                         Actual        Maximum         LOS F?
     v  = v              8095          9000            No
      Fi   F
     v                   3900          4400            No
      12
     v  = v - v          5669          9000            No
      FO   F   R
     v                   2426          3800            No
      R

___________Level of Service Determination (if not F)_______________
```

Density, $D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$ = 34.0 pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable,	D_S = 0.646	
Space mean speed in ramp influence area,	S_R = 47	mph
Space mean speed in outer lanes,	S_0 = 56.1	mph
Space mean speed for all vehicles,	S = 51.1	mph

Inputs

Freeway free-flow speed, SFF	65	mph
Weaving number of lanes, N	4	
Weaving segment length, L	1115	ft
Terrain type	Level	
Grade		%
Length		mi
Weaving type	A	Multilane or C-D
Volume ratio, VR	0.21	
Weaving ratio, R	0.28	

Conversion to pc/h Under Base Conditions

	Non-Weaving		Weaving		
	V_{A-C}	V_{B-D}	V_{A-D}	V_{B-C}	
Volume, V	5918	52	445	1063	veh/h
Peak-hour factor, PHF	0.96	0.88	0.96	0.88	
Peak 15-min volume, v15	1541	15	116	302	v
Trucks and buses	7	0	7	0	%
Recreational vehicles	0	0	0	0	%
Trucks and buses PCE, ET	1.5	1.5	1.5	1.5	
Recreational vehicle PCE, ER	1.2	1.2	1.2	1.2	
Heavy vehicle adjustment, fHV	0.966	1.000	0.966	1.000	
Driver population adjustment, fP	1.00	1.00	1.00	1.00	
Flow rate, v	6380	59	479	1207	pc/h

Weaving and Non-Weaving Speeds

	Weaving	Non-Weaving
Weaving intensity factor, Wi	1.34	0.77
Weaving and non-weaving speeds, Si	38.51	46.08
Number of lanes required for unconstrained operation, Nw (Exhibit 24-7)		1.26
Maximum number of lanes, Nw (max) (Exhibit 24-7)		1.40
Type of operation is		Unconstrained

Weaving Segment Speed, Density, Level of Service and Capacity

Weaving segment speed, S	44.27	mph
Weaving segment density, D	45.88	pc/mi/ln
Level of service, LOS	F	
Capacity for base condition, cb	7725	pc/h

Limitations on Weaving Segments

	Analyzed	If Max Exceeded See Note Maximum	Note
Weaving flow rate, Vw	1686	2800	a

Average flow rate (pcphpl)	2031	2350	b
Volume ratio, VR	0.21	0.35	c
Weaving ratio, R	0.28	N/A	d
Weaving length (ft)	1115	2500	e

Table 9-6 LOS worksheet for Section 1 in Scenario 1

Inputs

Freeway free-flow speed, SFF	65	mph
Weaving number of lanes, N	4	
Weaving segment length, L	1115	ft
Terrain type	Level	
Grade		%
Length		mi
Weaving type	A	Multilane or C-D
Volume ratio, VR	0.21	
Weaving ratio, R	0.28	

Conversion to pc/h Under Base Conditions

	Non-Weaving		Weaving		
	V A-C	V B-D	V A-D	V B-C	
Volume, V	5918	52	445	1063	veh/h
Peak-hour factor, PHF	0.96	0.88	0.96	0.88	
Peak 15-min volume, v15	1541	15	116	302	v
Trucks and buses	7	0	7	0	%
Recreational vehicles	0	0	0	0	%
Trucks and buses PCE, ET	1.5	1.5	1.5	1.5	
Recreational vehicle PCE, ER	1.2	1.2	1.2	1.2	
Heavy vehicle adjustment, fHV	0.966	1.000	0.966	1.000	
Driver population adjustment, fP	1.00	1.00	1.00	1.00	
Flow rate, v	6380	59	479	1207	pc/h

Weaving and Non-Weaving Speeds

	Weaving	Non-Weaving
Weaving intensity factor, Wi	1.34	0.77
Weaving and non-weaving speeds, Si	38.51	46.08
Number of lanes required for		
Unconstrained operation, Nw (Exhibit 24-7)		1.26
Maximum number of lanes, Nw (max) (Exhibit 24-7)		1.40
Type of operation is		Unconstrained

Weaving Segment Speed, Density, Level of Service and Capacity

Weaving segment speed, S	44.27	mph
Weaving segment density, D	45.88	pc/mi/ln
Level of service, LOS	F	
Capacity for base condition, cb	7725	pc/h

Limitations on Weaving Segments

	Analyzed	If Max Exceeded See Note Maximum	Note
Weaving flow rate, Vw	1686	2800	a
Average flow rate (pcphpl)	2031	2350	b
Volume ratio, VR	0.21	0.35	c
Weaving ratio, R	0.28	N/A	d
Weaving length (ft)	1115	2500	e

Table 9-7 LOS worksheet for Section 2 in Scenario 1

Flow Inputs and Adjustments		
Volume, V	6981	veh/h
Peak-hour factor, PHF	0.96	
Peak 15-min volume, v15	1818	v
Trucks and buses	6	%
Recreational vehicles	0	%
Terrain type:	Level	
Grade	0.00	%
Segment length	0.00	mi
Trucks and buses PCE, ET	1.5	
Recreational vehicle PCE, ER	1.2	
Heavy vehicle adjustment, fHV	0.971	
Driver population factor, fp	1.00	
Flow rate, vp	1873	pc/h/ln
Speed Inputs and Adjustments		
Lane width	12.0	ft
Right-shoulder lateral clearance	6.0	ft
