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THE UNIVERSITY OF TEXAS AT AUSTIN

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TRAFFIC OPERATIONS AND SAFETY BENEFITS OF ACTIVE TRAFFIC STRATEGIES ON TXDOT FREEWAYS

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Traffic congestion is an increasing problem in the nation's urban areas, leading to personal inconvenience, increased pollution, hampered economic productivity, and reduced quality of life. While traffic congestion tends to continuously increase, growth in transportation infrastructure is limited by financial and land availability constraints. This has placed an increasing emphasis on using active traffic management strategies (ATM), such as speed harmonization, peak-period shoulder use, and ramp metering, to efficiently manage congestion using existing freeway capacity. Safety implication of these strategies is of prime concern before they can be implemented on the ground. This project developed a series of interdependent models and a simulation framework to evaluate the traffic operations and safety benefits of ATM strategies. Four ATM scenarios were evaluated in this study: variable speed limits (VSL), peak-period shoulder use, VSL and shoulder use, and ramp metering. Overall these ATM strategies were found to homogenize traffic and create safer driving conditions, but did not increase the throughput of the freeway. The study calls for caution and comprehensive evaluation in the case of shoulder use as sudden one-lane drop at the end of the shoulder-use section may have adverse effect on traffic operations and safety. The ITS devices required to implement these strategies, enforcement issues, potential impediments in their implementations, and a framework for cost-benefit analysis to determine the economic viability are also discussed.</td></tr>
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Chapter 1. Introduction

1.1 Motivation

While traffic congestion tends to continuously increase, growth in transportation infrastructure is limited by financial and land availability constraints, especially in urban areas. This has lead to the use of Intelligent Transportation Systems (ITS) to manage existing transportation systems efficiently, including Active Traffic Management (ATM) strategies, which manage the transportation system by responding to prevailing road, traffic, and weather conditions in real time, in order to increase safety and operational reliability.

Many ATM strategies are relatively new in the United States, and public agencies have little guidance to suggest how, when, and where, ATM should be employed to *maximize benefits both to individual corridors and to the entire transportation system*. Innovative strategies continue to be developed, suggesting that *evaluation frameworks are needed both for existing strategies as well as for new methods* that may be developed in the future. A natural pattern for determining the effects of ATM is to trace the impacts from design decisions to the key measures of effectiveness, through the following four steps:

1. **Design and Geometry**. As ATM strategies are not familiar to all drivers, a standardized design is needed to ensure comprehension, including pavement markings, static signs, and language used on dynamic signs.
2. **Behavior and Compliance**. Drivers respond to ATM according to the information presented to them (that is, based on the design and geometry) and to their own motivations (such as a desire to arrive at the destination quickly). By quantifying these motivations along with the role of enforcement on compliance rates, the true effects of ATM can be more accurately predicted.
3. **Operations and Congestion**. The collective behavior of drivers in response to ATM determines the operational state of the traffic stream after implementation, including any changes in volume, travel speed, speed differentials, and merging frequency or bottleneck locations. These changes in system state are critical for determining step 4.
4. **Safety and Reliability**. Changes in traffic operations will manifest as changes in incident frequency and severity, travel reliability, and other key measures of effectiveness that determines the overall success of an ATM strategy.

The primary innovation described here is the collective examination of quantified models within each of the aforementioned areas. This is novel but critical since the relationship between agency decisions (design and geometry) and final outcomes (safety and reliability) is mediated through driver behavior and operational traffic flow relations which must be rigorously accounted for to construct transferable analysis as well as general guidelines, especially where innovative ATM strategies are considered.

Finally, the effects must be measured at both the corridor and network-level scales in order to ensure that benefits to a facility where ATM is implemented are not outweighed by detriments elsewhere, and an economic cost-benefit analysis must be applied to synthesize the safety, reliability, and other impacts into a comprehensive assessment.

To this end, we developed a comprehensive framework to evaluate their traffic operations and safety impacts on Texas freeways. In particular, we implemented efficient control algorithms for these ATM strategies, presented a multi-resolution simulation framework to evaluate their network-level effects, evaluated traffic operations and safety benefits of the ATM strategies through the development of multiple interdependent models, and made recommendations on the ITS devices requirement and enforcement. We also discussed potential impediments in their implementations. A cost-benefit framework has been presented to determine economic viability of these strategies. We have summarized these crucial steps in a comprehensive operational deployment plan. These components make up a research framework (see Figure 1.1) that can be utilized by the concerned agencies in their decision-making process by evaluate the traffic operations and safety implications of ATM strategies.

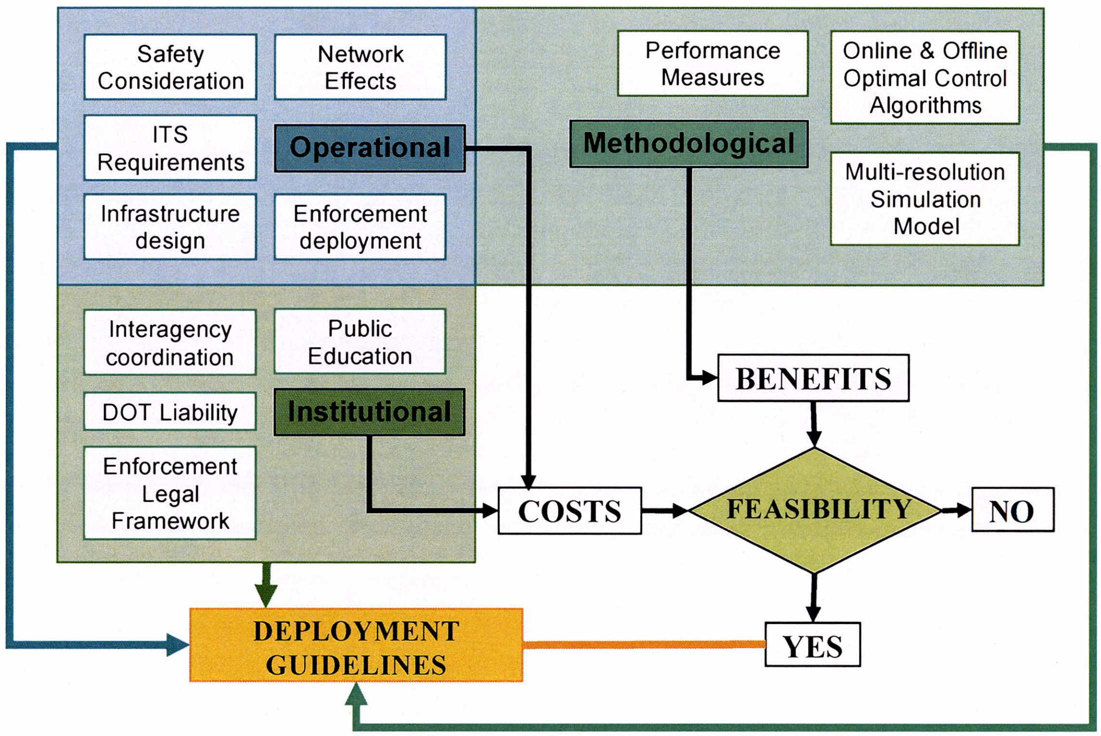


Figure 1.1: *Research framework*

1.2 Outline of the Report

The remainder of this report is as follows. In Chapter 2 we present a collection of representative past experiences with active traffic management strategies. Chapter 3 describes development of a simulation-modeling framework to evaluate the traffic operations and safety benefits of ATM strategies. Development and calibration of a simulation testbed for the purpose of this study is in Chapter 4. Chapters 5 and 6 provide a detailed analysis of traffic operations and safety benefits, respectively, of the ATM strategies through the use of the multiple interdependent models developed for this purpose. Recommendations on ITS and enforcement issues and a discussion of potential impediments are provided in Chapter 7. Chapter 8 presents a comprehensive feasibility analysis framework and it also discusses an operational deployment plan. Finally, Chapter 9 provides concluding remarks.

Chapter 2. Past Experiences with Active Traffic Management (ATM) Strategies

2.1 Speed Harmonization

Speed harmonization has been implemented in the past with various objectives, including postponement or prevention of the onset of congestion (United Kingdom), safety improvement (Germany), homogenization of traffic speeds in space and time (the Netherlands), freeway throughput augmentation, and pollution reduction. Previous experiences indicate that the strategy has been reasonably successful in achieving its objectives. For example, a case study in Germany found a 14% to 37% lower injury accident rate per vehicle-km in controlled highways (Metz et. al., 1997). The implementation of speed harmonization reduces the speed differential between and within lanes, and creates a more uniform and acceptable headway distribution thus reducing the potential for the occurrence of primary accidents (Lee et al., 2006). Moreover, speed harmonization leads to more stable traffic flow and reliable travel times, and the decline in the amount of stop-and-go traffic can result in significant air quality benefits (Wesseling et. al., 2003).

Successful implementation of speed harmonization typically requires a relatively dense ITS deployment, and efficient enforcement policies. Additionally, the development of appropriate control strategies for the selection of speed limits plays a fundamental role in the effectiveness of speed harmonization. The following sections describe some of the past experiences with speed harmonization implementation, and discuss issues related to the selection of control strategies.

2.1.1 Weather-controlled Speed Limits (Finland)

The objective of this study (Rama, 1999) was to investigate the effects of weather controlled speed limits on the mean speed and average headways. More specifically, it was aimed to examine whether such speed limits contribute to increased traffic safety. The speed limits were displayed using variable message signs (VMS); speed and headway data were collected using loop detectors.

Interesting outcomes of the study include:

- An increase in the speed limits during favorable road and weather conditions increased the mean speed levels.
- A decrease in the speed limits during adverse weather conditions reduced the mean speed and increased the average headway, which is desirable for traffic safety.
- The mean speed under favorable roadway conditions was reduced, while it was increased during normal and adverse road and weather conditions. This latter can be attributed to lack of driver education: people tend to drive slowly during poor weather conditions, but because of a lack of understanding of the displayed speed limits – drivers thought of them as recommended values rather than maximum values – people will end up driving faster than average.
- Speed and friction measurements indicated that the observed values were in agreement with the ones prescribed on the VMS in 76% of the cases

• Around 96% of the drivers considered variable speed limits (VSL) based on real-time weather and road conditions to be useful.
• People thought that VMS are easier to notice than conventional static signs.
• Most drivers were not aware that the VMS were displayed based on a control strategy. If they knew this information, there might be some potential to improve the effect of VSL.

2.1.2 M25 Motorway (UK)

The primary objective of a 1995 speed harmonization project on the M25 Motorway (Figure 2.1) was congestion management. Additional objectives were the creation of a more comfortable driving experience and the reduction of fuel consumption. The motorway was instrumented with dual loop detectors spaced every 500 m (0.3 miles) that provided speed, volume, and occupancy data. When volumes reached 1,650 vehicles per hour per lane (vphpl), the speed limit was reduced from the default value of 70 mph to 60 mph. When volumes reached 2050 vphpl, the speed limit is further reduced to 50 mph. Among the observed benefits were the following:

• Reduction of the number of collisions by over 10%.
• During weekdays, travel times were reduced in one direction of the motorway.
• The controlled motorway has contributed to a more reliable journey time.
• A uniform distribution of traffic across all the four lanes was observed.
• A uniform headway distribution was observed.
• Reduced emission and noise levels.
• More comfortable driving experience.

As noted above, travel times in only one direction were observed to decrease. In fact, travel times in the other direction were increased. This can be partly attributed to the lesser amount of flow in this latter direction. Travel times were also found to increase in the off-peak periods. However, this is not due to speed limit reductions, but rather to the stricter enforcement measures that were necessary for compliance (Harbord and Jones, 1996).

Figure 2.1: *Speed harmonization on M25, UK (Warren, 2000)*

Based on initial studies it was found that implementation using traditional message signs that were manually changed by the police didn't provide the desired results. Thus this pilot used a more responsive system, automatically controlled and with mandatory speed limits. The enforcement system was also automatic, using wet-film cameras to take pictures of vehicles. Speed control was provided with the aid of MIDAS (Motorway Incident Detection and Automatic Signaling) system. Speeds ranging from 20 to 60 mph were displayed. A red ring was used to indicate that the speed limits were mandatory. Statistical information provided by MIDAS was used to detect areas where enforcement could be more beneficial. A pre-set delay between the new speed limit and its enforcement was maintained, to allow for a safe change of speed (Harbord, 1998).

Detectors were located every 500 meters in all lanes. Data was processed by roadside stations to detect queues and slow moving traffic. Speed control was exercised when flow break down was about to occur. The initial implementation utilized fixed speed limits selected based on the time of the day. Later on, a dynamic control scheme that was based on simple flow thresholds sets the speed limit to be either 50 mph or 60 mph. Since the system was based on flow values, it was unable to detect when flow was reduced due to very low speeds, which caused the system to display high speed limits when traffic was stopped. It was considered to turn off the control system when congestion was set, but drivers indicated that they preferred the limits to continue to be displayed.

During the course of the pilot, a new feature to protect drivers at the end of a queue was added. The HIOCC (high occupancy) algorithm was implemented to detect queues and slow moving traffic. The system typically set speed limits of 40 mph on the section immediately prior to the end of the queue (and 50-60 mph prior to this 40 mph section).

Important observations from this study were:

- Speed measurement using some other equipment, e.g. laser, was recommended instead of radar technology.
- Analysis of data suggested that drivers did not alter vehicle speeds based on suggested speeds; it had to be enforced. After enforcement, compliance was very high.
- The number of drivers exceeding the speed limit diminished by 50%. However, it was noticed that if traffic limits were posted more than 1km apart (such that only one speed limit is visible at a time), drivers were found to speed between the gantries.
- Less lane changing was observed. Drivers did not see the point of changing lanes when everyone was driving at the same speed.
- Flow in the slow lane increased by 15%. More uniform and less extreme headways were observed.
- Injury accidents were reduced by 28%. This result was significant at a 95% confidence level, but it might have been influenced by roadwork.
- 60% of the drivers were happy with the system, and the fact that they could drive at a constant speed without worrying about changing lanes made them more comfortable.
- Travel time reduction could not be assessed with statistical significance. However, data suggested that the travel times were improved and their variability reduced.
- A 5% increase in traffic demand was observed during the pilot project, which was accommodated without increasing congestion.

2.1.3 A2 Motorway (The Netherlands)

The primary objective of the project was to reduce inefficiencies in lane utilization and speed differentials between lanes (Smulders 1990, 1992). A control strategy was developed to homogenize traffic flow by encouraging more uniform lane usage and less speed differential between lanes. Based on speed and volume data collected at dual loop detector stations every 500 m, the displayed speed limit could potentially be reduced to 90 km/h or 70 km/h from the standard speed limit of 120 km/h. Speed limits were only changed when volumes approached capacity. The choice of the speed limit is made every minute based on measurements of the average traffic speed on the section under study. Goal is to keep the difference between the average speed and the limit speed as small as possible. The objective of the system was not to reduce average speeds, but to reduce speed differences within and between lanes.

Important observations include:

- Over 1,300 drivers were asked how they experienced the system and whether it had influenced their driving behavior. A large majority said they had adjusted their behavior due to the speed control measure.
- A large majority said they had benefited from the measure. Among the benefits cited were improved traffic flow and a less hectic driving experience. Furthermore, VSL had a warning effect regarding congestion and unsafe situations.
- One in five respondents was unfamiliar with the purpose of the speed signaling devices. Awareness of these devices has been found to have a positive impact on the extent to which drivers adjust to the new speed limit.
- Analyses showed that VSL created more homogeneous traffic: less speed variations, less small headways and fewer shockwaves. Homogeneity was also achieved across lanes.
- The average speed on the motorway dropped while the average occupancy increased.
- No positive effect on the capacity could be demonstrated.

A second evaluation performed six months after the start of the experiment showed that the effects of the speed control had decreased to some extent, but still remained positive.

2.1.4 State of Washington (USA)

In Ulfarsson et al. (2005) speed harmonization was introduced to address the significant variations in speed due to the combined effects of vehicle mix, inclement weather and challenging road geometrics. Based on environmental data and pavement conditions, speed limits were reduced from 105 km/h to as low as 56 km/h in 16 km/h decrements. Under conditions of high speed and low speed variations, it was found that the mean speed was reduced, while variations were more prevalent. On the other hand, under conditions of low speed and high-speed variations, both mean speeds and speed variations were reduced. Hence this experiment shows that only under certain traffic conditions, speed harmonization gives optimal effects.

2.1.5 Albuquerque, New Mexico (USA)

Speed harmonization was implemented on I-40 (eastbound) in Albuquerque, New Mexico in 1989, with the main goal of minimizing accident risk and to inform motorists of downstream hazards. Traffic data were collected from inductive loop detectors placed in each lane perpendicular to the roadside station equipment, at an average spacing of 1.5 miles. Loop data was collected every 10 seconds and processed to calculate speed, volume, length of vehicle and standard deviation of speed. A slight reduction in accident rates was reported (FHWA, 2000).

2.1.6 VSL in Work Zones, Michigan (USA)

The VSL system was deployed during the summer of 2002 in a work zone on I-96, south and west of Lansing, Michigan (FHWA, 2004). Prior to the deployment of the system on the actual site, it was tested on a local route. The following interactions between various institutions were found necessary during various stages of the project:

- The Department of Transportation and the State Police Department.
- The Department of Transportation and the Work Safe Department.
- The Department of Transportation and the Michigan State University.

The Michigan Department of Transportation and the Michigan State Police have the legal authority to set speed limits in work zones within the state. According to the Michigan Vehicle Code, "a person operating a vehicle on a highway, when entering and passing through a designated work area where a normal lane or part of the lane of traffic has been closed due to highway construction, maintenance, or surveying activities, shall not exceed a speed of 45 miles per hour unless otherwise determined and posted by the state transportation department, a county road commission, or a local authority."

Two operational difficulties faced by the project personnel during the deployment of the project were: 1) Pneumatic tubes installed during the project were often ripped off by the traffic, leading to loss of data. (2) There were communication problems between successive deployment sites and the remote access of these sites.

Among the conclusions of the project were:

- Increased average speeds;
- Reduction in travel times, although the reduction was not significant enough to be perceived by the drivers;
- Less drivers violated the speed limit;
- There was a positive response from the drivers to the speed limit changes;
- The installation of the VSL system did not cause any additional safety issues. Whether it increased the safety was not clear.
- It was concluded that VSL might have more utility in longer and simpler work zones. The reason for this is that in shorter and hectic work zones the flow of traffic would be dominated more by factors such as road geometrics.
- System technology needs to be improved before it can be widely used.

2.1.7 Experiments in Utrecht and Rotterdam (The Netherlands)

The type of control strategy used was a simple homogenizing strategy developed by Van Toorenburg (1983). With this type of control an identical advisory speed limit is displayed for all lanes at a given set of consecutive signal stations at the same time (see Figure 2.2). The speed value is chosen out of a finite set and is in correspondence with the actual speed of the traffic stream. In almost all cases, this will lead to a value of 90 km/h, as this is approximately the mean

speed when traffic reaches the road capacity. In some cases 80 or 70 km/h will be displayed. The only parameter left to optimize in this type of control is the time to change speed limits.
The motivation for this type of control is that congestion is caused by severe inhomogeneity of the traffic stream, which exist when the traffic volume approaches the capacity of the road (See Van Toorenburg, 1983). It was concluded that exercising homogenizing control is advantageous in that it increases safety and reduces the probability of congestion.

The conclusions from the field experiments were:

- The instability of traffic flow – measured as the number of serious speed drops - significantly decreased with homogenizing control. The decrease that was measured amounted to about 50%.
- A small increase in the capacity was observed – 1 to 2 %.
- No significant effects were measured in other traffic characteristics such as mean speed, speed differences, and distribution over lanes.
- No serious implementation problems were observed indicating the relative ease of use and robustness of the control system.

Figure 2.2: *Speed harmonization in the Netherlands (Warren, 2000)*

2.1.8 Environmental Benefits of VSL—Austin, Texas (USA)

The motivation of this study is to reduce emissions. In a recent study by Wang and Walton (2003), it was found that freeways and expressways traffic could generate over 40 percent of NO_x in a metropolitan area such as Austin, Texas. It is widely recognized that high speeds usually cause high vehicular emissions. These high speeds are usually experienced during the off-peak hours. Thus by controlling the speed during these off peak hours, emission reductions can be realized. The static speed limit on the IH-35 in Austin is 65mph. In this experiment, the speed limit is reduced to 55mph on certain "Ozone Action days".

Important conclusions from the experiment were:

- During off-peak hours, lowering the traffic speed through VSL leads to lower NOx emissions. Given the large contribution from freeway/expressway traffic to NOx emissions, a VSL strategy can be an effective measure to reduce NOx. By reducing the speed limit from 65 mph to 55 mph on "Ozone Action days", the average daily total NOx emission in a 24-hr period can be reduced by approximately 17 % on the selected IH-35 segment.