Interchange density	0.50	interchange/mi
Number of lanes, N	4	
Free-flow speed:	Ideal	
FFS or BFFS	70.0	mi/h
Lane width adjustment, fLW	0.0	mi/h
Lateral clearance adjustment, fLC	0.0	mi/h
Interchange density adjustment, fID	0.0	mi/h
Number of lanes adjustment, fN	1.5	mi/h
Free-flow speed, FFS	68.5	mi/h
	Urban Freeway	
LOS and Performance Measures		
Flow rate, vp	1873	pc/h/ln
Free-flow speed, FFS	68.5	mi/h
Average passenger-car speed, S	65.8	mi/h
Number of lanes, N	4	
Density, D	28.4	pc/mi/ln
Level of service, LOS	D	

The Section 4 is an off-ramp junction in Scenario 1 because the Paisano on-ramp is blocked. Thus, Sections 3, 4, and 5 are defined to be ramp junction segments. All the specific inputs and outputs of these three segments in HCS2000 are listed below:

Table 9-8 LOS worksheet for Section 3 in Scenario 1

Type of analysis	Merge	
Number of lanes in freeway	4	
Free-flow speed on freeway	55.0	mph
Volume on freeway	6981	vph

On-ramp Data

Side of freeway	Right	
Number of lanes in ramp	1	
Free-flow speed on-ramp	35.0	mph
Volume on-ramp	819	vph
Length of first accel/decel lane	590	ft
Length of second accel/decel lane		ft

Adjacent Ramp Data (if one exists)

Does adjacent ramp exist?	Yes	
Volume on adjacent Ramp	497	vph
Position of adjacent Ramp	Upstream	
Type of adjacent Ramp	Off	
Distance to adjacent Ramp	1003	ft

Conversion to pc/h Under Base Conditions

Junction Components	Freeway		Ramp		Adjacent Ramp	
Volume, V (vph)	6981		819		497	vph
Peak-hour factor, PHF	0.96		0.88		0.87	
Peak 15-min volume, v15	1818		233		143	v
Trucks and buses	6		6		3	%
Recreational vehicles	0		0		0	%
Terrain type:	Level		Level		Level	
Grade		%		%		%
Length		mi		mi		mi
Trucks and buses PCE, ET	1.5		1.5		1.5	
Recreational vehicle PCE, ER	1.2		1.2		1.2	
Heavy vehicle adjustment, fHV	0.971		0.971		0.985	
Driver population factor, fP	1.00		1.00		1.00	
Flow rate, vp	7490		959		580	pcph

Estimation of V12 Merge Areas

L_{EQ} = 0.00 (Equation 25-2 or 25-3)

P_{FM} = 0.286 Using Equation 4

$v_{12} = v_F (P_{FM})$ = 2141 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	8449	9000	No
v_{R12}	3100	4600	No

Level of Service Determination (if not F)

Density, $D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$ = 25.5 pc/mi/ln

Level of service for ramp-freeway junction areas of influence C

Speed Estimation

Intermediate speed variable,	M_S = 0.366	
Space mean speed in ramp influence area,	S_R = 50.2	mph
Space mean speed in outer lanes,	S_0 = 46.2	mph
Space mean speed for all vehicles,	S = 47.6	mph

Freeway Data

Type of analysis	Diverge	
Number of lanes in freeway	4	
Free-flow speed on freeway	55.0	mph
Volume on freeway	7800	vph

Off-ramp Data

Side of freeway	Right	
Number of lanes in ramp	1	
Free-Flow speed on-ramp	35.0	mph
Volume on-ramp	255	vph
Length of first accel/decel lane	500	ft
Length of second accel/decel lane		ft

Adjacent Ramp Data (if one exists)

Does adjacent ramp exist?	Yes	
Volume on adjacent ramp	2331	vph
Position of adjacent ramp	Downstream	
Type of adjacent ramp	Off	
Distance to adjacent ramp	1619	ft

Conversion to pc/h Under Base Conditions

Junction Components	Freeway		Ramp		Adjacent Ramp	
Volume, V (vph)	7800		255		2331	vph
Peak-hour factor, PHF	0.96		0.72		0.98	
Peak 15-min volume, v15	2031		89		595	v
Trucks and buses	6		7		4	%
Recreational vehicles	0		0		0	%
Terrain type:	Level		Level		Level	
Grade	0.00	%	0.00	%	0.00	%
Length	0.00	mi	0.00	mi	0.00	mi
Trucks and buses PCE, ET	1.5		1.5		1.5	
Recreational vehicle PCE, ER	1.2		1.2		1.2	
Heavy vehicle adjustment, fHV	0.971		0.966		0.980	
Driver population factor, fP	1.00		1.00		1.00	
Flow rate, vp	8369		367		2426	pcph

Estimation of V12 Diverge Areas

L_{EQ} = 0.00 (Equation 25-8 or 25-9)

P_{FD} = 0.436 Using Equation 8

$v_{12} = v_R + (v_F - v_R) P_{FD}$ = 3856 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{Fi} = v_F$	8369	9000	No
v_{12}	3856	4400	No
$v_{FO} = v_F - v_R$	8002	9000	No
v_R	367	2000	No

Level of Service Determination (if not F)

Density, $D_R = 4.252 + 0.0086\, v_{12} - 0.009\, L_D$ = 32.9 pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable, D_S = 0.461

Space mean speed in ramp influence area, S_R = 49 mph

Space mean speed in outer lanes, S_0 = 55.4 mph

Space mean speed for all vehicles, S = 52.3 mph

Table 9-9 LOS worksheet for Section 4 in Scenario 1

```
_________________________Freeway Data__________________________

Type of analysis                            Diverge
Number of lanes in freeway                  4
Free-flow speed on freeway                  55.0          mph
Volume on freeway                           7545          vph

_________________________Off-ramp Data__________________________

Side of freeway                             Right
Number of lanes in ramp                     2
Free-Flow speed on-ramp                     35.0          mph
Volume on-ramp                              2331          vph
Length of first accel/decel lane            140           ft
Length of second accel/decel lane           140           ft

________________Adjacent Ramp Data (if one exists)___________

Does adjacent ramp exist?                   Yes
Volume on adjacent ramp                     255           vph
Position of adjacent ramp                   Upstream
Type of adjacent ramp                       Off
Distance to adjacent ramp                   1619          ft

___________Conversion to pc/h Under Base Conditions___________

Junction Components                    Freeway     Ramp        Adjacent
                                                               Ramp
Volume, V (vph)                        7545        2331        255        vph
Peak-hour factor, PHF                  0.96        0.98        0.72
Peak 15-min volume, v15                1965        595         89         v
Trucks and buses                       6           4           7          %
Recreational vehicles                  0           0           0          %
Terrain type:                          Level       Level       Level
     Grade                             0.00    %   0.00    %   0.00    %
     Length                            0.00    mi  0.00    mi  0.00    mi
Trucks and buses PCE, ET               1.5         1.5         1.5
Recreational vehicle PCE, ER           1.2         1.2         1.2
Heavy vehicle adjustment, fHV          0.971       0.980       0.966
Driver population factor, fP           1.00        1.00        1.00
Flow rate, vp                          8095        2426        367        pcph