- Traffic flow and speed patterns are primary factors affecting the effectiveness of a VSL strategy. Before the deployment of a VSL, the flow and speed patterns of the selected freeway/expressway should be carefully investigated.

Compared to fixed speed limits, the VSL strategy can be a promising way to balance travelers' need for mobility and the conservation of the environment.

2.2 Peak-period Shoulder Lane Use

According to the FHWA, 40% of the congestion in the United States is a result of insufficient capacity. Peak-period shoulder lane usage, also referred to as dynamic shoulder usage, is the temporary operation of hard shoulders as running lanes for normal traffic. It provides additional capacity when needed without major infrastructure expansion requirements. Previous experiences, such as the one described by Middelham (2003), suggest that this dynamic lane management strategy can be extremely beneficial in alleviating congestion, as long as it is implemented in conjunction with appropriate measures to avoid deterioration in the overall highway safety. In effect, hard shoulders are usually narrower, and their use as a temporary running lane detracts from their function as a safety lane. As a consequence, measures including lower speed limits during shoulder operation, restrictions on the type of vehicles allowed into the shoulder and on overtaking are usually enforced.

Even though the increased capacity provided by shoulder usage has the potential to alleviate congestion, the additional discharges from intervened sections may result in an overall deterioration of the network performance. This calls for a careful analysis of the network effects of peak-period shoulder usage before deployment. Cohen (2004) describes a peak-period shoulder implementation focused on the removal of a bottleneck. The project achieved its goal, resulting in a 16% capacity improvement, and a 25% speed increase in the intervened section. Nevertheless, the additional flow discharges generated by the congestion removal led to worsened congestion downstream. This translated into increased travel times and recurring congestion between many origin-destination pairs in the corridor.

As for speed harmonization, peak-period shoulder usage techniques tend to require relatively intense ITS technology deployment. In the case of peak-period shoulder use, these are also used to discard the presence of detained vehicles, pedestrians, or dangerous debris in the shoulder before opening it to the traffic, and to ensure adequate safety conditions. Next we present a representative sample of past experiences with dynamic shoulder use.

2.2.1 Temporary Hard Shoulder on A5, Hessen (Germany)

The federal state of Hessen, Germany, implemented shoulder lane use, as well as other traffic management strategies, as part of their integrated intelligent transportation system. Traffic Center Hessen controls the temporary use of hard shoulders. Traffic volumes are monitored, and shoulder lanes become accessible when a certain threshold is crossed. Approximately 80 video cameras are used to monitor the shoulder lane to check for obstructions.

The results were positive, as congestion was greatly reduced, and road safety suffered no negative changes:

- Improved traffic flow;
- Significant accident and congestion reduction;
- Capacity increase by 20%;

Because of these positive results, the state of Hessen plans to implement shoulder usage for congestion mitigation in more corridors in the future (Riegelhuth and Pilz, 2007).

2.2.2 A3-A86 Junction, Paris (France)

Cohen (2004) details the results of an experiment using the shoulder to increase the number of lanes (the hard shoulder was used to create a fifth lane in each direction). The experiment was implemented on the A3-A86 joint section, in the Seine-Saint-Denis department north of Paris. It has to be noted that the change was not temporary but permanent. The system was controlled by the local Traffic Control Centre, and supervised 24 hours a day.

In order to deal with the induced safety issues, dynamic equipment was installed, including:

- Emergency call boxes;
- Variable Message Signs;
- Automatic Incident Detection.

The impacts on capacity, speed and travel times were examined. In order to measure the impact on capacity, a measurement station was installed in the middle of the section with double loops providing the traffic flow rate, speed and occupancy rate parameters. The data was gathered before and after opening of the shoulder to regular traffic. The resulting capacity is shown in Table 2.1.

Table 2.1: Roadway capacity before and after opening of the shoulder lane to traffic

Lane	Capacity (veh/h)			
	Towards Paris		Towards Province	
	before	after	before	after
Right 1	2020	1335	2265	1865
Lane 2	2100	1635	1645	1750
Lane 3	1840	2040	2005	2000
Lane 4	2220	1755	2500	2035
Left 5		2390		2395
Section	*7890*	*8550*	*8100*	*9170*

The findings were that while capacity was increased, free flow speed did not change, and speeds during congested periods changed dramatically for certain origin-destination pairs. Similar results were obtained for the travel times. In fact, it was observed that for both the A3 and the A86 corridors there was increased congestion in at least one direction. With the opening of the hard shoulder, the bottleneck seemed to have moved.

2.2.3 M4, M25, and M42 (UK)

The British Highways Agency has developed an active traffic management (ATM) system, similar to the Dutch concept of dynamic traffic management, and is implementing it on a 16-km (10-mi) stretch of the M42 east of Birmingham in the West Midlands (Figure 2.3). While Britain has used variable speed control signs since 1964 and has been monitoring speeds and detecting incidents with its MIDAS system, closed-circuit cameras, and Trafficmaster™ APNR (photo billing and enforcement) systems for many years, the M42 combined these with new measures.

These innovations included use of the hard shoulder, as is done in the Netherlands, and new rapid response incident management practices borrowed from the United States. The purpose of the ATM pilot was to create more reliable travel times and congestion reduction, by providing drivers more and better traffic information and by responding more quickly to incidents.

Enforcement was realized via ANPR. Furthermore, the Highways Agency traffic officers also had the power to stop traffic, close roads, direct traffic and enforce laws. It was concluded that travel times were shorter, accident and emission rates declined, speed compliance increased. Moreover, drivers' reactions were positive.

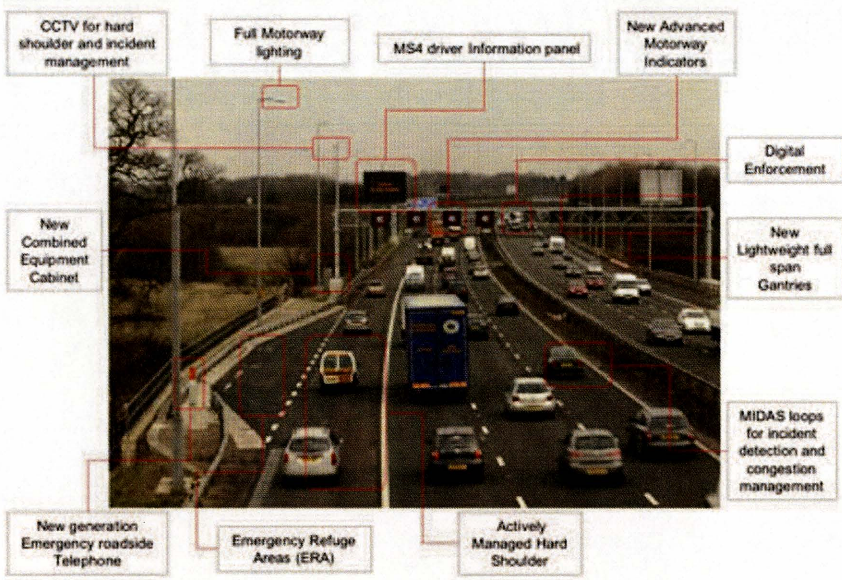


Figure 2.3: *Active traffic management on M42*

The Royal Society for the Prevention of Accidents (RoSPA, 2004), in a response to the highway agency statutory instrument consultation pack, reported on some issues resulting from the implementations of shoulder use employed in the past that would be relevant to the implementation on M42. The report mentions that early implementations in the US resulted in a higher number of collisions, but that sophisticated management strategies reduced this number. It also mentions that the opening of the shoulder lane is done regularly in the US during peak hours, whereas in the Netherlands traffic management centers only resort to this DTM strategy in case of heavy congestion. In the seven locations where it has been implemented in the Netherlands, emergency refuge areas have been implemented every 500m; they are equipped with MIDAS and phone services. Further, speed limits are strictly enforced and overtaking is restricted.

The main legal issue discussed in the report is the issue of keeping users from traveling in the hard shoulder beyond the specified time or space intervals.

Gaskell et al. (2004) describes the use of hard shoulders as an additional running lane when incidents occur or during recurring congestion. The study focuses its concerns on safety and identifies the following as key elements to guarantee safety:

- Provision of emergency refuge areas (ERAs) at 500m intervals
- Message signs at approximately every 500m to provide clear instructions to drivers. i.e. speed limits and so on
- Enforcement of low speed limits (50mph on all lanes)
- Refined monitoring system, involving loop detectors every 100m.

These monitoring systems are necessary since traffic operators need to be able to detect obstructions and incidents on the shoulder (e.g. pedestrians, debris and slow moving cars) before the opening of it as a running lane. Once the shoulder is opened to regular traffic, the main concern becomes the detection of incidents. The authors proposed to monitor via two types of sensors: loops (short range, high accuracy) and cameras (medium range and accuracy). The optimal spacing and location need to be determined and data fusion techniques should be developed to merge the information provided by the two sensor types.

2.2.4 DTM in the Netherlands

Middelham (2003) claims that the use of shoulder lanes as rush hour lanes is justified since the technical characteristics of vehicles have improved tremendously. As such, vehicle breakdowns are less likely and hard shoulders are rarely used. In order to implement such as a scheme, the author recognizes the need for video monitoring, variable speed limits, and escape points. In three pilot test sites, the capacity increased by more than 50%; at the same time safety was increased as well. Guidelines for the layout of the control equipment have been developed (Middelham, 2003). More than 500 km of roadway are expected to be equipped with this equipment in the coming years.

2.2.5 Autostrada del Brennero - Highway A22 (Italy)

Bergmeister et al. (2006) details a project in which an emergency lane is used to expand a two-lane-facility when traffic demand exceeds or is near capacity. It is of particular importance because it is being implemented on a 125km stretch, which is the longest to date. The plan includes adjusting structures (e.g. including bridges, overpasses and viaducts) to allow for the continuous running of the shoulder lane. The local Traffic Control Centre in charge of handling the managed lane gathers information from:

- Traffic Sensors: 20 induction-loop stations;
- Cameras: 41 video cameras;
- Weather sensors: 14 sensors measuring wind, rain, ice and visibility.

The VMS is used to control the speed of traffic, warn about congestion or treacherous conditions, direct traffic in the case of accidents, and deviate traffic for other reasons.

The biggest concern in this project was the management and control of transition conditions, i.e. opening and closing of the dynamic lane. In order to appropriately monitor the system and to determine when to open (close) the dynamic lane, gantries were located in 5km intervals, containing detection and monitoring systems, variable message signs and infraction detection devices. Cameras were located every kilometer to check roadway conditions before the opening (and after the closing) of the extra lane. Both the opening and closing of the dynamic lane is done on a per stretch basis. The speed of the traffic on adjacent lanes determines the opening (closing) of the emergency lane.

2.2.6 Hard Shoulders Usage in Germany

Kellermann (2000) examines the steps necessary to open hard shoulders to traffic and the conditions under which such measure is justifiable. General results suggest that opening shoulders has positive impacts on traffic quality and congestion. Safety concerns were addressed by implementing complementary measures, such as speed limits and other restrictions. The study identified the main function of hard shoulders as:

- Area to leave damaged vehicles;
- Lateral space to use for avoidance maneuvers if unexpected obstacles appear on the road;
- Temporary traffic road if there are accidents on any of the main lane;
- A place for maintenance crews to set up in order to perform work and winter maintenance;
- Place were emergency vehicles can run and where vehicles can be towed away.

In Germany, accident rates on roads without hard shoulders were found to be 25% higher than on those with hard shoulders. In general, opening hard shoulders to regular traffic should be considered as a temporary measure, unless speed limits can be enforced and refuge areas can be provided. The legal requirements depend on the type of implementation. If the hard shoulder is used temporarily, legislation regarding the use of the continuous white line will be required. Potential physical requirements identified by the study include:

- Shoulder reinforcement;
- Acceleration and deceleration lanes;
- Emergency stopping areas need to be provided;
- Drainage and slope requirements must be met.

The experiment showed that for permanently opened shoulder lanes a significant decrease in congestion (68-82%) was realized, as well as an increase in average speed (9%). As for safety, it is concluded that an emergency stopping lane is much more effective in reducing accidents than refuge areas.

For temporarily opened shoulder lanes, the number of accidents was reduced by half, if approach and exit sections were considered in addition to the highway segment. The temporary opening of shoulder lanes at pre-specified hours or at times of heavy congestion did not alter the overall accident rate: the number of congestion induced accidents decreased, but the number of lane changing accidents increased at the same time. Finally, the road maintenance cost increased because of the time limitations imposed by shoulder use on maintenance crews (maintenance crews could not operate during certain times of the day).

2.3 Ramp Metering and Junction Control

Ramp metering is the use of traffic signals at freeway on-ramps to control the rate of vehicles entering the freeway. The metering rate is set to optimize freeway flow and minimize congestion. The metering rate can be fixed, or responsive to local or system-wide conditions. Signal timing algorithms and real-time data from mainline loop detectors are often used for more effective results.

Ramp meters and other forms of onramp control have been used as an ATM strategy for several decades, dating to a 1963 installation on Chicago's Eisenhower Expressway. At present, ramp metering is used in dozens of cities in North America (including Columbus, Denver, Houston, Los Angeles, Minneapolis-St. Paul, New York City, and Seattle) and throughout the world (Australia, Germany, Japan, the Netherlands, New Zealand, South Africa, and the United Kingdom have all installed ramp meters) (Piotrowicz and Robinson, 1995).

The fundamental principle of ramp metering is to improve freeway flow by regulating merging traffic from onramps, typically in the form of an automated traffic signal requiring all merging vehicles to stop for several seconds. A typical ramp meter configuration is seen in Figure 2.4. This has two positive effects on the freeway traffic stream: first, platoons of merging vehicles are broken up, minimizing the disruption to the freeway; and second, the total number of merging vehicles can be restricted. This latter impact manifests in two forms, as the absolute number of merging vehicles can be set by the metering rate, and as additional delay to merging vehicles causes some drivers to switch routes and avoid the freeway altogether (Levinson et al., 2005). In many cases, these impacts increase vehicle throughput, and decrease freeway travel times and accident rates (Taylor and Meldrum, 2000).

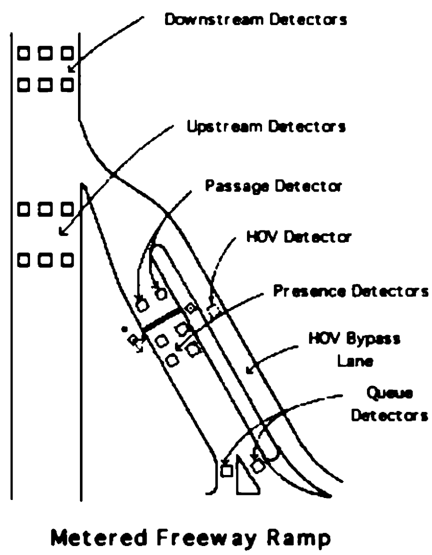


Figure 2.4: *Typical geometry for metered ramps*

(Source: California Center for Innovative Transportation)

However, the positive impact on freeway flow must be balanced against negative impacts caused by a large queue of vehicles growing on onramps. This can lead to increased perceived travel times on behalf of motorists, even as total travel time decreases (Levinson and Zhang, 2004; Levinson et al., 2005) and, in the case of very long queues, can disrupt arterial streets severely (Pearson et al., 2001). Some past experiences and researches with ramp metering impacts on safety are presented in following parts.

2.3.2 North American Survey in 1995 (USA)

A survey of traffic management centers using ramp metering reported that accidents on freeway systems under freeway management were reduced between 15 and 50 percent. While some other freeway improvements were implemented during the study periods, the combination of geometric, vehicle, and operational procedures resulted in significant reduction of accident rate.

2.3.3 Minneapolis-St. Paul freeways (USA)

A study performed for the Minnesota Department of Transportation (Mn/DOT) revealed the impacts of shutting down the extensive ramp metering system on Minneapolis-St. Paul area freeways for a 6-week evaluation period. The study analyzed data collected along 4 test corridors chosen to represent typical freeway configurations and conditions across the region. Results indicate that without ramp meters, there was:

- A 9% reduction in freeway volume and a 14% reduction in peak period throughput.
- A 22% decrease in freeway travel times with meters, which more than offsets the elimination of ramp delays. Meters result in an annual system-wide savings of 25,121 hours.
- A 7% reduction in freeway speeds.
- A 26% increase in crashes.
- A net annual increase in emissions of 1,160 tons without ramp metering.
- A decrease in fuel consumption without ramp metering of 5.5 million gallons. This was the only category where ramp metering had a negative impact.

Market research data collection results showed a number of changes in attitudes among area travelers that occurred once meters were shut down.

- Most survey respondents believed that traffic conditions worsened.
- Support for continued shutdown remained at 20%.

Analysis of the benefits and costs of the ramp metering system showed that when the costs of the entire congestion management system (including changeable message signs, traveler information, and other components) are factored in, the benefit/cost ratio for ramp metering is 5:1. When ramp meter benefits are compared to only those costs directly associated with ramp metering, the benefit/cost ratio is 15:1

2.3.4 Denver Metering System, Colorado (USA)

Initiated in the late 1970s, the Denver metering system started with five ramps on northbound I-25. Geometric improvements to bring acceleration lanes to standard length and improve interchange design were required. An early evaluation was performed during 1981 and 1982 with promising results.

- Speeds increased dramatically by 58%
- Vehicle hours of travel decreased by 37%
- Vehicle emissions dropped by 24%
- Accidents dropped by 5%

With metering, mainline flows exceeded 2450 vphpl on several occasions. Motorists shifted their arrival times to avoid ramp delays, and flows on area arterials increased from 100 to 400 vph, resulting in virtually no degradation of surface street conditions.

The Denver system was subsequently expanded to a centralized system with additional meters. A later evaluation suggested that central coordination was only beneficial when congested conditions (speeds less than 55 mph) existed. However, when speeds were near 55 mph, central coordination was of little benefit

2.3.5 Detroit, Michigan (USA)

Metering has been an important part of the Michigan DOT's Surveillance and Driver Information System (SCANDI). Metering was initiated in 1982 with six ramps on eastbound I-94, with many more ramps added later. Evaluation showed:

- Speeds increased by about 8%, even though volumes increased from 5600 vph to 6400 vph.
- The total number of accidents was reduced by nearly 50%
- The number of injury accidents dropped by 71%.

The evaluation also showed that significant additional benefits could be achieved by metering inter-freeway connectors to I-94.

2.3.6 Freeway Ramp Metering System in Arizona (USA)

A six-year study comparing accident frequency at nine ramp metering locations was conducted on the Superstition Freeway in Arizona. Three years of before and three years of after data were used to compare the change in the number of accidents both on the mainline and ramps. Ramp metering is only in operation during peak periods making it possible to also compare operations within periods of the day. Results indicate that:

- When ramp metering is in operation, rear-end and Sideswipe accidents were reduced 10% from the before case.
- When ramp metering was not in use accidents increased 33%.
- Accidents on the ramp increased in the after case due to traffic being required to stop on the ramp during metered periods.
- Accidents increased during periods when metering was not being used.

Overall, accidents increased 24% on both the mainline and ramps when metering was in effect, and 43% when metering was not in effect. These increases may be due do large increases in traffic volume between the two cases.

2.3.7 Portland, Oregon (USA)

In 1981 meters were installed along I-5, a major north-south link and important commuter route. Sixteen meters in fixed cycle operation were evaluated. With metering:

- Average northbound speeds increased from 16 to 41 mph
- As pre-metered conditions were less severe in the southbound direction, average speeds increased from 40 to 43 mph.
- Fuel consumption, including that caused by ramp delay, was reduced by 540 gallons per weekday.
- Overall there was approximately a 43% reduction in peak period accidents.

2.3.8 Seattle, Washington (USA)

Beginning in 1981, as part of the FLOW program, WDOT implemented metering on I-5 north of the Seattle CBD. A six-year evaluation consisted of seventeen southbound ramps during the AM peak and five northbound during the PM peak along a 6.9 mile test corridor. Over the study period:

- Travel time dropped from 22 minutes before metering to 11.5 minutes after
- Mainline volumes increased over 86% northbound and 62% southbound
- Accident rate dropped about 39%
- Average metering delays at each ramp remained at or below three minutes

2.3.9 Zoetemeer (Netherlands)

Initiated in 1989, nine ramp meters were in place by 1995. This evaluation focused on the A12 motorway between Utrecht and Hague. The road carried more than 110,000 vpd on weekdays, but became congested near Zoetemeer due to lane drops and weaving sections. For the 11 km study area, after using the ramp metering system, although ramp travel time increased by about 20 seconds, total system wide effects were positive:

- Bottleneck capacity increased by 3%
- Higher speeds during congested periods (from 46 to 53 kph)
- 13% shorter travel times (from 13.8 to 12.0 minutes)

2.3.10 M8 Motorway in Glasgow (Scotland)

In Glasgow, Scotland a freeway ramp metering system installed at an entrance ramp to the M8 motorway reduced the frequency of early merging by 29%. Glasgow is a city of approximately 625,000, within an urban area of 1.7 million people. The city is currently undertaking numerous projects to improve the safety of the roadway system. During the five-year period prior to the date of this study (1998), the city experienced a 52% reduction in the number of fatal accidents and a 19% reduction in injury accidents. Several other ITS projects are being undertaken in the area, including variable message sign assisted park and ride, traffic control strategies that assist public transportation at signalized intersections, and an integrated control strategy incorporating freeways and urban streets. The ramp-metering project evaluated is a part of the integrated control project. Findings from the study indicate that:

- The number of vehicles merging too early decreased after the implementation of ramp metering, from 35% to 25%.
- The amount of lane changing in the two lanes adjacent to the ramp appears to have dropped. Prior to implementation, the percentage of vehicles changing lanes in this area ranged from 6 to 15%, while after ramp metering began this number ranged from 1 to 8%.