_________________Estimation of V12 Diverge Areas______________

                  L   =    0.00    (Equation 25-8 or 25-9)
                   EQ
                  P   =    0.260   Using Equation  0
                   FD
                  v   = v  + (v - v ) P  =  3900   pc/h
                   12   R     F   R  FD

________________________Capacity Checks_________________________

                                Actual        Maximum         LOS F?
     v   = v                    8095          9000            No
      Fi    F
     v                          3900          4400            No
      12
     v   = v - v                5669          9000            No
      FO   F   R
     v                          2426          3800            No
      R
```

Level of Service Determination (if not F)

Density, $D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D = 34.0$ pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable, $D_S = 0.646$

Space mean speed in ramp influence area, $S_R = 47$ mph

Space mean speed in outer lanes, $S_0 = 56.1$ mph

Space mean speed for all vehicles, $S = 51.1$ mph

Table 9-10 LOS worksheet for Section 5 in Scenario 1

Inputs

Freeway free-flow speed, SFF	65	mph
Weaving number of lanes, N	4	
Weaving segment length, L	1153	ft
Terrain type	Level	
Grade		%
Length		mi
Weaving type	A	Multilane or C-D
Volume ratio, VR	0.18	
Weaving ratio, R	0.35	

Conversion to pc/h Under Base Conditions

	Non-Weaving		Weaving		
	V A-C	V B-D	V A-D	V B-C	
Volume, V	5918	52	445	778	veh/h
Peak-hour factor, PHF	0.96	0.88	0.96	0.88	
Peak 15-min volume, v15	1541	15	116	221	v
Trucks and buses	7	1	7	1	%
Recreational vehicles	0	0	0	0	%
Trucks and buses PCE, ET	1.5	1.5	1.5	1.5	
Recreational vehicle PCE, ER	1.2	1.2	1.2	1.2	
Heavy vehicle adjustment, fHV	0.966	0.995	0.966	0.995	
Driver population adjustment, fP	1.00	1.00	1.00	1.00	
Flow rate, v	6380	59	479	888	pc/h

Weaving and Non-Weaving Speeds

	Weaving	Non-Weaving
Weaving intensity factor, Wi	1.18	0.64
Weaving and non-weaving speeds, Si	40.21	48.56
Number of lanes required for unconstrained operation, Nw (Exhibit 24-7)		1.13
Maximum number of lanes, Nw (max) (Exhibit 24-7)		1.40
Type of operation is		Unconstrained

Weaving Segment Speed, Density, Level of Service and Capacity

Weaving segment speed, S	46.85	mph
Weaving segment density, D	41.65	pc/mi/ln
Level of service, LOS	E	
Capacity for base condition, cb	7979	pc/h

Limitations on Weaving Segments

		If Max Exceeded See Note	
	Analyzed	Maximum	Note
Weaving flow rate, Vw	1367	2800	a
Average flow rate (pcphpl)	1951	2350	b
Volume ratio, VR	0.18	0.35	c
Weaving ratio, R	0.35	N/A	d
Weaving length (ft)	1153	2500	e

Table 9-11 LOS worksheet for Section 1 in Scenario 2

Inputs

Freeway free-flow speed, SFF	65	mph
Weaving number of lanes, N	4	
Weaving segment length, L	1153	ft
Terrain type	Level	
Grade		%
Length		mi
Weaving type	A	Multilane or C-D
Volume ratio, VR	0.18	
Weaving ratio, R	0.35	

Conversion to pc/h Under Base Conditions

	Non-Weaving V A-C	Non-Weaving V B-D	Weaving V A-D	Weaving V B-C	
Volume, V	5918	52	445	778	veh/h
Peak-hour factor, PHF	0.96	0.88	0.96	0.88	
Peak 15-min volume, v15	1541	15	116	221	v
Trucks and buses	7	1	7	1	%
Recreational vehicles	0	0	0	0	%
Trucks and buses PCE, ET	1.5	1.5	1.5	1.5	
Recreational vehicle PCE, ER	1.2	1.2	1.2	1.2	
Heavy vehicle adjustment, fHV	0.966	0.995	0.966	0.995	
Driver population adjustment, fP	1.00	1.00	1.00	1.00	
Flow rate, v	6380	59	479	888	pc/h

Weaving and Non-Weaving Speeds

	Weaving	Non-Weaving
Weaving intensity factor, Wi	1.18	0.64
Weaving and non-weaving speeds, Si	40.21	48.56
Number of lanes required for unconstrained operation, Nw (Exhibit 24-7)		1.13
Maximum number of lanes, Nw (max) (Exhibit 24-7)		1.40
Type of operation is		Unconstrained

Weaving Segment Speed, Density, Level of Service and Capacity

Weaving segment speed, S	46.85	mph
Weaving segment density, D	41.65	pc/mi/ln
Level of service, LOS	E	
Capacity for base condition, cb	7979	pc/h

Limitations on Weaving Segments

	Analyzed	Maximum	If Max Exceeded See Note
Weaving flow rate, Vw	1367	2800	a
Average flow rate (pcphpl)	1951	2350	b
Volume ratio, VR	0.18	0.35	c
Weaving ratio, R	0.35	N/A	d
Weaving length (ft)	1153	2500	e

Table 9-12 LOS worksheet for Section 2 in Scenario 2

Volume, V	6696	veh/h
Peak-hour factor, PHF	0.96	
Peak 15-min volume, v15	1744	v
Trucks and buses	6	%
Recreational vehicles	0	%
Terrain type:	Level	
Grade	0.00	%
Segment length	0.00	mi
Trucks and buses PCE, ET	1.5	
Recreational vehicle PCE, ER	1.2	
Heavy vehicle adjustment, fHV	0.971	
Driver population factor, fp	1.00	
Flow rate, vp	1796	pc/h/ln
Speed Inputs and Adjustments		
Lane width	12.0	ft
Right-shoulder lateral clearance	6.0	ft
Interchange density	0.50	interchange/mi
Number of lanes, N	4	
Free-flow speed:	Ideal	
FFS or BFFS	70.0	mi/h
Lane width adjustment, fLW	0.0	mi/h
Lateral clearance adjustment, fLC	0.0	mi/h
Interchange density adjustment, fID	0.0	mi/h
Number of lanes adjustment, fN	1.5	mi/h
Free-flow speed, FFS	68.5	mi/h
	Urban Freeway	
LOS and Performance Measures		
Flow rate, vp	1796	pc/h/ln
Free-flow speed, FFS	68.5	mi/h
Average passenger-car speed, S	66.7	mi/h
Number of lanes, N	4	
Density, D	26.9	pc/mi/ln
Level of service, LOS	D	

In Scenario 2, the Sections 3-5 are defined to be to ramp junction segments. Their specific inputs and outputs in HCS2000 are listed below:

Table 9-13 LOS worksheet for Section 3 in Scenario 2