2.3.11 Pilot Ramp Metering Projects (Germany)

Ramp metering is fairly new in Germany. Because geometric constraints on both freeways and surface streets provide limited storage, applications are limited as a whole. The concept was first tested in 1999 in five pilot projects on the A40, yielding positive results:

- Congestion decreased more than 50% during peak periods
- Traffic incidents at the ramps decreased 40%
- Average speeds on the A40 increased by more than 10 km/h during peak travel periods

As a result of these successful tests, more ramp metering systems are being installed across the country.

2.3.12 Ramp Metering Projects (UK)

The first pilot ramp-metering project in the United Kingdom was installed on the M6 near Birmingham in 1986. Its purpose was to reduce congestion at the ramp by limiting traffic entering the motorway from the ramp and to enhance traffic flow downstream of the junction. A detailed assessment of operations through data monitoring yielded promising results:

- On average, vehicle flow increased 5% at the implementation location
- Motorway speeds increased 14 to 18% at some locations
- No negative impacts were experienced from ramp meter queues spilling into the adjacent intersections
- Driver compliance was high.

As a result of the pilot's success, the system was updated and expanded to five additional locations on the M6, and another 30 are implemented on the M27 in 2007.

Besides those practical experiences, some researches also evaluated the influence of ramp-metering on safety.

2.3.13 Evaluation of Freeway-merging Safety as Influenced by Ramp-metering Control

In the research of Cima (1977), the traffic-conflict technique was modified to evaluate the relative safety of freeway merging with and without the use of entrance ramp-metering control. Six types of traffic conflicts were defined for the entrance ramp and acceleration lane:

- Braking on ramp
- Braking for lead vehicle
- Weaving around lead vehicle
- Entering second lane
- Entering side by side
- Entering late

Five conflicts were specified for the freeway lane (merge lane) adjacent to the acceleration lane:

- Weaving around entering vehicle
- Braking for entering vehicle
- Weaving around lead and entering vehicles
- Braking for lead entering vehicles
- Avoiding encroaching vehicles

A three-level severity rating (routine, moderate, and serious) was also developed to assess the seriousness of each conflict. An existing ramp-metering control installation was investigated during freeway levels of service C and D. A two-way analysis of variance was performed on the traffic-conflict data by using, as the independent variables, ramp-control condition (on and off) and freeway level of service (C and D).

The study revealed a significant reduction of 11.6 percent in all traffic conflicts when ramp control was activated. Analysis results indicate that acceleration-lane conflicts significantly decreased when ramp-metering control was used. Merge-lane conflicts were found to be related more to freeway level of service than to ramp control. However, merge-lane, multiple-vehicle conflicts and their severity decreased when ramp control was in effect.

2.3.14 Qualifying Effects of Ramp Metering on Freeway Safety

Lee et al. (2006) researched qualifying effects of ramp metering on freeway safety. They proposed a real-time crash prediction model and used this model to investigate the effect of the local traffic-responsive ramp metering strategy on freeway safety. Safety benefits of ramp metering are quantified in terms of the reduced crash potential estimated by the real-time crash prediction model. Driver responses to ramp metering and the consequent traffic flow changes were observed using a microscopic traffic simulation model and crash potential was estimated for a 14.8 km section of I-880 in Hayward, California and a hypothetical isolated on-ramp network. The results showed that ramp metering reduced crash potential by 5–37% compared to the no-control case. It was found that safety benefits of local ramp metering strategy were only restricted to the freeway sections in the vicinity of the ramp, and were highly dependent on the existing traffic conditions and the spatial extent over which the evaluation was conducted. The results provide some insight into how a local ramp metering strategy can be modified to improve safety (by reducing total crash potential) on longer stretch of freeways over a wide range of traffic conditions.

2.3.15 Ramp Metering Impacts on Driver Behavior

As with any other ATM strategy, any effect is due to changes in driver behavior. Using an instrumented vehicle and 11 video cameras, Wu et al. (2007) measured detailed driving performance of drivers merging at on ramps and those on motorway carriageways in a ramp metering controlled intersection with and without ramp metering control. The main behavioral parameters used for the study include: speed, headway, acceleration and deceleration, sizes of accepted gap, merge distance, speed at merge, etc. Based on the study, it is believed that ramp metering does result in driving behavior changes of traffic on the motorway carriageway and on ramp. It improves the merge condition of traffic at the on ramp, but may cause minor reduction of speeds of traffic on motorway carriageway during the metering time. Design of ramp meters and notification signs is critical for ensuring driver familiarity; Davis et al. (2000) provide a detailed overview of design and geometry standards regarding ramp meter implementation in California. Describing driver behavior when waiting in ramp metering queue involves additional effort: spending time in a queue is generally regarded as worse than spending an equivalent amount of time driving (Hensher, 2001), and increases driver stress and aggression (Hennessy and Wiesenthal, 1999). Thus, sufficient enforcement is needed to prevent violations (Lancaster et al., 1997), especially when a meter is first activated during the peak period.

As discussed above, with proper enforcement and compliance, ramp metering can lead to quantifiable benefits in terms of traffic volume, travel delay, travel speed, and reduced accident rates due to improved merging and reduced demand for freeways.

2.4 Dynamic Signing and Re-Routing

Using variable message signs (VMS) to provide real-time reliable information to users is an essential component of an effective ATM scheme. A critical component of this goal is the use of advanced technologies to provide dynamic rerouting information to users. These signs provide users with a more satisfactory and less stressful trip because they are more informed about roadway conditions (Middelham, 2006).

The FHWA report on Active Traffic Management strategies states a few operational guidelines that need to be followed while providing users with dynamic signing and re-routing. These are: (1) a commitment to providing alternate route information to roadway users in response to nonrecurring congestion; (2) adequate installation of sign gantries along a facility at critical locations to ensure that sufficient advance notice of alternate routes is provided; (3) deployment in conjunction with speed harmonization and temporary shoulder use; (4) connection to a traffic management center that serves as the focal point for the system; (5) connection to adjoining traffic management centers to coordinate alternate route information based on roadway conditions and special events in adjoining regions; and (6) coordination with local communities to minimize the impact of alternate route information on the arterial network.

This strategy has been used in many countries and benefited travelers. We will discuss this in detail in following parts.

2.4.1 Dynamic Rerouting and Traveler Information (Germany)

Germany has a national goal to adequately serve 80 percent of all trips on the motorway network by standardized real-time traffic and traveler information (RTTI) by 2010. A critical component of this goal is the use of advanced technologies to provide dynamic rerouting information to users. As Figure 2.5 shows, Germany installs rotational prism guide signs that change with traffic conditions. If an incident occurs along a facility, operators at the TMC deploy alternate guide sign information combinations that provide alternate route information to roadway users. Similar information is also provided on full-matrix dynamic message signs (DMS) installed on other roadways. On facilities that employ speed harmonization combined with temporary shoulder use, the signs change so that the information displayed for the operational lanes is appropriate.

Germany also has initiated a concerted effort to standardize messages on dynamic message signs to reduce the likelihood of motorist confusion. German DMS display messages follow a set of basic principles to ensure comprehension by the most users:

- Internationally understandable legends
- As little text as possible
- As much text as unavoidable
- Symbols and signs of the Vienna Convention preferred

Figure 2.5: *Dynamic rerouting in Germany*

Another component of the RTTI system is the traffic message channel. This information channel provides traffic-related messages to motorists via onboard Global Positioning System (GPS) units that can decode them into preferred languages. Nearly 5 million units are already in use and their cost is decreasing.

RTTI transmitted via public radio is considered a major tool for traffic managers. The Traffic Message Channel of the Radio Data System (RDS-TMC) has been operational since 1997. In Germany today, RDS-TMC messages are transmitted without program interruptions on roughly 50 radio program chains. These digitally encoded traffic messages are used in in-vehicle navigation systems. This enables road operators to include routes not equipped with variable direction signs or variable message signs in their management schemes. It is a traffic policy objective to enable free access to safety-related traffic messages. Therefore, no subscription fees are charged for RDS-TMC information.

The operation of a digitally encoded information channel has enabled traffic managers to automatically include information from roadside traffic detectors in the information chain. Typically, 2 to 5 minutes are needed to provide information about congestion.

The success of RDS-TMC in Europe and especially in Germany has contributed to its worldwide application, including in the United States, Australia, and Singapore. Efforts are underway to use digital transmission channels such as digital multimedia broadcasting (DMB) in the future to provide a higher transmission capacity so that local and urban information can also be included.

2.4.2 Dynamic Route Information Panels (Netherlands)

Dynamic route information was first used in the Netherlands in 1990. Today, more than 100 gantries displaying these panels are used across the country on major motorways with another 22 planned. Shown in Figure 2.6, these panels (which can be either DMS or rotational prism signs) are intended to provide en route information on queues, major incidents, and appropriate routes.

Figure 2.6: *Dynamic route information panels in the Netherlands*

The systems provide users with a more satisfactory and less stressful trip because they are more informed about roadway conditions. Several assessment studies indicate that under normal conditions, between 8 and 10 percent of motorists adhere to the revised route information and that overall network performance may increase up to 5 percent. While the information panels are an effective congestion management strategy today, the Dutch believe that the long-term usefulness of this strategy may be limited because they anticipate that all critical road and traffic information will eventually be provided to the user in the vehicle. Furthermore, Dutch law does not require the government to provide traveler information directly to users. Instead, information is sold or provided to independent information service providers who repackage that information and disseminate it through various sources.

2.4.3 Impacts on Driver Behavior and Safety

Chatterjee et al. (2002) researched driver response to VMS information in London. The resultant models indicate that the location of the incident and the message content are important factors influencing the probability of diversion. Erke et al. (2007) investigated the effects of route guidance VMS on speed and rout choice. Result showed that there was high compliance with the message in VMS. About every fifth vehicle changed route choice according to the recommendation, and almost none drove as far as the closed road section. Speed measurements of 3342 vehicles showed large speed reductions, and video observations showed that large proportions of vehicles braked while approaching the VMS. Safety problems may result directly from distraction, or indirectly from the reactions of the drivers to the distraction. Using a real-time crash prediction model and a microscopic traffic simulation model, Lee et al. (2006) examined the effect of control factors of automated strategies of variable speed limits on the crash potential reduction and total travel time. The study results indicated that variable speed limits could reduce crash potential by 5–17%, by temporarily reducing speed limits during risky traffic conditions when crash potential exceeded the pre-specified threshold. Another way to study the safety implications of using dynamic signing is by using an agent based simulation approach, where the road users are the agents. Wahle et al. (2002) used this approach to study the impact of information on a two-route scenario. They found out that the potential benefit of providing information depends mainly on the type of information provided. Dia (2002) also used agent-based simulation to determine the factors influencing drivers' behavior and their propensity to change route and adjust travel patterns.

Dynamic signing and re-routing works in conjunction with the other ATM strategies, especially shoulder utilization and speed harmonization (see Figure 2.5). The decisions regarding speed limit changes and utilization of shoulders as congestion management efforts should transform seamlessly into effective re-routing and signing in order to ensure that the drivers and users are informed of the congestion reduction strategies well in advance and as they keep changing. In Germany, the facilities that employ speed harmonization combined with temporary shoulder use, the signs change so that the information displayed for the operational lanes is appropriate (FHWA).

2.5 Safety Analysis

The impact of geometric design on safety has been studied from multiple perspectives; however, by comparison, relatively limited research has been conducted on the impact of traffic operations strategies or ITS deployment on safety performance. Safety prediction methodologies and safety performance assessments are typically conducted using historical crash data to assess the influence of changes in geometry, traffic volume (volume relative to capacity), and speed limits. In the last decade, researchers have begun to explore predicting safety performance or crash potential through the use of microsimulation and surrogate measures of safety. This relatively recent development provides the opportunity to evaluate the potential safety performance of facilities due to changes in traffic flow characteristics (e.g., speed variation, queue formation, density).

Krammes and Glascock (1992) and Anderson et al. (1999) conducted previous research examining safety and geometric design consistency. The use of historical traffic data to assess the safety of freeways has been extensively studied as well. Historical crash and traffic data has also been used to explore the relationship between traffic volumes and volume-to-capacity ratios with crash rates (Ceder and Livenh, 1982; Gwynn, 1967; Zhou and Sisiopiku, 1997). The influence of speed limits on safety has been examined. Thorton and Lyles (1996) are one of many sets of researchers who have explored the influence of speed limits on safety by comparing the safety performance of freeways when the speed limit is either 55 mph or 65 mph. Their study did not find a significant difference in safety performance. However, Raju et al. (1992), found the number of fatal accidents increases with the speed limit for a rural interstate highway in the state of Iowa.

More recent research in the last decade focusing on quantifying safety performance as a function of traffic flow characteristics includes a series of studies and papers written by Lee et al. in 2002, 2003, and 2004. Lee et al. (2002, 2003) identified four crash precursors: coefficient of variation of speed (which is defined as the standard deviation of speed divided by the mean speed) upstream of a (crash) location, average density upstream of a location, average difference in speed upstream and downstream of a specific location and covariance of volume difference (between adjacent lanes) upstream and downstream of a specific location. Lee et al. (2004) apply the crash potential function to trigger speed harmonization (i.e., change the speed limit posted via variable speed limit algorithm and related ITS infrastructure). When the crash potential exceeds a certain level, the speed limit is reduced. They found the speed harmonization beneficial for the freeway's safety performance.

Similarly, Abdel-Aty et al. (2005) used matched-case control logistic regression to model the probabilities of crash for Interstate 4 in Orland, Florida. A distinction was made between crashes in the low-speed and high-speed regimes. Factors that were found to be statistically significant in contributing to crash likelihood were the coefficient of variation in, average occupancy and standard deviation of volumes (for the low-speed regime) and average occupancy, standard deviation of volumes and average volumes (for the high speed regime).

Another related approach, which was undertaken by Oh et al. (2001), used loop detector data for a freeway section in California in an effort to identify reliable indicators or precursors to actual crash events. They found the standard deviation of speed 5 minutes before the crash occurrence is the best indicator for predicting an actual crash or collision. With this precursor variable, they developed probability density functions to determine whether the current traffic condition belongs to either normal or disruptive traffic conditions. Oh et al. (2001) found the reducing speed variation decreases the crash likelihood.

Chapter 3. Simulation Framework for Evaluating ATM Strategies

A simulation framework for evaluating traffic operation and safety benefits of active traffic management strategies was developed. This framework constituted multiple traffic simulation models for evaluating both corridor-level and network-level effects on traffic operations. The framework also contained a traffic safety model that uses outputs from the traffic operations models to perform safety analysis. Traffic operations and safety benefits were evaluated using various performance metrics. The rest of the chapter presents each component of this simulation analysis framework in detail.

3.1 Simulation Framework for Traffic Operations Analysis

The implementation of active traffic management (ATM) strategies necessitates a careful cost-benefit analysis to justify the associated costs of ATM implementation and maintenance of required infrastructure. Naturally, the most straightforward way to perform such analysis is the selection of a test corridor and the actual implementation of the strategies under consideration. However, due to financial constraints, this might not be feasible as active traffic management strategies typically require extensive use of ITS. An alternative and economically viable way to perform such analysis is via traffic simulation.

Traffic simulation can be performed at various levels of detail: micro, meso, and macro. Each of these types of simulation has its own advantages and disadvantages. For instance, microsimulation can model driver behavior most accurately, but it is computationally intensive. On the other hand, mesoscopic simulation is less computationally intensive, but comes at the expense of a less detailed modeling of driver behavior. In this report, we propose a hybrid traffic simulation approach to eliminate the above stated disadvantages. More specifically, we consider the combination of micro- and mesoscopic traffic modeling. ATM strategies are typically implemented along a specific corridor of a freeway, but their overall impact can be network-wide. Therefore it is imperative to perform a more detailed simulation study (i.e., microsimulation) at the corridors where active traffic management strategies are being applied, whereas in order to capture network effects (and to maintain a reasonable computation time), a less detailed but sufficiently accurate simulation (i.e., mesoscopic simulation) can be performed for a larger network area.

The rest of this chapter is organized as follows. First, we provide a discussion of microsimulation modeling techniques. Then we examine its mesoscopic counterpart. The proposed multi-resolution approach is detailed thereafter. The discussion of the traffic operations simulation models is followed by the presentation of the Surrogate Safety Assessment Model (SSAM, 2010). SSAM uses output from the traffic operation models to perform conflict analysis in order to evaluate the safety impacts of ATM strategies. Then the chapter ends with a discussion on performance measures for evaluating the impact of ATM strategies on traffic operations and safety.

3.1.1 Microscopic Modeling Techniques

Microscopic simulators model individual vehicles with a high degree of realism. This includes vehicle-specific lane changing and driving behavior, detailed signalization and gap acceptance models, and simulating at very small time intervals, often less than a second. In order to simulate the longitudinal and lateral movements of individual vehicles, microsimulators employ detailed car following and lane changing models. These models incorporate the human behavioral element of real traffic via the use of parameters. While a high level of detail is the key attraction of microsimulators, accounting for such detail requires calibration of a large number of parameters. Moreover, the model parameters are highly sensitive and often deliver misleading results if not chosen correctly. Therefore, proper calibration and validation is necessary when using microsimulation models. Accommodating this level of detail greatly increases the computational requirement, and microscopic analysis is typically confined to relatively small analysis zones and, hence, may give rise to boundary effects (Burghout et al., 2005).

An alternative to the traditional, discrete time-step microsimulators is event-based simulators, such as that built into Dynameq (INRO, 2009). In Dynameq, realistic but simplified traffic models can be calibrated very easily with only a handful of parameters, each with real-world significance. Thus, its event-based supply-side simulator provides an order of magnitude performance improvement over traditional time-step traffic microsimulation. As the traffic simulation is event-based, the traffic phenomena that trigger congestion are modeled explicitly, including signals, conflicting movements at intersections, lane permissions for turning movements and vehicle classes, and weaving.

Various microsimulation packages exist. Well-known commercially available microsimulation packages include, but are not limited to, CORSIM (McTrans, 2009), VISSIM (PTV, 2009), and Paramics (QuadStone Paramics, 2009). For a detailed description of these various alternatives, we refer to their respective websites.

3.1.2 Mesoscopic Modeling Techniques

Route choice models predict the paths that users will follow when traveling in a transportation network. Classically, route choice forms the fourth and final step of the transportation planning process. Route choices are modeled after travel demand is known and the choice of mode has already been made. The most common assumption is that all users choose the route that minimizes their travel time; this allows routes to be found using efficient network algorithms such as those developed by Dijkstra (1959), Dial (1969), Loui (1983), Ziliaskopoulos and Mahmassani (1993), and Waller and Ziliaskopoulos (2002). The state-of-the-art in this domain can identify the routes chosen by travelers in the presence of time-varying and uncertain costs, both of which are critical for accurately modeling real-world transportation networks.

Because all users are simultaneously trying to minimize their travel times, the resulting state is an equilibrium in which nobody can reduce their travel time by unilaterally switching their routes. This condition was first identified by Wardrop (1952) and Beckmann et al. (1956), and is known as the user equilibrium (UE) condition. The traditional solution procedure to solve for UE is based on Frank-Wolfe algorithm (Frank and Wolfe, 1956) although faster and more accurate methods have been developed by Jayakrishnan et al. (1994a), Bar-Gera (2002), and Dial (2006). Adaptations have also been made to account for user error in perceiving travel times, leading to a class of stochastic user equilibrium models, some limited demand elasticity effects, and multiclass equilibrium models where different types of users (for instance, transit, HOVs, and single-occupant vehicles) see different travel times.

These traffic assignment models have sound mathematical properties that allow solutions to be found efficiently; however, a major shortcoming is their inability to account for how traffic evolves over time. This is critical when considering dynamic traffic management strategies that require the explicit modeling of time. Moreover, traffic itself is inherently dynamic, consisting of congestion that evolves over time, and queues that form and dissipate at traffic signals. Thus, by explicitly modeling changing network conditions, dynamic modeling can represent traffic flow in a far more accurate manner. These needs have been addressed with the creation of dynamic traffic assignment (DTA) models, which can account for these factors. The first DTA model was developed by Merchant and Nemhauser (1978), and a thorough overview of progress since then is available in Peeta and Ziliaskopoulos (2001).