```
_________________________Freeway Data____________________________

Type of analysis                            Merge
Number of lanes in freeway                  4
Free-flow speed on freeway                  55.0           mph
Volume on freeway                           6696           vph

__________________________On-ramp Data__________________________

Side of freeway                             Right
Number of lanes in ramp                     1
Free-flow speed on-ramp                     35.0           mph
Volume on-ramp                              819            vph
Length of first accel/decel lane            590            ft
Length of second accel/decel lane                          ft

__________________Adjacent Ramp Data (if one exists)_____________

Does adjacent ramp exist?                   Yes
Volume on adjacent Ramp                     497            vph
Position of adjacent Ramp                   Upstream
Type of adjacent Ramp                       Off
Distance to adjacent Ramp                   1003           ft

_____________Conversion to pc/h Under Base Conditions_____________

Junction Components                    Freeway     Ramp        Adjacent
                                                               Ramp
Volume, V (vph)                        6696        819         497        vph
Peak-hour factor, PHF                  0.96        0.88        0.87
Peak 15-min volume, v15                1744        233         143        v
Trucks and buses                       6           6           4          %
Recreational vehicles                  0           0           0          %
Terrain type:                          Level       Level       Level
     Grade                                     %           %          %
     Length                                    mi          mi         mi
Trucks and buses PCE, ET               1.5         1.5         1.5
Recreational vehicle PCE, ER           1.2         1.2         1.2
Heavy vehicle adjustment, fHV          0.971       0.971       0.980
Driver population factor, fP           1.00        1.00        1.00
Flow rate, vp                          7184        959         583        pcph

_________________Estimation of V12 Merge Areas_____________________

                  L   =    0.00    (Equation 25-2 or 25-3)
                   EQ
                  P   =    0.286   Using Equation  4
                   FM
                  v   = v   (P  ) =   2054   pc/h
                   12    F    FM

________________________Capacity Checks____________________________

                          Actual          Maximum         LOS F?
     v                    8143            9000            No
      FO
     v                    3013            4600            No
      R12


___________Level of Service Determination (if not F)_______________
```

Density, $D_R = 5.475 + 0.00734\, v_R + 0.0078\, v_{12} - 0.00627\, L_A$ = 24.8 pc/mi/ln

Level of service for ramp-freeway junction areas of influence C

Speed Estimation

Intermediate speed variable,	M_S = 0.359	
Space mean speed in ramp influence area,	S_R = 50.3	mph
Space mean speed in outer lanes,	S_0 = 46.9	mph
Space mean speed for all vehicles,	S = 48.1	mph

Freeway Data

Type of analysis	Diverge	
Number of lanes in freeway	4	
Free-flow speed on freeway	55.0	mph
Volume on freeway	7515	vph

Off-ramp Data

Side of freeway	Right	
Number of lanes in ramp	1	
Free-Flow speed on-ramp	35.0	mph
Volume on-ramp	255	vph
Length of first accel/decel lane	500	ft
Length of second accel/decel lane		ft

Adjacent Ramp Data (if one exists)

Does adjacent ramp exist?	Yes	
Volume on adjacent ramp	2331	vph
Position of adjacent ramp	Downstream	
Type of adjacent ramp	On	
Distance to adjacent ramp	1619	ft

Conversion to pc/h Under Base Conditions

Junction Components	Freeway		Ramp		Adjacent Ramp	
Volume, V (vph)	7515		255		2331	vph
Peak-hour factor, PHF	0.96		0.72		0.98	
Peak 15-min volume, v15	1957		89		595	v
Trucks and buses	6		7		4	%
Recreational vehicles	0		0		0	%
Terrain type:	Level		Level		Level	
Grade	0.00	%	0.00	%	0.00	%
Length	0.00	mi	0.00	mi	0.00	mi
Trucks and buses PCE, ET	1.5		1.5		1.5	
Recreational vehicle PCE, ER	1.2		1.2		1.2	
Heavy vehicle adjustment, fHV	0.971		0.966		0.980	
Driver population factor, fP	1.00		1.00		1.00	
Flow rate, vp	8063		367		2426	pcph

Estimation of V12 Diverge Areas

L_{EQ} = 0.00 (Equation 25-8 or 25-9)

P_{FD} = 0.436 Using Equation 8

$v_{12} = v_R + (v_F - v_R)\, P_{FD}$ = 3722 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{Fi} = v_F$	8063	9000	No
v_{12}	3722	4400	No
$v_{FO} = v_F - v_R$	7696	9000	No
v_R	367	2000	No

Level of Service Determination (if not F)

Density, $D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D = 31.8$ pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable,	$D_S = 0.461$	
Space mean speed in ramp influence area,	$S_R = 49$	mph
Space mean speed in outer lanes,	$S_0 = 55.8$	mph
Space mean speed for all vehicles,	$S = 52.4$	mph

Table 9-14 LOS worksheet for Section 4 in Scenario 2

```
_____________________________Freeway Data_____________________________

Type of analysis                           Diverge
Number of lanes in freeway                 4
Free-flow speed on freeway                 55.0           mph
Volume on freeway                          7260           vph

_____________________________Off-ramp Data____________________________

Side of freeway                            Right
Number of lanes in ramp                    2
Free-Flow speed on-ramp                    35.0           mph
Volume on-ramp                             2331           vph
Length of first accel/decel lane           140            ft
Length of second accel/decel lane          140            ft

__________________Adjacent Ramp Data (if one exists)__________________

Does adjacent ramp exist?                  Yes
Volume on adjacent ramp                    255            vph
Position of adjacent ramp                  Upstream
Type of adjacent ramp                      Off
Distance to adjacent ramp                  1619           ft