Unlike the models mentioned in the previous section, there is no standard DTA model. Currently, the most promising approaches are based on traffic simulation models, as these are able to capture the most realism in how traffic evolves over time. Examples of simulation-based DTA software include DynaMIT (Ben-Akiva et al., 1997), VISTA (Ziliaskopoulos and Waller, 2000), DYNASMART (Jayakrishnan et al., 1994b), and DynaCHINA (Lin and Song, 2007). These often use efficient traffic propagation procedures such as the cell transmission model (Daganzo, 1994).

3.1.3 Multi-Resolution Simulation

From the previous sections, it is clear that both microscopic as well as mesoscopic simulation have their advantages and disadvantages. The former is far more accurate when properly calibrated, but computationally intensive, whereas the latter is less detailed and less computationally intensive. Hence it is natural to consider the so-called hybrid models, which combine the advantages of the two modeling paradigms (Figure 3.1).

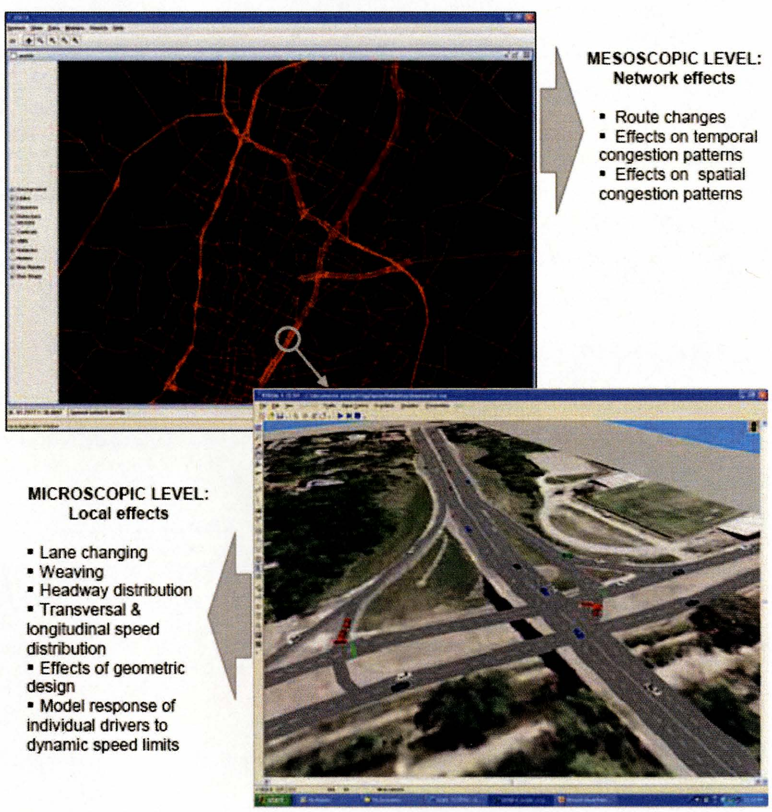


Figure 3.1: *Multi-resolution modeling approach*

This multi-resolution approach is particularly appropriate when considering active traffic management strategies: these strategies tend to be employed locally (microsimulation can yield accurate traffic patterns in these local areas), while the consequences of such measures can be potentially felt at a much larger scale (mesoscopic simulation can yield sufficiently accurate traffic patterns in the entire network within a reasonable computational time). In particular, microsimulation is able to capture the impacts of the active etraffic management strategies on driving behavior such as lane changing and weaving. On the other hand, mesoscopic simulation is able to accurately model the change in route choice and temporal congestion patterns at the network level.

3.2 Simulation Framework for Traffic Safety Analysis

Safety is of paramount importance in operation of transportation systems. Typically, safety analysis of highway systems is conducted by analyzing vehicle crash reports generated by the police over a long period of time. The infrequent and unpredictable nature of highway crashes poses a challenge in collecting sufficient crash data to perform statistically significant safety analysis. Moreover, this approach cannot be used to evaluate new traffic-control strategies and roadway designs that are yet to be implemented in the field.

Under the guidance of the Federal Highway Administration (FHWA), Siemens Corporation developed the Surrogate Safety Assessment Model (SSAM), which combines microsimulation of traffic and automated "conflict analysis". SSAM performs "conflict analysis" by using the space-time vehicle trajectories produced by microscopic simulation models implementing various traffic-control strategies. A vehicle conflict is defined as a scenario that will result in a collision if no evasive action were taken by the two vehicles involved. The complete simulation analysis framework for evaluating traffic operations and safety benefits of active traffic management strategies is show in Figure 3.2.

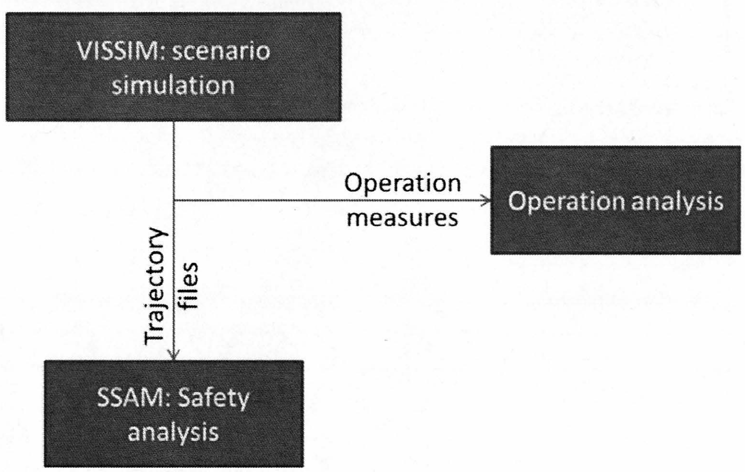


Figure 3.2: *Simulation analysis framework for evaluating ATM strategies*

The framework of the models developed for predicting the impact of novel ATM strategies can be briefly described as follows:

1) Calibrate and validate a base case microscopic simulation model in VISSIM. Simulate various ATM strategies in the VISSIM model and obtain operations measures to evaluate the impact of the ATM strategies on traffic operations. Perform simulation experiments for an ATM strategy multiple times using different "seed numbers" in VISSIM (we simulated each ATM strategy ten times) to perform statistical analysis.
2) Export the corresponding vehicle trajectories from the VISSIM model for further safety analysis in the SSAM. A vehicle trajectory file contains a complete set of space-time trajectories of all the vehicles for the entire simulation duration in VISSIM. Import multiple sets of vehicle trajectory files into the SSAM for all the ATM scenarios to perform safety analyses. These safety analyses included: various surrogate safety measures, frequency analysis of different conflict types, and visual display of conflicts on the road network.
3) For network-level impacts of the ATM strategies, obtain changes in throughput, capacity, average speed, etc. from the microscopic simulation models implementing the ATM strategies, and incorporate these changes in the regional VISTA dynamic traffic assignment model. The VISTA model evaluates the effect on the corridor-level changes on regional traffic flow pattern by modeling route choice behavior at the network level.

SSAM was used to perform safety analysis for the active traffic management (ATM) scenarios implemented in this study. SSAM calculates several surrogate safety measures using the vehicle trajectory files obtained from the VISSIM models to evaluate safety, and determines the types and frequency of the conflicts. SSAM also provides statistical comparisons of conflict frequencies and surrogate safety measures between two scenarios using the T-test. It can graphically display the locations of conflicts on the network by conflict types, and aids visual analyses of safety results.

3.3 Performance Measures

An objective evaluation of the effectiveness of active management strategies is at least as important as the implementation itself: without an objective evaluation, it is simply not known whether the newly introduced strategies are effective at all. Moreover, it is necessary to justify the investment of large amount of financial resources to new projects. We need performance measures to evaluate ATM strategies objectively. In this section we present and discuss various performance measures that are able to capture the effectiveness of the active traffic management strategies considered in this project. In the discussion, we distinguish local and network-wide traffic operations performance measures. Local performance measures are designed to capture the local impacts, i.e., the impacts on the test corridor itself (and its direct surroundings). When local measures indicate an improvement in the local traffic conditions, it is not necessarily true that the network-level impact is positive as well. In fact, one can imagine situations in which bottlenecks shift to other parts of the network, resulting in an overall worse system-wide condition. Network-level performance measures are designed to capture these global impacts. This section also discusses surrogate measures of safety.

In the following discussion, we will discuss both the traffic operations (local and global) and traffic safety measures of performance. The change in a certain performance measure is computed based on the conditions before and after implementation of ATM strategies.

3.3.1 Traffic Operations Performance Measures (Local)

Relative change in average speed

Speed harmonization lowers the speed limit temporarily in order to create conditions for a "smoother" flow of traffic. Hence, one might hypothesize that speed harmonization can potentially lower the average speed on the test corridor. On the other hand, one might argue that because of the more uniform traffic flow, it is conceivable that the average speed among all road users on the test corridor increases. With peak-period shoulder use, an additional running lane is provided. It is to be expected that the average speed will increase if the number of vehicles traveling on the test corridor remains the same. In general, for both active traffic management strategies, a positive change in the average speed is desired.

Relative change in average density

The average density of a road section is defined as the number of vehicles per unit length of the road. Everything else being equal, a low average density is preferred as this is beneficial for the safety of road users. Hence in terms of the relative change in average density, it is desirable to see a change that is large in magnitude and negative in sign.

Relative change in average throughput across a section

The average throughput across a given section is defined as the average number of vehicles passing a given section within a given amount of time. Obviously, everything else being the same, a larger throughput is preferable.

3.3.2 Traffic Operations Performance Measures (Network-wide)

Relative change in total trip time between a given origin-destination pair

For both speed harmonization and peak-period shoulder use one can anticipate that drivers change their routes as a result of the new driving conditions. Travel times between certain locations might increase because of the increased travel demand on certain roads (for example, some people might avoid roads that are subject to speed harmonization and take a detour). On the other end, it is also conceivable that travel times between other origin-destination (OD) pairs decrease because of the traffic management strategies. Ideally, one would like to see large negative changes (i.e., shorter travel times) in the trip time between any given OD pair.

Relative change in total system-wide travel time

The previous performance measure considers the relative change in travel time for all OD pairs individually. As a decision maker, it is often not possible to examine the individual changes. Rather, one performance measure for the entire network is desired. Such performance measure is provided by the change in total system travel time, which is defined as the sum of the changes in the individual OD pairs.

3.3.3 Traffic Safety Performance Measures

Safety performance of a facility is a critical consideration when exploring the use of new technologies. Previous studies evaluating the variable speed limits and speed harmonization have found safety benefits in applying these strategies, particularly when employed in adverse weather conditions. Safety prediction methodologies and safety performance assessments are typically conducted using historical crash data to assess the influence of changes in geometry, traffic volume (volume relative to capacity), and speed limits. In the last decade, researchers have begun to explore predicting safety performance or crash potential through the use of microsimulation and surrogate measures of safety. This relatively recent development provides the opportunity to evaluate the potential safety performance of facilities due to changes in traffic flow characteristics (e.g., speed variation, queue formation, density). Previous research examining safety and geometric design consistency was conducted by Krammes and Glascock (1992) and Anderson et al. (1999). The use of historical traffic data to assess the safety of freeways has been extensively studied as well. Historical crash and traffic data has also been used to explore the relationship between traffic volumes and volume-to-capacity ratios with crash rates (Ceder and Livenh, 1982; Gwynn, 1967; Zhou and Sisiopiku, 1997).

However the above approaches are not suitable for evaluating safety impacts of novel ATM strategies that have not been implemented in the field yet. Therefore, a simulation-based safety analysis approach, which combines microsimulation and automated "conflict analysis", is best suited for evaluating safety impacts of ATM strategies. The Surrogate Safety Assessment Model (SSAM) of the Federal Highway Administration adopted this approach.

SSAM was used to perform safety analysis for the active traffic management (ATM) scenarios implemented in this study. SSAM calculates several surrogate safety measures using the vehicle trajectory files obtained from the VISSIM models to evaluate safety, and determines types and frequency of the conflicts. The safety measures we used for this project included:

- **TTC**: the minimum time-to-collision value observed during the conflict.
- **PET**: the minimum post encroachment time observed during the conflict. Post encroachment time is the time between when the first vehicle last occupied a position and the second vehicle subsequently arrived at the same position. A value of 0 indicates an actual collision.
- **MaxS**: the maximum speed of either vehicle throughout the conflict
- **DeltaS**: the difference in vehicle speeds as observed at the simulation time where the minimum TTC value for this conflict was observed.
- **DR**: the initial deceleration rate of the second vehicle.
- **MaxD**: the maximum deceleration of the second vehicle.
- **MaxDeltaV**: the maximum change between conflict velocity and the post-collision velocity of either vehicle in the conflict. This is a surrogate for the severity of the conflict, calculated assuming a hypothetical collision of the two vehicles in the conflict.

SSAM also provides statistical comparisons of conflict frequencies and surrogate safety measures between two scenarios using the T-test. It can graphically display the locations of conflicts on the network by conflict types, and aids visual analyses of safety results.

Using above stated models and simulation analysis framework, we implemented various ATM strategies on the testbed road network and evaluated their impacts on traffic operations and safety. Various operation measures were compared and analyzed using evaluation function of VISSIM. Safety impacts were tested using the SSAM. The application of these models and our analysis based on the output of these models provided a clear view about the potential impact of those ATM strategies, and also helped us to come up with the guidelines for properly using those strategies.

3.4 Implementation in VISSIM/VISTA

The starting point of the multi-resolution approach is a simulation model of the Austin area in the mesoscopic traffic simulator VISTA. In order to illustrate the ATM strategies investigated in this project using a real-world example, we selected the stretch of MoPac (Loop 1) in Austin between Enfield Road and 45th Street (see Figure 3.3) for demonstration purposes. This corridor is known for its recurring congestion. Moreover, for this stretch of Loop 1 we have sufficient data for calibration and we possess a calibrated mesoscopic simulation model to assess the network impacts. More details about the development of a testbed model are presented in Chapter 4.

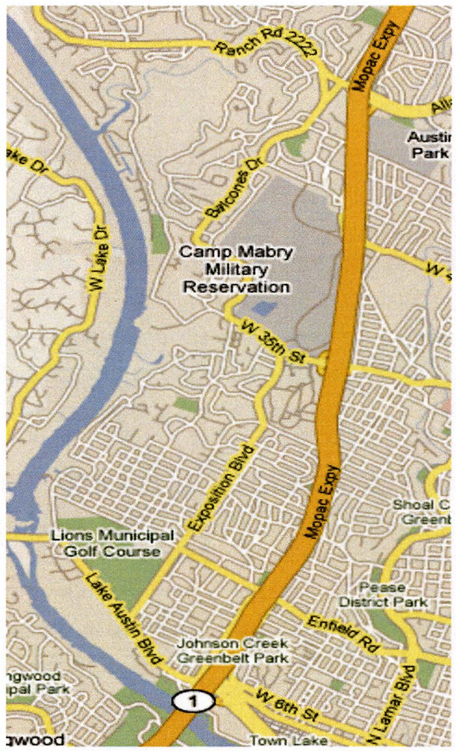


Figure 3.3: *Loop 1 test corridor for ATM strategies*

The first step in the approach is to perform traffic simulation in VISTA for the "entire" Austin network (engineering judgment is needed here to determine what constitutes the "entire" network, i.e., the parts of the network where the effects of the ATM strategies are likely to have impact). Based on this simulation run, performance measures about the current (i.e., pre-ATM strategies implementation) network are collected. Furthermore, the vehicular flows at the boundaries of the Loop 1 area under study are also extracted from the simulation results. These boundary conditions are then fed into the VISSIM network of Loop 1 that we have built for this project. Microsimulation is performed to determine the current, pre-ATM strategies implementation traffic conditions on Loop 1. The next step is to implement ATM strategies. Because of the modified speed limits and/ or roadway geometrics, it is to be expected that people change their driving behavior, routes, and so on. In order to determine the local impact of the particular ATM strategy, we again perform microsimulation in VISSIM, after the implementation of ATM strategies. New performance measures are collected for the post-ATM strategies implementation conditions. A comparison between these pre- and post-ATM conditions will reveal the local impact of the ATM strategies. In order to evaluate the network-wide impact of the ATM strategies, the post-ATM measures on Loop 1 have to be represented in VISTA's Austin network (VISTA, 2010). Figure 3.4 provides a schematic diagram of this process.

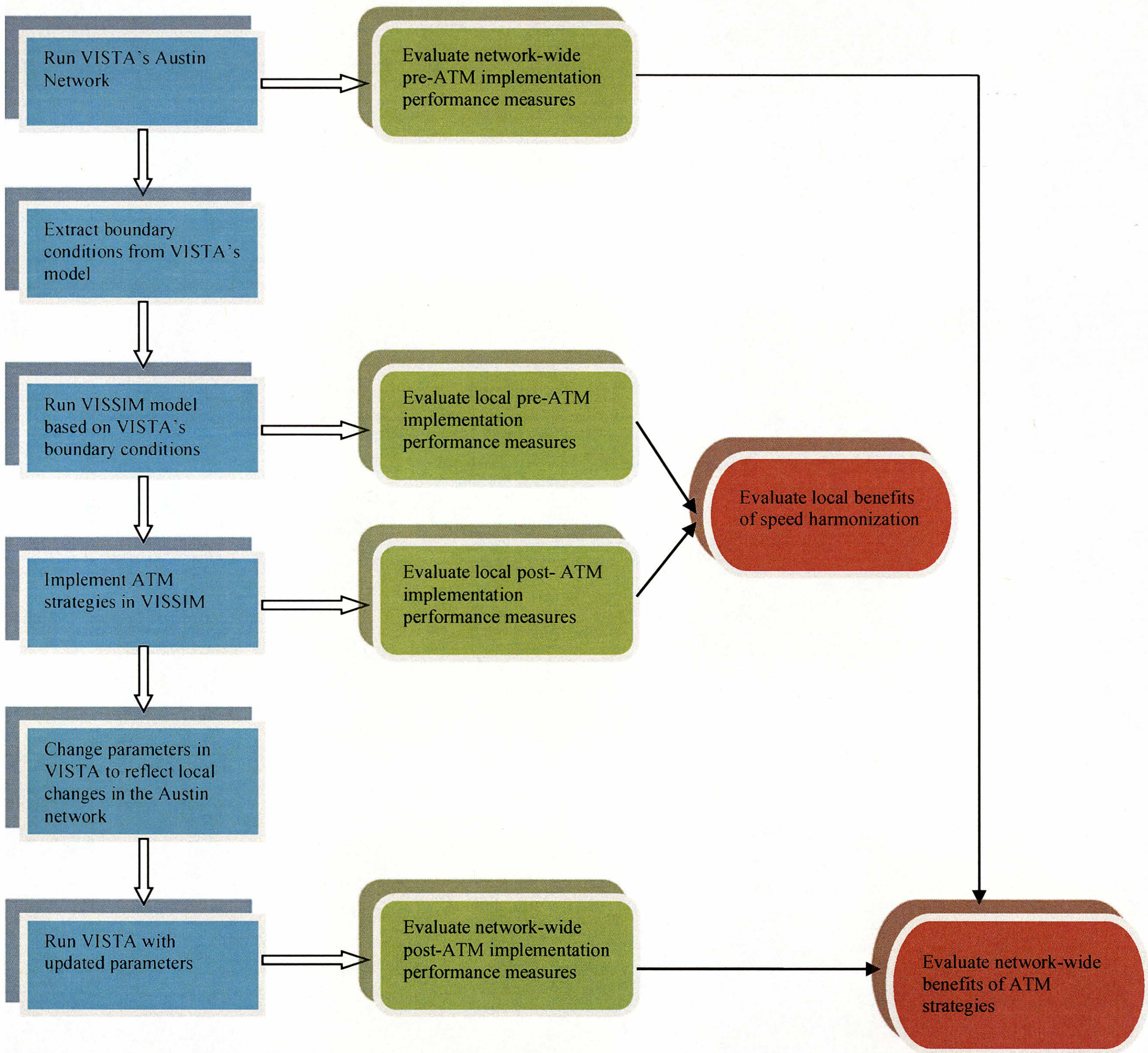


Figure 3.4: *A multi-resolution approach for ATM strategies evaluation*

To this end, we will perform a systematic procedure in which parameters in VISTA's Austin model (e.g., cell capacities) are changed to reflect the change in traffic conditions on Loop 1. That is, parameters in the VISTA network will be changed so that the traffic patterns, post-ATM implementation, observed on Loop 1 are approximately the same in both the VISSIM as well as the VISTA network. For instance, in case of peak-period shoulder use, one can temporarily increase cell capacities in VISTA to represent the addition of a shoulder lane as a running lane. Once these settings have been found, we perform a new run of mesoscopic simulation (with the new, post-ATM implementation settings). Based on the simulation results, performance measures can be calculated and compared with the (network-wide) pre-ATM implementation measures. This comparison will yield the network-wide impacts of the ATM strategies.

Chapter 4. Development and Calibration of a Testbed Model

4.1 Selection of a Texas Freeway Testbed for ATM Implementations

A section of the Missouri-Pacific (MoPac) Expressway (northbound) in Austin, Texas was selected as a testbed for this study. The testbed is a 4-mile section of MoPac from the 5th Street near downtown Austin in south to RM 2222 in north.

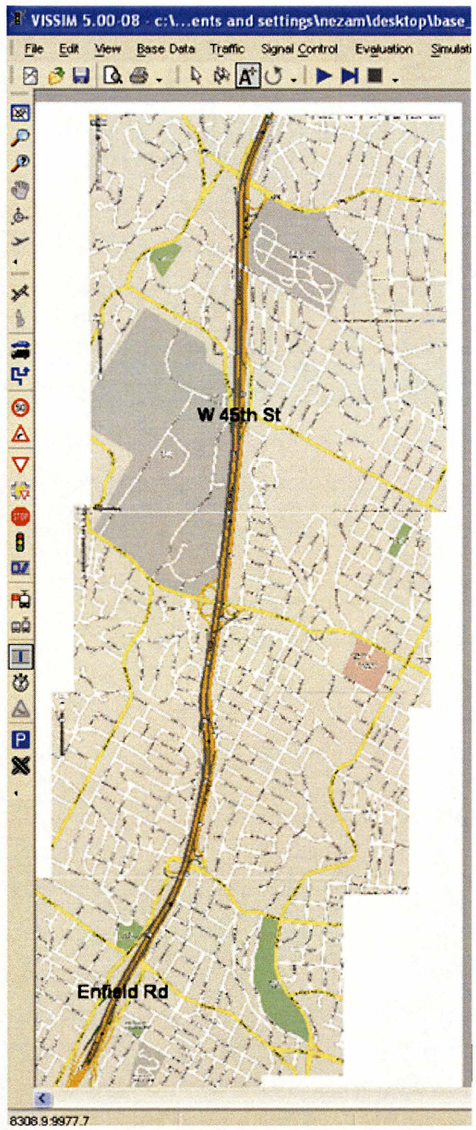


Figure 4.1: *VISSIM model of MoPac testbed*

The selection of this testbed to evaluate the impact of active traffic management (ATM) strategies on traffic operations and freeway safety was governed by the following considerations:

- Prominence of this corridor: MoPac Expressway is one of the two major north-south corridors in the Central Texas region and connects downtown Austin to other parts of the region. Improved traffic conditions on MoPac due to ATM implementations will alleviate congestion in the Austin metropolitan area and will have regional network-wide effect on traffic flow pattern.
- Recurrent congestion: This segment of the freeway carries heavy traffic during peak hours and sees recurrent congestion (see Table 1 on page 6). This makes the corridor a suitable candidate site for ATM implementation, which seeks to alleviate congestion using intelligent transportation systems (ITS) when the roadways are operating near capacity.
- Bus-on-shoulder use consideration: MoPac is under consideration for bus-on-shoulder use by the Texas Department of Transportation (TxDOT). Therefore, peak-period shoulder use on this testbed will aid TxDOT in making decision regarding shoulder use (*1*).
- Data availability: Loop detector data were available at three locations inside the testbed, which were used to calibrate the model. Loop data were obtained from the Austin office of the Texas Transportation Institute for the year 2007 and 2008.

4.2 Selection of Software Tools to Study the Implications of ATM

Different software tools were selected to comprehensively study the implications of ATM strategies on traffic operations and safety of the freeways. These software tools study transportation networks in different paradigms, but at the same time they use inputs from each other to model the effect on ATM strategies consistently. Three software models were chosen for the purpose of this study, and they are described below.

To predict the impact of ATM strategies on traffic operations and safety, three types of models were developed: **VISSIM models for corridor-level micro-simulation** of ATM strategies, **SSAM models for safety analysis**, and regional dynamic traffic assignment **VISTA model for network-level effects** of ATM strategies.

4.2.1 VISSIM Microscopic Simulation Model

Microscopic simulation models study the network at the finest level of detail by simulating the movements of individual vehicles. They employ calibrated car-following and lane-changing models to achieve modeling at such detailed level. This makes the microscopic models a suitable tool to evaluate the effect of ATM strategies that seek to influence the behavior of individual vehicles. For the purpose of this study, VISSIM microscopic simulation model was selected (PTV, 2009). VISSIM provides tools and functionalities to implement various ATM strategies in the simulation model. In addition, vehicle actuated programming (VAP) module of VISSIM makes it possible to implement traffic control strategies in response to real-time traffic conditions. VAP is primarily meant to model actuated signal control system, where traffic control decisions are based on real-time traffic data collected through loop detectors coded in the simulation model. However, VAP also provides the ability to change posted speed limit on roadway links, and thus aids in the implementation of variable speed limits. Similarly, real-time shoulder-use decisions can be made using the VAP based on prevailing traffic conditions.

The research team has prior experience with implementing speed harmonization and peak-period shoulder use in VISSIM software. This further aided the decision of selecting VISSIM for modeling the safety implication of active traffic management strategies.

VISSIM is a leading microscopic simulation program for multi-modal traffic flow modeling. It is an ideal tool to simulate different traffic scenarios before starting implementation. We created a road network to simulate Mopac Expressway (Loop 1) in Austin between 5th Street and RM 2222. To make the simulation model more accurately reflect the traffic condition of this specific road network, it was calibrated using loop detector data for Mopac Expressway obtained from Texas Transportation Institute's (TTI) Austin office. We simulated the base scenarios first. Then we applied different ATM strategies on the same network and compared various evaluation measures under different conditions.

VISSIM offers a wide range of evaluations that can be used to assess the performance of the whole or part of the traffic systems. The operation measures we directly obtained from VISSIM included throughput, speed at different locations, coefficient of variance of speed, average number of stops, delay, density, etc. Using these operation measures, we can compare the performance of the network under different scenarios, finding out the advantages and disadvantages of each strategy, and analyzing what kind of improvement or modification should we do when we apply those strategies. VISSIM can also export vehicle trajectory files that can be used to evaluate traffic safety conditions.

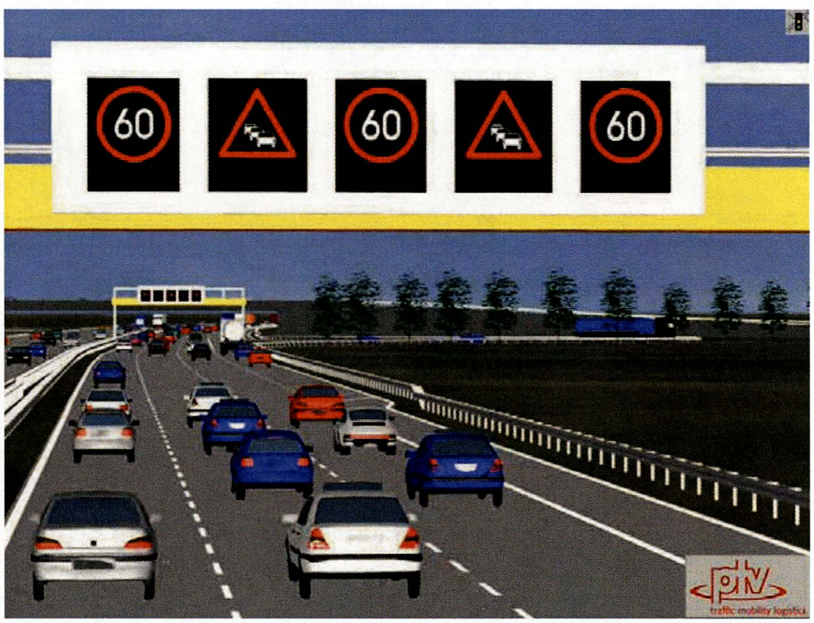


Figure 4.2: *Example of VISSIM traffic management application (PTV America)*

4.2.2 VISTA Mesoscopic Simulation Model

Since the microscopic models simulate movements of individual vehicles using car-following and lane-changing models, they are computationally intensive. Therefore, microscopic models are limited in their capability to model a larger geographical area. This precludes them from efficiently modeling the regional network-wide effect of implementing localized ATM strategies to alleviate congestion. Mesoscopic simulation models are less detailed than the microscopic models, but they can model larger networks. Such models predict regional traffic flow patterns and route flows at a regional scale due to localized changes in the traffic conditions. For the purpose of this study, VISTA mesoscopic simulation model was selected to study the network-wide effect on implementing ATM strategies to manage urban freeway congestion (Ziliaskopoulos and Waller, 2000). VISTA has a cell-transmission based dynamic traffic assignment model which can predict time-dependent route flow patterns in response to time-varying network conditions.

VISTA is a transportation-modeling framework providing Dynamic Traffic Assignment (DTA), traffic simulation, and reporting over the Internet. Developed at Northwestern University, the VISTA model has been successfully used on transportation projects across the USA and Europe. VISTA can simulate the movements of multiple modes across large networks and incorporate the effects of ITS and traveler information systems into driver behavior. We use VISTA in this project to extend corridor-level analysis to include network-level effects, thus considering the impact on feeder routes and parallel routes due to "induced demand".

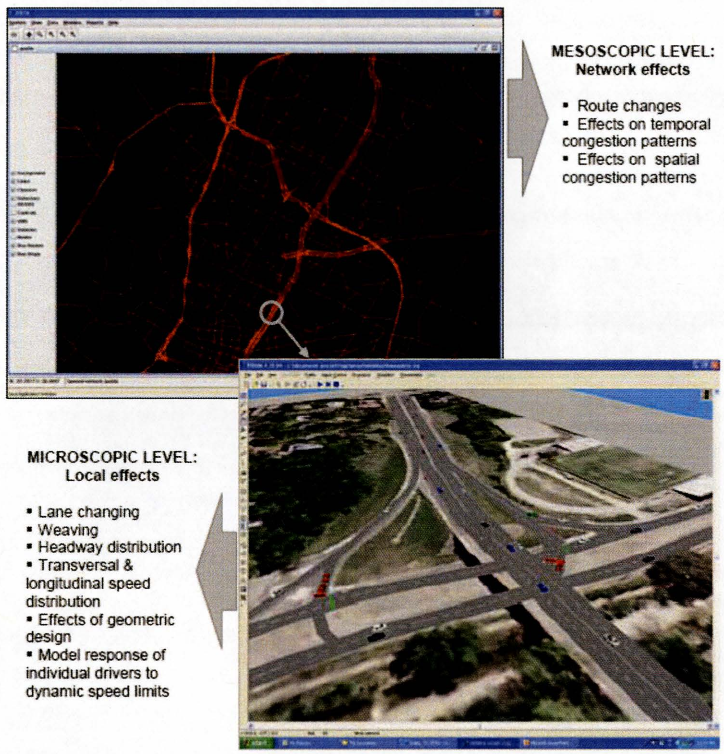


Figure 4.3: *VISSIM/VISTA hybrid analysis framework*

4.2.3 Surrogate Safety Assessment Model (SSAM)

Safety analysis of roadways often requires analyzing police crash reports archived over a long period. Since crashes happen infrequently and assessment of the pre-crash conditions by the police is largely a subjective decision, the statistical models built using the crash reports can take a long time and may contain error.

Under the guidance of the Federal Highway Administration (FHWA), Siemens Corporation has developed a Surrogate Safety Assessment Model (SSAM) which combines microsimuation and automated "conflict analysis" to evaluate safety of traffic flow control strategies (SSAM, 2010). SSAM is suitable for evaluating safety implications of traffic flow control strategies that are yet to be implemented. To evaluate the safety implications of ATM strategies, such strategies first need to be implemented in a microscopic simulation model. SSAM performs "conflict analysis" by using the space-time vehicle trajectories produced by microscopic simulation models implementing various traffic-control strategies. A vehicle conflict is defined as a scenario that will result in a collision if no evasive action were taken by the two vehicles involved (see Figure 4.4).

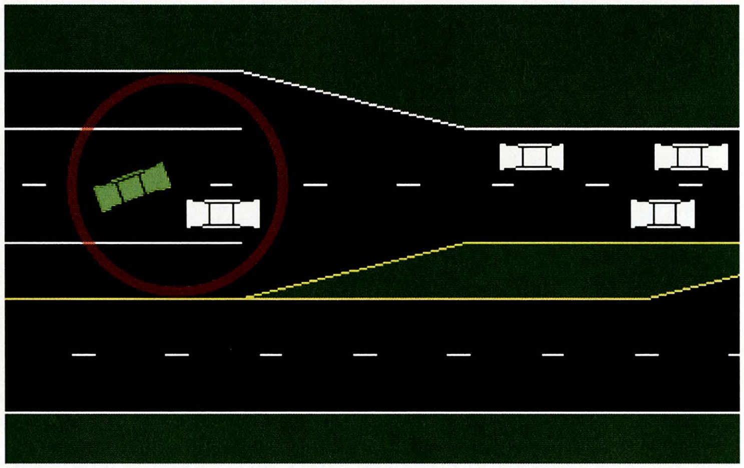


Figure 4.4: *Conflict scenario resulting from a lane change maneuver (SSAM, 2010)*

SSAM calculates several surrogate safety measures using the vehicle trajectory files, and uses these surrogate safety measures to identify and classify the type of conflicts. Examples of surrogate safety measures are: minimum time-to-collision, minimum post-encroachment time, maximum speed differential, maximum speed, etc. SSAM also provides mechanism for statistical comparisons of conflict frequencies and surrogate safety measures for two alternative cases using the student's t-distribution for hypothesis testing. It can also graphically display the location of conflicts on the networks by conflict types, and aids visual analyses of the safety results.

4.3 Traffic Data

4.3.1 Traffic Operations Data

Loop detector data for the MoPac testbed was obtained from the Austin office of the Texas Transportation Institute (TTI). Loop data was obtained at three detector locations situated inside the testbed for the years 2007 and 2008. Exact locations of the detectors and their positions inside the testbed are: 91/2 Street (south), Westover Road (middle), and W 45th Street (north). 15-min period loop data from 2007 were analyzed for 3PM - 7PM period to assess traffic conditions during the evening peak hours. Vehicular density was used to calculate the level of service (LOS) using the Highway Capacity Manual. Since the average percentage of trucks was less than 1.5%, density in veh/mi/ln units is assumed to be very close to that in pc/mi/ln units.

Table 4.1: Sample loop detector data

STN_ID	LANE_QTY	STA_TIME	VOLUME	FLOWRATE	OCC	SPEED	TRUCKS	SPD_DAYS	VOL_DAYS	OCC_DAYS	TRK_DAYS	Year
164	2	0:00	99.989	199.98	0.94	61	1.2811	185	185	185	185	2007
164	2	0:15	85.622	171.24	0.76	61	1.4162	185	185	185	185	2007
164	2	0:30	72.822	145.64	0.62	61	1.1676	185	185	185	185	2007
164	2	0:45	61.848	123.7	0.49	61	1.0217	184	184	184	184	2007
164	2	1:00	53.005	106.01	0.4	61	1.1749	183	183	183	183	2007
164	2	1:15	44.978	89.956	0.31	61	1.0608	181	181	181	181	2007
164	2	1:30	39.783	79.567	0.26	61	1.3556	180	180	180	180	2007
164	2	1:45	36.654	73.307	0.24	61	1.4749	179	179	179	179	2007
164	2	2:00	32.514	65.028	0.22	61	1.3503	177	177	177	177	2007
164	2	2:15	32.282	64.565	0.2	61	1.2768	177	177	177	177	2007

Loop detector data were available at three locations inside the testbed. Exact locations of the detectors and their positions inside the testbed are at the intersection of Mopac with the following streets: 91/2 Street (south end of the testbed), Westover Road (middle of the testbed), and W 45th Street (north end of the testbed). Loop data were obtained from the Austin office of the Texas Transportation Institute (TTI). 15-min period loop data were analyzed for 3PM–7PM period to assess traffic conditions during the evening peak hours. Vehicular density (in veh/mi/ln units) was used to calculate the level of service (LOS) using the Highway Capacity Manual (TRB, 2000). Since the average percentage of trucks was less than 1.5%, density in veh/mi/ln units is assumed to be very close to that in pc/mi/ln units.

Table 4.2: Level of Service for the Testbed

	Density (veh/mi/ln)			Level of Service (LOS)		
Time	9 ½ St (south)	Westover Rd (middle)	45th St (north)	9 ½ St (south)	Westover Rd (middle)	45th St (north)
3 PM - 4 PM	33	36	33	D	E	D
4 PM - 5 PM	47	57	44	F	F	E
5 PM - 6 PM	52	61	50	F	F	F
6 PM - 7 PM	34	43	37	D	E	E

4.3.2 Traffic Safety Data

Traffic safety researchers have developed several statistical models that express safety as a function of traffic flow characteristics. These statistical models are of the form:

$$\text{Safety} = f(X_1, X_2, \ldots, X_n)$$

Safety is modeled as a function of traffic flow characteristics $X_1, X_2, \ldots, X_n$. Where X_i's can be one of the several traffic flow variables: speed, density, speed variability, coefficient of variation of speed, flow, etc. Safety can be expressed as the likelihood of collision, number of crashes/year, number of crashes/vehicle miles traveled (VMT), safety index, or a similar measure. Modeling of the relationship between Safety and X_i's is done similar to linear regression or discrete choice modeling, and it requires large amount of traffic crash data to estimate the coefficients of the X_i's.

Alternatively, the Surrogate Safety Assessment Model (SSAM) can be used to evaluate safety implications of ATM strategies. The SSAM can be integrated with VISSIM, and it will use vehicle trajectories generated by VISSIM to evaluate the safety impact of ATM strategies. Therefore, the SSAM does not require traffic crash reports to perform safety analysis. This approach is ideal for safety evaluation of ATM strategies that have not been implemented in the field yet.

4.4 Calibration and Validation of Testbed Model

VISSIM microscopic simulation model of Mopac Expressway was calibrated for volume and validated using speed values. Origin-destination (O-D) flow matrix (i.e., freeway mainline entrance and exit flows, on-ramp entrance flows, and off-ramp exit flows) and the driver behavior parameters of VISSIM were the two main inputs that required adjustments for the model to replicate real life traffic conditions. Car-following and lane-changing models of VISSIM are robustly calibrated using real life data. Since the base model of Mopac Expressway is a simple freeway with on-ramps and off-ramps, the calibrated parameters of VISSIM were deemed to be appropriate for the testbed model. The researchers decided that if the model was not satisfactorily calibrated after adjusting the O-D matrix, then the microscopic driver behavior parameters would be adjusted to improve the model's fidelity.

4.4.1 Origin-destination Matrix Estimation

The testbed corridor contains only one direction of the freeway (Mopac Expressway northbound) with several on-ramps and off-ramps. Therefore, a traffic zone associated with a ramp is either only supplying vehicles to the freeway mainline or is only receiving vehicles from the freeway mainline. Therefore, all the rows corresponding to off-ramps (i.e., destinations) in the O-D matrix were empty. Similarly, all the columns corresponding to on-ramps (i.e., origins) in the O-D matrix were also empty. This greatly simplified the initial estimation of the O-D matrix and rendered it a linear problem. The detailed steps of the O-D matrix estimation to match loop counts on the freeway mainline are explained in Nezamuddin and Al-Deek (2008).

4.4.2 Model Calibration

Geoffrey E. Heavers (GEH) statistic was used to compare observed loop detector volumes with those obtained from the simulation model. The GEH statistic is a modified chi-square statistic that incorporates both relative and absolute differences. It is designed to compare simulated and observed hourly traffic volumes:

$$GEH = \sqrt{\frac{(simulated - observed)^2}{0.5 \times (simulated + observed)}}$$

For GEH < 5, flows can be considered a good fit; for 5 < GEH < 10, flow may require further investigation; and for 10 < GEH, flow cannot be considered a good fit. The following criteria were established for a successfully calibrated and validated model:

- Volumes should have GEH < 5 for 85% of the checkpoints (FHWA, 2004)

- Simulated volume should have an error of < 5% for 85% of the checkpoints
- Speed should be validated within 5 mph for 85% of the checkpoints.

Table 4.3 presents comparison of observed volumes and simulated volumes obtained from the calibrated microscopic model.