____________Conversion to pc/h Under Base Conditions__________________

Junction Components                   Freeway      Ramp        Adjacent
                                                               Ramp
Volume, V (vph)                       7260         2331        255       vph
Peak-hour factor, PHF                 0.96         0.98        0.72
Peak 15-min volume, v15               1891         595         89        v
Trucks and buses                      6            4           7         %
Recreational vehicles                 0            0           0         %
Terrain type:                         Level        Level       Level
     Grade                            0.00     %   0.00     %  0.00    %
     Length                           0.00     mi  0.00     mi 0.00    mi
Trucks and buses PCE, ET              1.5          1.5         1.5
Recreational vehicle PCE, ER          1.2          1.2         1.2
Heavy vehicle adjustment, fHV         0.971        0.980       0.966
Driver population factor, fP          1.00         1.00        1.00
Flow rate, vp                         7789         2426        367       pcph

__________________Estimation of V12 Diverge Areas_____________________

                  L   =    0.00    (Equation 25-8 or 25-9)
                   EQ
                  P   =    0.260   Using Equation  0
                   FD
                  v   = v  + (v - v ) P  =  3820   pc/h
                   12   R     F   R  FD

______________________Capacity Checks_________________________________

                          Actual        Maximum         LOS F?
     v  = v               7789          9000            No
      Fi   F
     v                    3820          4400            No
      12
     v  = v - v           5363          9000            No
      FO   F   R
     v                    2426          3800            No
      R
```

Level of Service Determination (if not F)

Density, $D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$ = 33.3 pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable, D_S = 0.646

Space mean speed in ramp influence area, S_R = 47 mph

Space mean speed in outer lanes, S_0 = 56.5 mph

Space mean speed for all vehicles, S = 51.2 mph

Table 9-15 LOS worksheet for Section 5 in Scenario 2

Freeway Data

Type of analysis	Diverge	
Number of lanes in freeway	4	
Free-flow speed on freeway	55.0	mph
Volume on freeway	7260	vph

Off-ramp Data

Side of freeway	Right	
Number of lanes in ramp	2	
Free-Flow speed on-ramp	35.0	mph
Volume on-ramp	2331	vph
Length of first accel/decel lane	140	ft
Length of second accel/decel lane	140	ft

Adjacent Ramp Data (if one exists)

Does adjacent ramp exist?	Yes	
Volume on adjacent ramp	255	vph
Position of adjacent ramp	Upstream	
Type of adjacent ramp	Off	
Distance to adjacent ramp	1619	ft

Conversion to pc/h Under Base Conditions

Junction Components	Freeway		Ramp		Adjacent Ramp	
Volume, V (vph)	7260		2331		255	vph
Peak-hour factor, PHF	0.96		0.98		0.72	
Peak 15-min volume, v15	1891		595		89	v
Trucks and buses	6		4		7	%
Recreational vehicles	0		0		0	%
Terrain type:	Level		Level		Level	
Grade	0.00	%	0.00	%	0.00	%
Length	0.00	mi	0.00	mi	0.00	mi
Trucks and buses PCE, ET	1.5		1.5		1.5	
Recreational vehicle PCE, ER	1.2		1.2		1.2	
Heavy vehicle adjustment, fHV	0.971		0.980		0.966	
Driver population factor, fP	1.00		1.00		1.00	
Flow rate, vp	7789		2426		367	pcph

Estimation of V12 Diverge Areas

L_{EQ} = 0.00 (Equation 25-8 or 25-9)

P_{FD} = 0.260 Using Equation 0

$v_{12} = v_R + (v_F - v_R) P_{FD}$ = 3820 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{Fi} = v_F$	7789	9000	No
v_{12}	3820	4400	No
$v_{FO} = v_F - v_R$	5363	9000	No
v_R	2426	3800	No

Level of Service Determination (if not F)

Density, $D_R = 4.252 + 0.0086\, v_{12} - 0.009\, L_D$ = 33.3 pc/mi/ln

Level of service for ramp-freeway junction areas of influence D

Speed Estimation

Intermediate speed variable, D_S = 0.646

Space mean speed in ramp influence area, S_R = 47 mph

Space mean speed in outer lanes, S_0 = 56.5 mph

Space mean speed for all vehicles, S = 51.2 mph