Table 4.3: Calibration results for the testbed

Detectors on Mopac NB	Time	Observed Volume (veh/hr)	Simulated Volume (veh/hr)	(Simulated – Observed) %	GEH
9 1/2 St	4 PM - 5 PM	2448	2515	2.7	1.3
Westover Rd	4 PM - 5 PM	4605	4322	-6.1	4.2
45th St	4 PM - 5 PM	5176	5199	0.4	0.3
9 1/2 St	5 PM - 6 PM	1826	1767	-3.2	1.4
Westover Rd	5 PM - 6 PM	3717	3897	4.8	2.9
45th St	5 PM - 6 PM	4702	4742	0.9	0.6

Table 4.3 displays that the simulation model was satisfactorily calibrated for volumes. After the volumes were calibrated, speed values were satisfactorily validated for all the checkpoints except at the 45th Street during 4PM–5PM, which had an error of 5.5 mph.

Chapter 5. Impact of ATM Strategies on Traffic Operations

Three active traffic management (ATM) strategies were implemented in the testbed using VISSIM microsimulation model to evaluate their impact on freeway's traffic operations: 1) variable speed limits, 2) peak-period shoulder use, and 3) ramp metering. A brief description of their implementation is presented next, followed by a numerical analysis to evaluate their impact on traffic operation using various performance measures.

5.1 Variable Speed Limits

One of the most important determinants of the success of speed harmonization through variable speed limits (VSL) is the control strategy. That is, the control algorithm that determines when and how the speed limit is to be adjusted on a given road segment. Online control algorithms for VSL are based on prevailing traffic conditions and determine optimal speed limit in real time. Online control algorithm for speed harmonization requires a dense deployment of ITS infrastructure for efficient operation. A large body of literature on online VSL control is available. For completeness, we first present a representative sample of the literature on online control strategies.

5.1.1 Variable Speed Limits Algorithms

Online control algorithms are characterized by the use of real-time traffic and environmental data to determine optimal speed limits. In the literature, numerous algorithms ranging from simple and easily implementable to advance and rather fancy rules have been proposed (Sisiopiku, 2001). In the following we only provide a small, but representative, sample to illustrate the typical features of these algorithms.

Washington State and Finland use simple matrices (that are dependent on traffic conditions) to select advisory speeds. In the Netherlands, a two-step algorithm is implemented (Smulders, 1992). In the first stage, a decision on whether speed limit intervention is necessary is made by comparing the one-minute traffic flow volumes with upper and lower boundary values. If speed limit intervention is deemed to be necessary on a section, the new speed limits (typically between 70 and 90 km/h, or 43 and 56 mph) are chosen out of a finite set, after comparing the average of the mean speed of the signal stations of a section with several boundary values. Nevada uses a logic tree to deduce the optimal speed limit from current traffic conditions (using variables such as speed, visibility, and pavement conditions). In the United Kingdom, the speed limit reduces from 70 mph to 60 mph when volume exceeds 1,650 veh/hour/lane. It reduces to 50 mph when the volume exceeds 2050 veh/hour/lane.

More advanced and fancy algorithms have been developed in other instances, for example, based on fuzzy logic (ADOT, 2002). In Lee et al. (2004), a procedure is proposed to dynamically determine the time at which speed limits are to be reduced, i.e., when speed harmonization is to be applied. They use a regression model predicting the expected number of crashes (they called it crash potential) based on factors such as speed differentials and difference in volumes across lanes. When the crash potential is above a certain threshold, the speed limit is reduced to some value in a pre-specified finite set of speed limits. Using microscopic simulation, they found that optimal intervention durations (i.e., the length of the time intervals in which the speed limit is constant should not be too short. They recommended a value between 5 to 10 minutes). It was found that variable speed limits are beneficial for the crash potential. A similar study was conducted in Abdel-Aty et al. (2006). Hegyi et al. (2005) conducted a theoretical study to use speed harmonization to minimize or eliminate shock waves induced by congestion. The model proposed by these authors searched for speed limits resulting in minimum total travel time, subject to a safety constraint preventing speed changes of more than 10km/h at a time. Advanced control theory was used.

What is remarkable about the above sample of theoretical research and real world case studies is that they all report benefits, either for congestion reduction or safety (or both). Of course, different works report numerically different benefits (we believe that this is purely a consequence of different model assumptions or driver characteristics, rather than the superiority of one of the algorithms). Hence this leads us to the conjecture that there does not seem to be significant differences in the proposed control algorithms currently available in the literature. In light of this observation, we suggest the use of simple online control algorithms. Except for the fact that similar (compared to fancy algorithms) beneficiary results have been reported in the literature, these algorithms are simple to implement and, perhaps more importantly, much more transparent to the operators at traffic management centers.

As we have noted, we believe that simple online control algorithms can potentially be as effective as more involved algorithms. Moreover, the majority of *real-life and successful* instances of speed harmonization (as opposed to simulation exercises) employ these relatively simple strategies (e.g., in the United Kingdom). Motivated by these examples, we suggest using a modified version of the control strategy used by Allaby et al. (2007) for Queen Elizabeth Way near Toronto, Canada. This algorithm is presented in Figure 5.1.

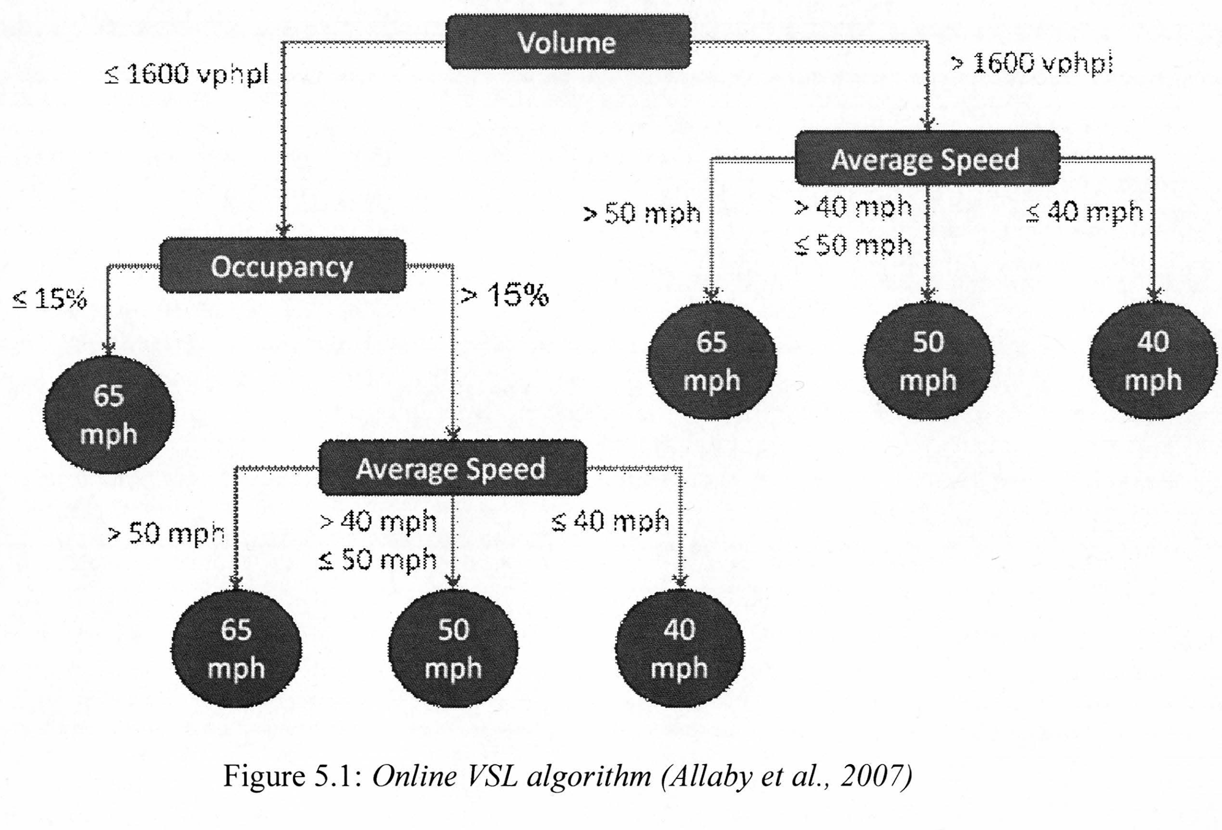


Figure 5.1: *Online VSL algorithm (Allaby et al., 2007)*

Real-time traffic data can now be used to determine the start of modified speed limits. Moreover, due to real-time information, the speed limit can now be easily modified (i.e., increased and decreased) multiple times during the peak-hour period. Of course, this frequency should not be too high in order to prevent "erratically" changing speed limits (Allaby et al., 2007). The basic idea of the algorithm is to examine each bottleneck (starting from the most downstream, and working backwards) and determine the associated optimal speeds upstream of it. Again, in this particular study only one segment is considered. A general framework for online VSL implementation is presented here.

5.1.2 VSL Implementation Framework

Input

- $(\bar{u}_s, q)$-curves for each of the n road segments. Note that we can extract the maximum capacities $c_0(k)$, k = 1, 2, ... n of the road segments from these curves.
- Current speed limits $s_0(k)$, k = 1, 2, ... n of the road segments.
- The minimum intervention duration T_{min}, *i.e.*, the minimum time interval in which the speed limit remains constant.

Output
A set of dynamically changing speed limits for each of the road segments.

INITIALIZATION $c(k) \leftarrow c_0(k), s(k) \leftarrow s_0(k)$

FOR $k = n, n\text{-}1, .., 2$
    IF $q(k) \approx c_0(k)$
        FOR all road segments $r = k\text{-}1, k\text{-}2, \ldots, 1$
            DO select a speed $u(r)$ for segment r using the online VSL algorithm.

            END DO
        END FOR
    END IF
    set $c(r) \leftarrow c_0(r)$
END FOR
Display new speed limit vector $s(r)$
Wait for T_{min} time units, set $s(r) \leftarrow s_0(r)$ and repeat the algorithm.

Online VSL algorithm is implemented in the VISSIM microsimulation model using VISSIM's vehicle actuated programming (VAP) module. An implementation code was written in VAP (see Appendix A), which checked traffic conditions every 5 minutes and selected a speed limit using the algorithm presented here.

Remarks:

- Different values of T_{min} are reported in the literature (e.g., Lee et al. (2004) suggest a value between 5 to 10 minutes; Abdel-Aty et al. (2006) recommends a value of 10 minutes). We adopted T_{min} = 5 min for our purposes.
- Note that when for a given iteration of the algorithm no segment satisfies the condition $q(k) \approx c_0(k)$, then the speed limit will be returned to its original value.
- As in the offline algorithm, the speed selection can be accomplished in various ways, both subjectively (e.g., based on engineering judgment) as well as objectively using the fundamental diagrams. We prefer the latter method and adopted an algorithm such as presented in Figure 4.1.
- Note that in a given iteration, $s(k)$ is a non-increasing sequence. That is, once the speed limit of a given section has been lowered, the algorithm ensures that (in the same iteration) subsequent speed limit modifications are such that the speed limit can only be further reduced on the section in question. This prevents that the lowering of speed limits in support of downstream bottleneck i are cancelled when we consider bottleneck i-m, where $0 < m < i$.
- If we made the additional assumption that visibility information and pavement conditions are known, we can also incorporate this information into the proposed algorithm by imposing rules on the speed selection procedure (e.g., "if visibility is less than x feet, then set speed limit equal to y mph").

5.2 Peak-period Shoulder Use

For the same reasons as in variable speed limits, we propose to use simple and transparent control strategies to govern temporary shoulder use. Furthermore, as recommend by FHWA (2007), we assume that the shoulder is used only when speed harmonization is active. Before we present the specifics of the proposed control algorithm, let us discuss what we can anticipate from the use of the shoulder from a traffic flow theory perspective. A typical speed-flow curve is given in Figure 5.2. This speed-flow curve fully determines the maximum capacity of the road segment under consideration. The use of the shoulder lane is equivalent to the addition of capacity. Hence we can anticipate that the maximum capacity (i.e., the right most point on the speed-flow curve) shifts to the right. Moreover, by assumption, the speed limit is lowered, which results in the downward translation of the speed-flow curve. This process is illustrated for a different speed-flow curve in Figure 5.3. As can be seen, the curve expands to the right and contracts in the vertical direction when the speed limit is decreased to 100 km/h (62 mph). The resulting capacity is increased from about 3300 vph to more than 500 vph.

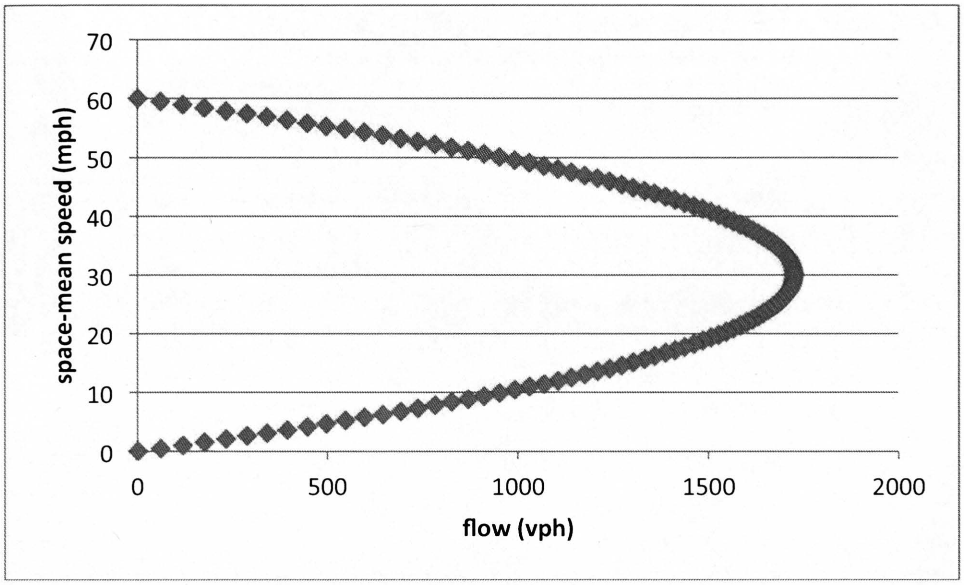


Figure 5.2: *Typical speed-flow curve*

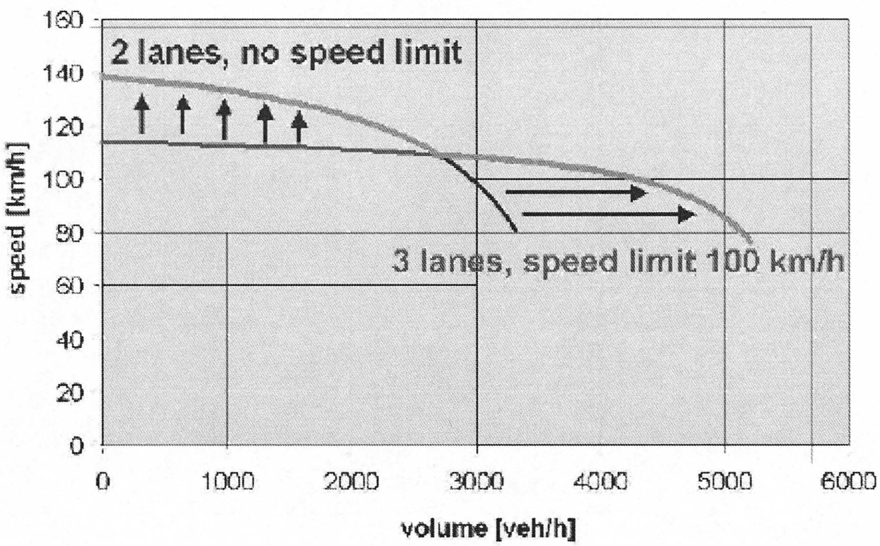


Figure 5.3: *Change of the speed-flow curve due to temporary shoulder use (FHWA, 2007)*

The generic pseudo-code for the proposed control algorithm is given as follows (recall that we assume that speed harmonization is active when calling this procedure).

5.2.1 Peak-period Shoulder Use Implementation Framework

Step 1. Check if shoulder lane is free of objects. If the shoulder lane is free, go to Step 2; otherwise, repeat Step 1 after some time.

Step 2. Open shoulder lane for traffic.

Step 3. If the average flows on the lanes are less than a pre-specified value, then close the shoulder lane.

Again, shoulder use is implemented in conjunction with both offline VSL and online VSL. As was seen in Table 4.2, traffic level of service (LOS) during the study period of 4:00 p.m.–6:00 p.m. is "F" for most part, and it is clear that this section of Mopac is severely lacking capacity. Therefore, a decision was made to open the shoulder for the entire duration of the study. In this case study, we considered temporary use of the shoulder between Enfield Road and West 35th Street. Heavy vehicles were barred from using the shoulder.

5.2.2 Design Guidelines for Shoulder Lanes

Recommended design guidelines for shoulder lanes during peak traffic congestion periods are discussed in this section. These recommendations were derived using the following references: the *Texas Department of Transportation (TxDOT) Roadway Design Manual*, AASHTO *Policy on Geometric Design*, *Manual on Uniform Traffic Control Devices*, and previous case studies in which shoulders were used during peak congestion periods. The following recommendations are based on the assumption that shoulders will be used as travel lanes to manage congestion when operating speeds on freeways are 35 mph or less. This assumption is key to allowing for smaller dimensions for lane and shoulder widths than typically used for freeway segments.

Topics discussed in the following sections are typical freeway cross-section dimensions, shoulder lane width, width for the "acting" shoulder (i.e., the shoulder when the shoulder lanes are in use), pavement design for the shoulder lane, transition areas, entrance/exit ramps, accommodations for incident management, and special considerations. The dimensions and geometric considerations discussed are written from a broad perspective. Each candidate site will need to be reviewed in detail with specific design plans developed to ensure that freeway geometry, when shoulders are used as travel lanes, remains consistent with driver expectations and fits into the self-explaining roadway design.

5.2.3 Typical Freeway Cross-Section Dimensions

The *TxDOT Roadway Design Manual*, Chapter 3, Section 6 presents design criteria for freeways. The minimum lane width typically used is 12 feet. The shoulder widths vary depending on number of lanes. A four-lane facility has an inside (i.e., left-hand side) shoulder width of 4 feet and an outside shoulder width of 10 feet. A six or more lane facility has an inside and outside shoulder width of 10 feet. These dimensions along with the expected freeway operating speeds under congested periods were taken into consideration for developing the basic criteria for the shoulder lanes. These are discussed in more detail.

Shoulder Lane Width

As noted in the introduction, freeway operating speeds (i.e., 85th percentile speeds) are expected to be 35 mph or less when shoulders are used for travel lanes. These slower operating speeds allow for the shoulder lanes to be less than the typically required 12-foot width for travel lanes. Due to the slower operating speeds and change in character of the freeway under congested periods, the design criterion for shoulder lanes were developed from TxDOT's standards for freeways, urban arterials, and suburban arterials.

Based on TxDOT Roadway Design Manual for urban and suburban arterials (Chapter 3, Sections 2 and 3) and the assumed operating conditions (i.e., speeds 35 mph or less), a minimum lane width of 10 feet could be considered permissible for the shoulder lanes, if heavy vehicle volumes are low or heavy vehicles are restricted from using the shoulder lanes. To allow heavy vehicles to use the shoulder lanes, a width of at least 11 feet is desirable. A width of 11 feet for the shoulder lanes would provide approximately a 15-inch lateral distance on both sides of a typical heavy vehicle (width of 8.5 feet) between the vehicle and the edge of the shoulder travel lane.

Acting Shoulder Width

Acting shoulders are the resulting shoulders when the inside and/or outside shoulders are used as travel lanes. Based on the current design standards for shoulder widths already noted and recognizing the shoulders will be used as travel lanes during peak congested periods, we recommend a 2- to 4-foot acting shoulder (i.e., *shy distance*) measured from the edge of the shoulder lane to the edge of the paved cross-section. The 2- to 4-foot acting shoulder will provide lateral support to the shoulder lane as well as a shy distance or lateral buffer zone for vehicles to avoid medians, drainage facilities, and other roadside features beyond the paved cross-section.

Pavement

We recommend upgrading the structural composition of the pavement for the shoulder lanes to be consistent with the mainline lanes particularly if the shoulder lanes will be used during peak hours each weekday (i.e., shoulder pavement should be full depth). In highly congested areas, this could be as frequent as 4 to 6 hours per day, 5 days per week. At most this would account for 65 days in one year or 1/5 of the calendar year; therefore, upgrading the structural integrity of the shoulder lanes could be accomplished through an existing freeway maintenance schedule. The slope of shoulders should be 2% or less for driver comfort.

Transition Areas

Transition areas are the areas in which the shoulders are added (i.e., opened to vehicles for use) as travel lanes and dropped as travel lanes. We recommend using a taper rate of 10 to 1 to add the shoulder lanes for use in peak periods (i.e., open the shoulder for use at a rate of one lateral foot for every 10 feet traveled). This would effectively open the shoulder for use over a distance of 100 feet. We recommend using a taper rate of 50 to 1 to drop (i.e., close) the shoulder. This rate is consistent with the taper rates cited in the *TxDOT Roadway Design Manual*, Chapter 3, Section 6 for merging traffic from entrance ramps with the freeway mainlines. Using this rate to drop the shoulder lanes as travel lanes provides sufficient time for drivers to merge back into mainline traffic as congested periods dissipate and operating speeds exceed 35 mph; it also provides a reasonable transition when shoulder lanes must be dropped due to changes in geometry such as a narrow bridge deck where shoulders cannot be used as travel lanes.

A key consideration for transition areas is the means of enforcing the opening and closing of shoulders as travel lanes. Where shoulder lane use is repetitive during the peak hours, overhead gantries with lane use symbols are commonly used to denote whether or not the shoulder lane is open as a travel lane. However, without physical delineation or self-enforcing roadway geometry there is no guarantee the lane use symbols will be consistently obeyed. Part 6 of the *Manual on Uniform Traffic Control Devices* (MUTCD) presents temporary traffic control devices typically applied in work zones but potentially applicable in this context. For example to close the shoulder lane for use as a travel lane, an arrow panel capable of displaying chevrons indicating the need to merge into mainline traffic could be placed 200 to 500 feet in advance of the start of the taper with temporary channelizing devices placed along the taper. This configuration could be used in conjunction with overhead gantries displaying the appropriate lane use symbol.

Entrance/Exit Ramps

Freeway entrance and exit ramps, used in conjunction with the outside shoulder lane as a travel lane, present a challenging geometric environment that will be unique to each candidate site. Discussed here are some key considerations and potential solutions that can be used as starting points when considering the design and implementation for specific freeway corridors. The primary conflict related to shoulder use and entrance/exit ramps is the location where entrance ramps merge into the mainline. The interface between the gore point, the merge point, and the vehicles using the shoulder as a travel lane is a critical point of conflict at entrance ramps. The point where exiting traffic diverges from mainline traffic onto an exit ramp is operationally of less concern because vehicles are diverging rather than merging. Typically, entrance ramps are on the right-hand side therefore the outside shoulder lane will be of primary consideration; however, if there is a left-hand side entrance, the same considerations discussed would apply to the inside shoulder lane operations.

The conflict point at the entrance ramps and the outside shoulder is illustrated in Figure 5.4. The green arrows represent vehicles traveling on the outside shoulder and the red arrows represent vehicles entering on the on-ramp.

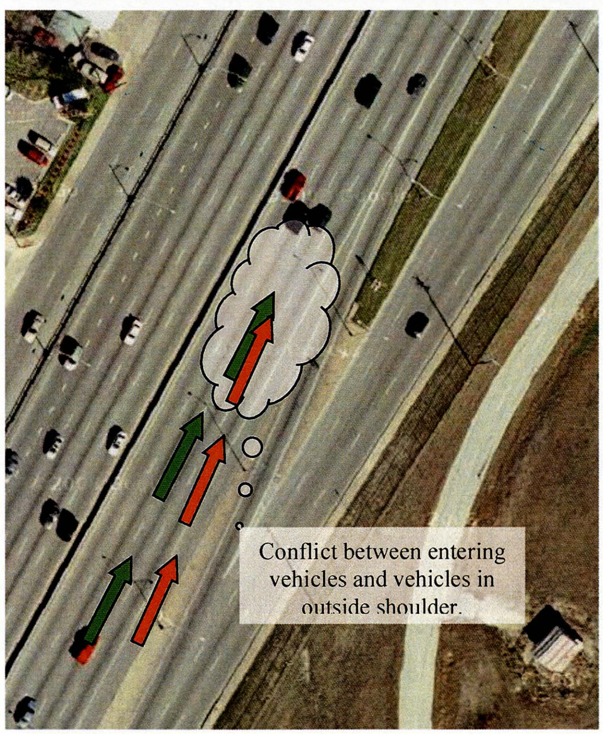
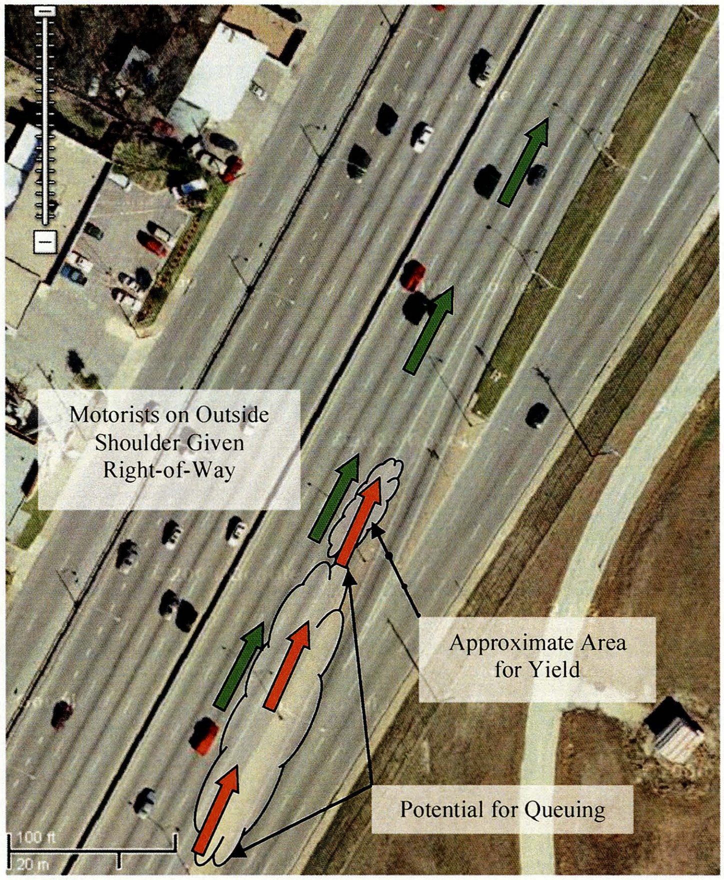


Figure 5.4: *Conflict between entering vehicles and vehicles in outside shoulder lane*

The conflict occurs as the entrance ramp merges into the outside shoulder creating a painted gore point where the entrance ramp lane becomes parallel with the mainline. To successfully be able to use the outside shoulder lane, right-of-way needs to be given to the vehicles traveling in the outside shoulder with vehicles merging onto the freeway yielding from the ramp. This would allow the vehicles in the outside shoulder lane to continue moving with mainline traffic. The vehicles in the outside shoulder lane would travel across the painted gore or merge area through the parallel auxiliary lane typically used as an entrance/exit lane and then back onto the painted shoulder once past the exit accompanying the entrance. Figure 5.5 illustrates this operation.



Operations with vehicles travelling in the outside shoulder

Some key considerations related to this solution are communicating these operations to the motorists and ensuring the motorists entering the freeway have ample sight distance and acceleration distance to join mainline traffic. These considerations are discussed in the following section.

5.2.4 Communicating Expected Operations to Motorists

If the outside shoulder were used as a travel lane today, the existing geometry, signs, and pavement markings would not intuitively indicate to drivers that vehicles traveling in the outside shoulder lane are permitted to drive through the painted entrance merge areas across the entrance/exit auxiliary lane and back onto the outside shoulder while entering vehicles yield to them. Potential options to communicate these operations are either static or changeable message signs, pavement markings, physical delineation markers, and/or ramp meters.

Signing

Static or changeable message signs appear the most applicable in the near term as they can be used on a temporary basis without physically impeding or changing freeway operations during off peak hours. The MUTCD does not currently contain any static signs applicable to peak period shoulder use; therefore, using changeable message signs mounted on either mobile trailers or permanent structures is one of the more readily available means to communicate with motorists in the outside shoulder lane and those on the on ramp. New traffic control devices specific for peak period shoulder use could be explored in the long term to further improve operations.

Potential locations for these signs are noted here.

- Locations for Signs for Motorists on the On-Ramp:
  - One changeable message or static sign placed shortly after the entrance to the on-ramp; and
  - A second changeable message or static sign placed at the location where vehicles on the ramp should yield.
- Locations for Signs for motorists on the Mainline:
  - Changeable message or static signs placed such that vehicles in the outside shoulder lane are able to clearly see the sign;
  - One changeable message or static sign 250 feet in advance of the entrance ramp merge; and
  - A second sign 125 feet in advance of the entrance ramp merge.

These sign locations would provide motorists on the mainline two opportunities to see and comprehend the messages. At travel speeds of approximately 35 mph, motorists would be given nearly 5 seconds from the first sign and approximately 2.5 seconds from the second sign to perceive and react to the information they're given.

This is consistent with the industry standard 2.5 seconds for driver perception-reaction time. Signing plans for candidate sites should be reviewed on a site-specific basis to ensure new signs are appropriately integrated with existing signs to prevent overloading the drivers with too much information.

Ramp Meters

If already present on the ramps, ramp meters can be used to stop oncoming traffic on the ramp, providing the right-of-way to vehicles using the outside shoulder lane. For locations without ramp meters, their addition could be an effective long-term traffic control device used with static or changeable message signs. Ramp meters could be used as a "Phase 1" treatment to manage congestion; using the shoulders as travel lanes could be "Phase 2" of congestion management. During off-peak periods, the ramp meters would be turned off and lay dark to allow for typical free flowing merge movements.

Enforcement and Education

When initially implementing peak period shoulder use for outside shoulders, it would be beneficial to have law enforcement physically present at the entrance and exit ramps to ensure motorists understand and follow the adjusted freeway operations. Similarly, a public education campaign via short TV commercials, notices on the TxDOT website, and public flyers via mail regarding the new operations during peak congestion periods would help familiarize motorists with the new operations.

Sight Distance and Acceleration

The operations discussed earlier change the merge conditions for entering vehicles from a free flowing movement to one requiring entering vehicles to slow sufficiently to yield and possibly to stop, look for an acceptable gap, and accelerate to a speed comparable to mainline operating speed to merge. These operations will only be applicable under congested periods when operating speeds are lower than typical freeway operating speeds, which will help ease the driving task for entering vehicles. These tasks also become easier when a ramp meter is present to help identify when a motorist should proceed to enter mainline traffic. However, for each entrance ramp along a candidate freeway corridor, the location of where entering vehicles should yield (or stop) will need to be determined based on the available sight distance from the ramp, whether or not the ramp will be controlled by a ramp meter, and the maximum anticipated operating speed under which the outside shoulder will be used as a travel lane.

In general, the yield (or stop) point for entering vehicles should be located where sight distance is sufficient to see a gap and acceleration distance is sufficient to reach a speed comparable to mainline traffic. Based on *TxDOT Roadway Design Manual*, Chapter 3, Section 6, freeway ramps should enable vehicles to leave and enter the mainline freeway at a speed of at least 50% of the freeway's design speed. Applying that specific design standard to this particular instance, the yield point on the on-ramp should be located to allow sufficient distance for motorists to accelerate to at least 50% of the maximum expected freeway operating speeds (e.g., 35 to 40 mph) when the outside shoulder is used as a travel lane.

5.2.5 Summary of Entrance/Exit Ramps

The primary concern regarding entrance/exit ramps and the use of the shoulder as travel lanes is the point of conflict when an entrance ramp merges into a shoulder travel lane. To reconcile this conflict, the motorists in the shoulder travel lane would need to be given right-of-way to proceed with mainline traffic and the vehicles entering the freeway would need to yield from a position on the entrance ramp. To communicate and enforce these modified operations, a combination of traffic control devices (e.g., changeable message signs, ramp metering), police presence, and public education are recommended. Each entrance/exit ramp location along each candidate freeway corridor will present unique geometry that will need to be reconciled through design plans specific to each site.

5.2.6 Incident Management

Incident management, as discussed here, consists of 1) providing a travel-way for emergency vehicles to access disabled vehicles or to serve people in need of medical attention; and 2) providing a refuge separate from the mainline travel-way for disabled vehicles to wait for assistance. Guidelines for incident management during peak period shoulder use are discussed in this section. As noted in previous sections, each candidate freeway corridor will present unique circumstances that may ease or complicate the task of providing for emergency vehicle access and disabled vehicle refuges.

Peak period shoulder use can occur under three basic conditions: 1) on a four-lane freeway with the outside shoulder being used as a travel lane; 2) on a six-lane freeway with only one shoulder used as a travel lane (either inside or outside); or 3) on a six-lane freeway with both shoulders used as travel lanes. Under scenario 2), one shoulder (either the inside or outside shoulder) is available to use as a refuge area for disabled vehicles and as a travel lane for emergency vehicles. Therefore, the following discussion will focus on scenarios 1) and 3) where the shoulder(s) typically available for emergency vehicle use and disabled vehicles are being used as travel lanes.

Emergency Vehicle Access

Emergency vehicle access should be reviewed on a case-by-case basis with access options for each candidate site identified based on the geometry and freeway access. Several potential solutions are discussed in this section.

In the presence of monitoring equipment, dynamic lane assignment symbols, and changeable message symbols, a lane or set of lanes can be closed to vehicles at the time of an incident, clearing a travel way for emergency vehicles.

Freeways with breaks in the center median would allow emergency vehicles to access an incident by traveling on the freeway section designated for the opposite direction, and crossing over to access the incident.

In the absence of these unique characteristics, emergency vehicle access can be provided by ensuring slopes adjacent to the freeway (i.e., just beyond the edge of travel way) are rounded and change at a rate of 1 vertical foot to every 6 horizontal foot (1V:6H) or flatter. This slope is negotiable by vehicles and could be used by emergency vehicles as a travel way.

There may be additional viable solutions depending on the unique characteristics along the freeway corridor.

Disabled Vehicles Refuges

Vehicle refuges provide a location for disabled vehicles to exit the travel way, call for service via a call box, and wait for service or help to arrive. Previous instances where shoulders are used as travel lanes during peak periods were accompanied by refuge areas approximately every 1/3 of a mile. A refuge area of 15 feet in width and 150 feet in length using a taper rate of 1 to 15 (i.e., develop and merge the refuge area over a distance of 225 feet) would provide sufficient space for multiple disabled vehicles and accompanying service vehicles.

5.2.7 Additional Considerations

Discussed here are additional considerations related to the design and corresponding operations of shoulder travel lanes. These considerations include the impact of horizontal curves, providing sufficient vertical clearance from overhead infrastructure, providing sufficient lateral distance from roadside objects and/or mitigating insufficient distances, freeway operations when the peak periods occur in the dark, and using intelligent transportation systems to monitor freeway operations.

Horizontal Curves

Candidate freeway corridors with frequent or long horizontal curves may require additional reconstruction (compared to tangent stretches of freeway) to accommodate use of shoulder lanes as travel lanes. Depending on the design speed of the facility and the radius of the horizontal curve, the superelevation of the inside or outside shoulder may be too steep to safely serve as a travel lane or may not be sufficiently wide to serve as a travel lane. Horizontal curves will need to be reviewed on a case-by-case basis to ensure the superelevation and cross slopes for the shoulders (in addition to the other characteristics discussed) are suitable for carrying traffic.

Vertical Clearance

Each bridge, gantry, or other structure passing over the top of a freeway will need to be reviewed to ensure sufficient vertical clearance exists over the shoulder lanes if they are to be used as travel lanes. *TxDOT Roadway Design Manual* specifies a minimum vertical clearance of 16.5 feet over the useable roadway. To use the shoulders as travel lanes, this clearance will need to be confirmed for each piece of overhead infrastructure.

Horizontal Clearance

Horizontal clearance is the distance from the edge of travel way to the nearest fixed object. When the shoulders are used as travel lanes, the horizontal clearance from the edge of the shoulder lane to the nearest fixed object should be reviewed to ensure the distance meets standards. If the distance does not meet standards, the fixed object may be moved or mitigations (e.g., crash cushions) may be identified and implemented to lessen the severity of a vehicle hitting the fixed object. *TxDOT Roadway Design Manual* cites a minimum horizontal clearance of 30 feet for freeway mainlines and 16 feet for freeway ramps in the absence of a barrier or other treatment of safety appurtenances (Chapter 2, Section 6). AASHTO's *Roadside Design Guide* presents potential roadside barriers and other safety appurtenance treatments to mitigate instances of insufficient horizontal clearance.

Freeway Operations in the Dark

During winter months, the traditional commuting peak hours do not occur in full daylight; therefore it is conceivable that the use of shoulder lanes will occur at times when drivers' visibility is relatively limited. The traffic control devices used to communicate the modified operations while peak shoulders are in use should meet night-time visibility standards as outlined by the MUTCD. Candidate sites should also be reviewed to ensure sufficient lighting is provided along the freeway corridor; adjustments, maintenance, or upgrades may be needed depending on the site.

Intelligent Transportation Systems

Intelligent transportation systems (ITS) and tools can play a beneficial role in effectively implementing, monitoring, and ensuring safe freeway operations during peak periods when shoulder lanes are in use. Traffic monitoring devices such as video cameras and loop detectors are beneficial to monitor freeway operating speeds, identify and respond quickly to incidents, and measure the level of congestion present on the freeway. This information plays a critical role in determining when, where, and for how long shoulders are used as travel lanes. This ITS infrastructure requirements are discussed in further detail in Chapter 7. To achieve an effective system, the ITS infrastructure will need to be integrated to complement the roadway geometry and other traffic control devices present.

5.3 Ramp Metering

Various ramp metering implementation studies have been presented in Chapter 2. For the purpose of this study, Asservissement Linéaire d'Entrée Autoroutière (ALINEA) linear control algorithm for ramp metering was selected due its ease of implementation and efficiency in improving traffic conditions ((Papageorgiou et al., 1991). The basic idea behind ALINEA is to maintain an optimal occupancy on the mainline that will maximize the throughput. It was developed by the application of classical feedback theory that can be stated as:

$$r(k) = r(k-1) + K_R\lfloor o_{des} - o_{out}(k)\rfloor$$

Where:

o_{des} – desired occupancy;

$o_{out}(k)$ – the measured occupancy in the mainline section during time interval k ;

$r(k)$ – the metering rate for time interval k;

K_R – a regular parameter.

If the measured occupancy at cycle k is found to be lower (higher) than the desired occupancy o_{des}, $r(k)$ is increased (decreased) as compared with $r(k-1)$. This algorithm reacts smoothly even to slight differences between o_{des} and $o_{out}(k)$ and may thus prevent congestion in an efficient manner, stabilizing traffic flow at a higher throughput level.

It has been proved that ALINEA is not sensitive with respect to K_R, so we use the K_R value of 70, which is the most often used value in other ramp metering researches. We also set the desired occupancy equals to 20%, which can be adjusted according to local conditions. Ramp metering was implemented on the first two ramps on Mopac northbound in the testbed model: $1^{st}/6^{th}$ street on-ramp and Enfield Rd. on-ramp.

5.4 Numerical Analysis of ATM scenarios

To test the effectiveness of active traffic management strategies, five different models were developed (including the base case). The following four ATM scenarios were compared with the base case model to evaluate their impact on traffic operations of the freeway:

1) Variable speed limits: VSL was implemented in a 2.5-mile section of Mopac Expressway (northbound) between Enfield Rd. and W 45^{th} Street.
2) Peak-period shoulder use: Left shoulder use was implemented in a 1.5-mile section of Mopac Expressway (northbound) between Enfield Rd. and W 35^{th} Street.
3) VSL and shoulder use: both VSL and shoulder use were implemented simultaneously.
4) Ramp Metering: Ramp Metering strategy ALINEA was implemented on two on-ramps.

Performance measures were obtained from the VISSIM microscopic simulation model of testbed to evaluate the impact of ATM strategies on traffic operations. Each of the five scenarios (including the base case) was run using ten different random seed numbers. Differences in traffic operations measures between the ATM scenarios and the base case were tested for significance using the T-test. Tables 5.1-5.4 present the comparison results.

Nine traffic operations performance measures, listed in the first column of the tables, were chosen to compare the various ATM scenarios. Throughputs and speeds were measured in the middle (Westover Rd.) and at the end of the VSL corridor (West 40th St.) for the duration of study. Vehicular delays were obtained for both the entire Mopac Expressway model and just the VSL corridor. Similarly, coefficient of variation (c.o.v) of speed and density were obtained from loop detectors embedded in the simulation model.

The first column for each of the four ATM scenarios (VSL, Shoulder use, VSL & Shoulder use, and Ramp metering) reports average values of the performance measures across ten simulation runs. The second column reports percentage difference between the base case and the ATM scenario, along with p-values for the T-test in parenthesis. A p-value of less than 0.05 indicates that the difference in performance measure between the base case and the ATM scenario is significant at 95% confidence level.

Table 5.1: Base case vs. variable speed limits (VSL)

	Base case	VSL	
	Average value	Average value	% change from the base case (p-value)
Throughput-mid (veh)	8458	8190	-3% (8.3E-05)
Throughput-end (veh)	10406	10104	-3% (8.3E-05)
Speed-mid (mph)	25	20	-18% (7.6E-06)
Speed-end (mph)	33	27	-18% (2.1E-06)
# Stop/Vehicle	17	17	-1% (7.2E-01)
Delay-network (sec/veh)	72	77	7% (6.1E-02)
Delay-corridor (sec/veh)	363	260	-28% (8.2E-07)
Speed c.o.v	0.54	0.49	-10% (4.0E-02)
Density (veh/mi)	91	87	-4% (6.4E-01)

Table 5.2: Base case vs. shoulder use

	Base case	Shoulder Use	
	Average value	Average value	% change from the base case (p-value)
Throughput-mid (veh)	8458	8661	2% (2.9E-05)
Throughput-end (veh)	10406	10706	3% (1.4E-05)
Speed-mid (mph)	25	49	98% (5.3E-09)
Speed-end (mph)	33	26	-22% (5.6E-05)
# Stop/Vehicle	17	15	-11% (6.1E-02)
Delay-network (sec/veh)	72	56	-22% (8.6E-04)
Delay-corridor (sec/veh)	363	193	-47% (5.4E-07)
Speed c.o.v	0.54	0.14	-74% (1.5E-07)
Density (veh/mi)	91	27	-70% (1.4E-05)

Table 5.3: Base case vs. VSL & shoulder use

	Base case	VSL & Shoulder Use	
	Average value	Average value	% change from the base case (p-value)
Throughput-mid (veh)	8458	8591	2% (6.2E-04)
Throughput-end (veh)	10406	10556	1% (6.4E-03)
Speed-mid (mph)	25	26	3% (1.1E-01)
Speed-end (mph)	33	21	-36% (3.6E-07)
# Stop/Vehicle	17	15	-13% (2.2E-02)
Delay-network (sec/veh)	72	57	-21% (3.7E-04)
Delay-corridor (sec/veh)	363	152	-58% (7.8E-09)
Speed c.o.v	0.54	0.05	-91% (2.7E-07)
Density (veh/mi)	91	59	-35% (3.5E-03)

Table 5.4: Base case vs. ramp metering

	Base case	Ramp metering	
	Average value	Average value	% change from the base case (p-value)
Throughput-mid (veh)	8458	8357	-1% (1.79E-02)
Throughput-end (veh)	10406	10270	-1% (1.14E-02)
Speed-mid (mph)	25	21	-16% (7.39E-
Speed-end (mph)	33	28	-15% (3.37E-
# Stop/Vehicle	17	14	-18% (1.26E-03
Delay-network (sec/veh)	72	301	318% (1.12E-04)
Delay-corridor (sec/veh)	363	280	-23% (2.19E-09)
Speed c.o.v	0.54	0.46	-15% (8.20E-04

5.4.2 Multi-resolution Analysis

Variable speed limits and shoulder use have immediate effect on traffic condition in the test corridor. These effects are captured by a microsimulation model, and are presented in previous sections. The capacity of the test corridor changes due to implementation of these strategies. Change in freeway capacity affects route choice of users, and thus VSL and shoulder use have a larger network-wide impact. A multi-resolution analysis was performed to determine the network level effect of these strategies.

Network level effects are studied by mesoscopic models, which are better suited for large scale network analysis and are less computationally intensive. The mesoscopic model used in this research is a cell-transmission based model, and it takes three inputs to perform network analysis: travel demand data, roadway capacity, and free-flow speed. Network level impact of VSL and shoulder use has been quantified by making an equivalent change in roadway capacity of the test corridor inside the regional Austin network in the mesoscopic model (Figure 5.6).

The results obtained after VSL and shoulder use implementation (Tables 5.3) indicate that there was no significant increase in throughput of freeway in the test corridor. Because the freeway is operating at or over capacity during the period of simulation (Table 4.2), it can be concluded that these strategies did not increase the capacity of the test corridor. While there was no impact on capacity due to these strategies alone, the opening of shoulder to traffic added physical space, and hence capacity, in the shoulder use section. Therefore, addition of capacity due to shoulder use was incorporated in the regional mesoscopic model of the Austin network (Figure 5.6).

Two scenarios were simulated in the mesoscopic model: base case and VSL & shoulder use implementation. Link flows for these two cases were extracted and their difference (VSL & shoulder use minus base case) is plotted in Figure 5.6. Links with thicker and darker color indicate that there was increase in their flows when VSL and shoulder use were implemented in the test corridor.

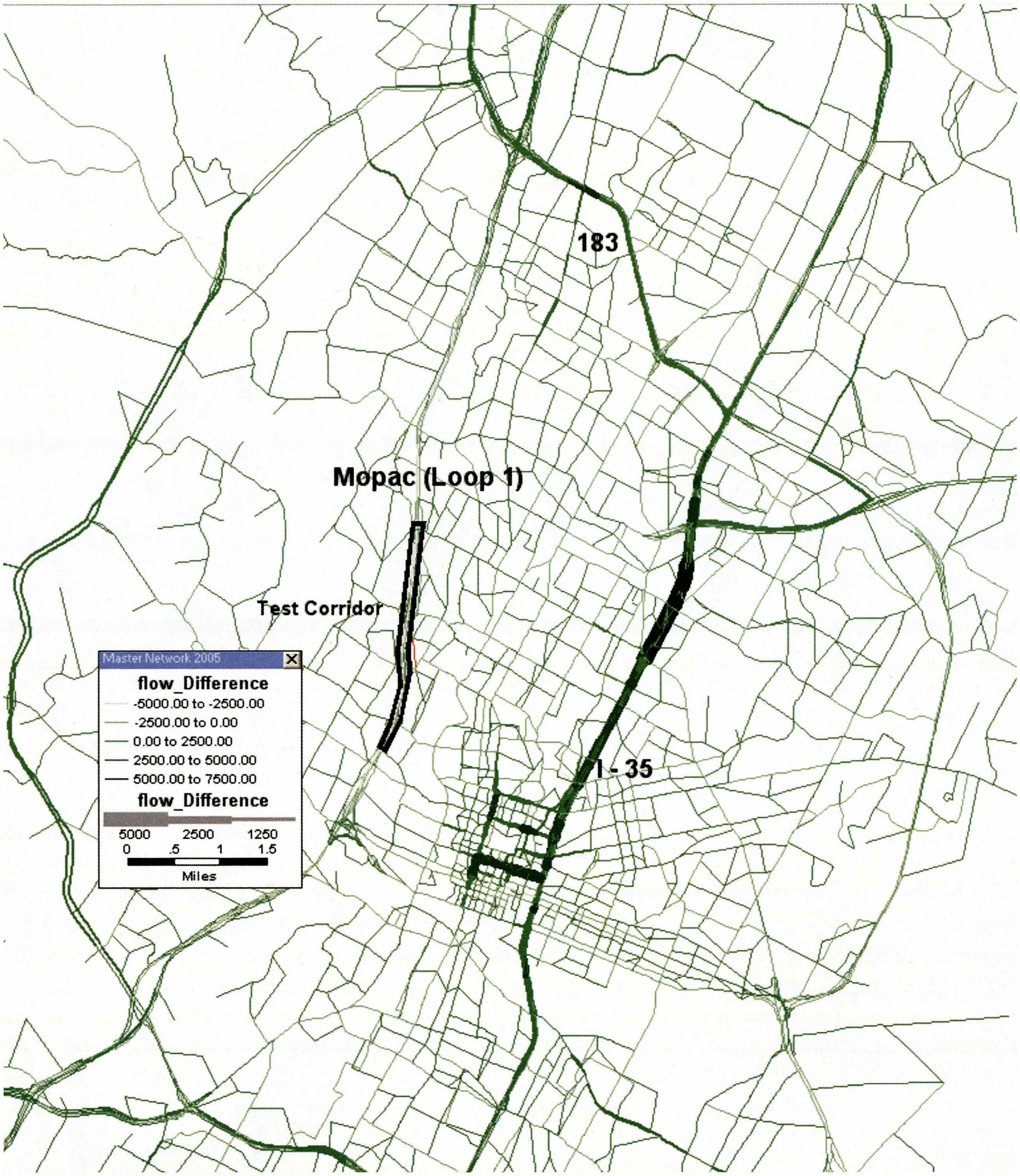


Figure 5.5: *Network-wide effect on link flows (VSL & shoulder use—base case)*

The results indicate that there was an increase in traffic using IH 35 near downtown and further up north. Correspondingly, traffic flow on arterials feeding to IH 35 in the downtown area has also increased. Mopac (Loop 1) and IH 35 are two major north-south corridors in the Austin metropolitan area. This implies that more traffic is using IH 35 to travel to and from the northern part of the city. This may be due to the adverse effect on travel time and speed that shoulder use had on traffic in the test corridor. Shoulder use created bottleneck toward the end of shoulder-use section due to sudden reduction in freeway capacity, and it led to a 36% decrease in speed there (Table 5.3). The adverse effect on flow in the test corridor may also be compounded by shorter shoulder use section in the test corridor. If shoulder is open to traffic from downtown in the south to US 183 in the north, then the test corridor is more likely to be an attractive option for travelers. In such cases, benefits obtained from better driving conditions in a longer shoulder use section are likely to outweigh the effect of bottleneck at the end of the shoulder use section.

The results obtained from multi-resolution analysis indicate that when shoulder is open to traffic for a short length, and without mitigating the negative impact of the bottleneck created, it may make the VSL and shoulder use section less attractive. However the research team cautions that a more detailed analysis should be performed before drawing strong conclusions for the network-wide impact on VSL and peak-period shoulder use. This analysis is presented to emphasize the importance of performing network-level analysis. Since the primary focus of this study is to evaluate safety impacts of ATM strategies, a detailed study of network-level analyses is not presented here.

5.5 Results and Conclusions

ATM strategies had little effect on throughput. This is consistent with the goal of VSL to harmonize vehicles' speed and smooth traffic flow; some high-speed vehicles are forced to decrease their speed and this resulted in a slightly lower throughput. The throughput results are also in accordance with the earlier findings of Abdel-Aty et al. (2006) and Papageorgiou et al. (2008), which reported little to no effect on throughput due to VSL. Although peak-period shoulder use increases capacity of roadways, a small increase in throughput is a result of the following: combined effect of VSL and shoulder use, constrained by demand and bottlenecks at the beginning and end of the shoulder use segment. Speed harmonization effects of VSL can be observed from the reduced operating speeds and reduced coefficients of variation of speed.

VSL decreased speed in the middle of the test section. Shoulder use increased speed in midsection by reducing traffic density due to additional space available. The effect of the simultaneous implementation of VSL and shoulder use on speed is a combination of the two opposite effects. Speed consistently decreased at the end of the shoulder use segment due to the bottleneck creation as a result of one lane-drop.

VSL and shoulder use consistently harmonized the traffic flow and improved travel condition by reducing the speed variability, reducing traffic density, and reducing stop-n-go traffic (by reducing the number of stops per vehicle). Reductions in these performance measures also allude to creation of safer driving conditions. The two ATM strategies implemented simultaneously improved travel time at the network level and corridor level.

In conclusion, VSL harmonized traffic flow, reduced lane-changing conflicts, and created safer driving condition. Simultaneous implementation of VSL and shoulder use consistently harmonized the traffic flow and improved travel condition by reducing speed variability, traffic density, and stop-n-go condition (by reducing the number of stops per vehicle). It did not have significant impact on throughput, as is reported in previous studies on VSL. It had an overall positive effect on speed harmonization for the traffic. It was found that VSL and shoulder use decreased delay per vehicle, and overall these strategies created safer driving condition by reducing speed variability.

Ramp metering reduced the average number of stops per vehicle and speed variation. It also reduced the corridor delay. However, the network delay was worsened significantly. This is because ramp metering is sacrificing the travel time of on-ramp vehicles to reduce the delay of mainline.

Chapter 6. Impact of ATM Strategies on Traffic Safety

6.1 Surrogate Safety Assessment Model (SSAM)

Safety is of paramount importance in operation of transportation systems. Typically, safety analysis of highway systems is conducted by analyzing vehicle crash reports generated by the police over a long period of time. The infrequent and unpredictable nature of highway crashes poses a challenge in collecting sufficient crash data to perform statistically significant safety analysis. Moreover, this approach cannot be used to evaluate new active traffic management strategies and roadway designs that are yet to be implemented in the field.

Under the guidance of the Federal Highway Administration (FHWA), Siemens Corporation developed the Surrogate Safety Assessment Model (SSAM), which combines microsimulation of traffic and automated "conflict analysis". SSAM performs "conflict analysis" by using the space-time vehicle trajectories produced by microscopic simulation models implementing various traffic-control strategies. A vehicle conflict (see Figure 6.1) is defined as a scenario that will result in a collision if no evasive action were taken by the two vehicles involved (SSAM, 2010).

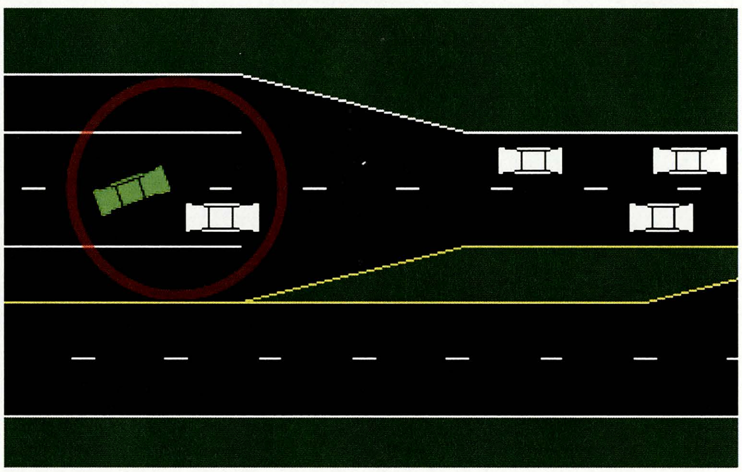


Figure 6.1: *Conflict scenario resulting from a lane change maneuver (SSAM, 2010)*

SSAM was used to perform safety analysis for the active traffic management (ATM) scenarios implemented in this study. SSAM calculates several surrogate safety measures using the vehicle trajectory files obtained from the microsimulation model to evaluate safety, and determines the types and frequency of the conflicts. SSAM also provides statistical comparisons of conflict frequencies and surrogate safety measures between two scenarios using the T-test. It can graphically display the locations of conflicts on the network by conflict types, and aids visual analyses of safety results.

6.2 Safety Analysis of ATM Scenarios

Various ATM scenarios were simulated in VISSIM microsimuation model, and the corresponding vehicle trajectories were exported for further safety analysis in the SSAM. A vehicle trajectory file contains a complete set of space-time trajectories of all the vehicles for the entire simulation duration in VISSIM. Ten sets of vehicle trajectory files, each corresponding to a different random seed number for VISSIM simulation, were imported into the SSAM for all the ATM scenarios to perform statistical analysis of safety measures.

6.2.1 Surrogate Safety Measures

Following surrogate safety measures obtained from the SSAM were used to evaluate safety:

TTC: the minimum time-to-collision value observed during the conflict.
PET: the minimum post encroachment time observed during the conflict. Post encroachment time is the time between when the first vehicle last occupied a position and the second vehicle subsequently arrived at the same position. A value of 0 indicates an actual collision.
MaxS: the maximum speed of either vehicle throughout the conflict
DeltaS: the difference in vehicle speeds as observed at the simulation time where the minimum TTC value for this conflict was observed.
MaxDeltaV: the maximum change between conflict velocity and the post-collision velocity of either vehicle in the conflict. This is a surrogate for the severity of the conflict, calculated assuming a hypothetical collision of the two vehicles in the conflict.

Individual analysis for each scenario and T-test between the ATM scenarios and the base case were performed. The results of the individual analysis using the SSAM surrogate safety measures are shown in Table 6.1.

Table 6.1: Conflict magnitude summary

Surrogate safety measures	Base case		VSL		Shoulder use		VSL & shoulder use	
	Mean	Variance	Mean	Variance	Mean	Variance	Mean	Variance
TTC	1.13	0.16	1.13	0.15	1.1	0.19	1.1	0.19
PET	2.22	0.97	2.25	0.91	2.15	1.08	2.17	1.05
MaxS	5.67	12.17	5.38	8.4	5.84	13.62	5.56	8.87
DeltaS	2.28	8.16	2.06	5.65	2.37	9.38	2.16	6.18
MaxDeltaV	1.24	2.7	1.12	1.81	1.29	3.04	1.18	1.99

Surrogate safety measures	Ramp metering	
	Mean	Variance
TTC	1.13	0.15
PET	2.25	0.91
MaxS	5.57	11.80
DeltaS	2.14	6.93
MaxDeltaV	1.16	2.25

Surrogate safety measures differences between the ATM scenarios and the base case were tested using T-test at 95% confidence level (see Tables 6.2 – 6.5).

Table 6.2: SSAM Safety Measures – base case vs. VSL

Surrogate safety measures	t-critical	Base vs. VSL		
		t value	Significant	Mean Difference
TTC	3.39	5.31	YES	0.005
PET	3.39	13.10	YES	0.032
MaxS	3.39	-35.81	YES	-0.287
DeltaS	3.39	-33.63	YES	-0.221
MaxDeltaV	3.39	-34.48	YES	-0.129

Table 6.3: SSAM Safety Measures – base case vs. shoulder use

Surrogate safety measures	t-critical	Base vs. Shoulder use		
		t value	Significant	Mean Difference
TTC	3.39	-27.29	YES	-0.028
PET	3.39	-28.71	YES	-0.073
MaxS	3.39	18.40	YES	0.165
DeltaS	3.39	11.89	YES	0.088
MaxDeltaV	3.39	11.68	YES	0.049

Table 6.4: SSAM Safety Measures – base case vs. VSL & shoulder use

Surrogate safety measures	t-critical	Base vs. VSL & Shoulder use		
		t value	Significant	Mean Difference
TTC	3.39	-24.41	YES	-0.026
PET	3.39	-20.03	YES	-0.051
MaxS	3.39	-13.99	YES	-0.114
DeltaS	3.39	-17.59	YES	-0.118
MaxDeltaV	3.39	-17.03	YES	-0.065

Table 6.5: SSAM Safety Measures – base case vs. ramp metering

Surrogate safety measures	t-critical	Base vs. Ramp metering		
		t value	Significant	Mean Difference
TTC	3.39	6.302	YES	0.007
PET	3.39	9.837	YES	0.026
MaxS	3.39	-10.48	YES	-0.099
DeltaS	3.39	-18.92	YES	-0.141
MaxDeltaV	3.39	-19.15	YES	-0.082

Table 6.2 illustrates that all the SSAM safety measures were significantly different between an ATM scenario and the base case. Variable speed limits (VSL) reduced the likelihood of conflict by increasing the time-to-collision (TTC) and post encroachment time (PET) between the vehicles. VSL also decreased the severity of conflict by reducing the maximum speed of vehicle (MaxS), speed differential between the two vehicles involved in a conflict (DeltaS), and the maximum change in speed pre- and post-collision (MaxDeltaV). Overall, VSL created a safer driving condition.

The effect of peak-period shoulder use on safety measures was opposite of that of VSL (Table 6.3). Shoulder use increased the likelihood of collision by decreasing the time-to-collision (TTC) and post encroachment time (PET) between the vehicles. It also increased maximum speed (MaxS) and speed differential measures (DeltaS, MaxDeltaV), thus increasing the severity of conflicts. This indicates that the shoulder use strategy should be applied carefully, and with safety considerations in mind.

When both VSL and peak-period shoulder use were implemented simultaneously, maximum speed (MaxS) and speed differential measures (DeltaS, MaxDeltaV) decreased in comparison with the base case (Table 6.4). Thus, simultaneous use of VSL and shoulder use had an overall positive effect on speed harmonization for the traffic, and it reduced the severity of the conflicts. However this scenario decreased the time-to-collision (TTC) and post encroachment time (PET) between the vehicles, thereby increasing the likelihood of conflicts. This is due to the fact that the benefits of VSL were partly offset by the opposite influence of shoulder use. The results obtained from the safety analysis are in accordance with those obtained from the traffic operations analysis presented in Chapter 5. Both the results call for greater safety and operational considerations in the case of shoulder use, especially towards the end of the shoulder-use segment. They also indicate that part of the benefits of speed harmonization due to VSL may be offset by the opposite effect of shoulder use.

Ramp metering had a similar effect on safety as VSL, which reduced the likelihood of conflict, and decreased the severity of conflicts (Table 6.5).

6.2.2 Types and Frequency of Conflicts

Besides the SSAM safety measures, we also compared the total number of conflicts and different types of conflicts between the base case and the ATM scenarios. Different conflict types in the SSAM are defined using the conflict angle calculated for each pair of conflicting vehicles, which is based on the angle at which these vehicles converge to a hypothetical collision point. The conflict angle is used for classification as follows:

- Rear-end: || conflict angle || < 30 °.
- Lane-change: 30 ° ≤ || conflict angle || ≤ 85 °.
- Crossing: || conflict angle || > 85 °.

The results for different types of conflicts are shown in Figure 6.2 below.

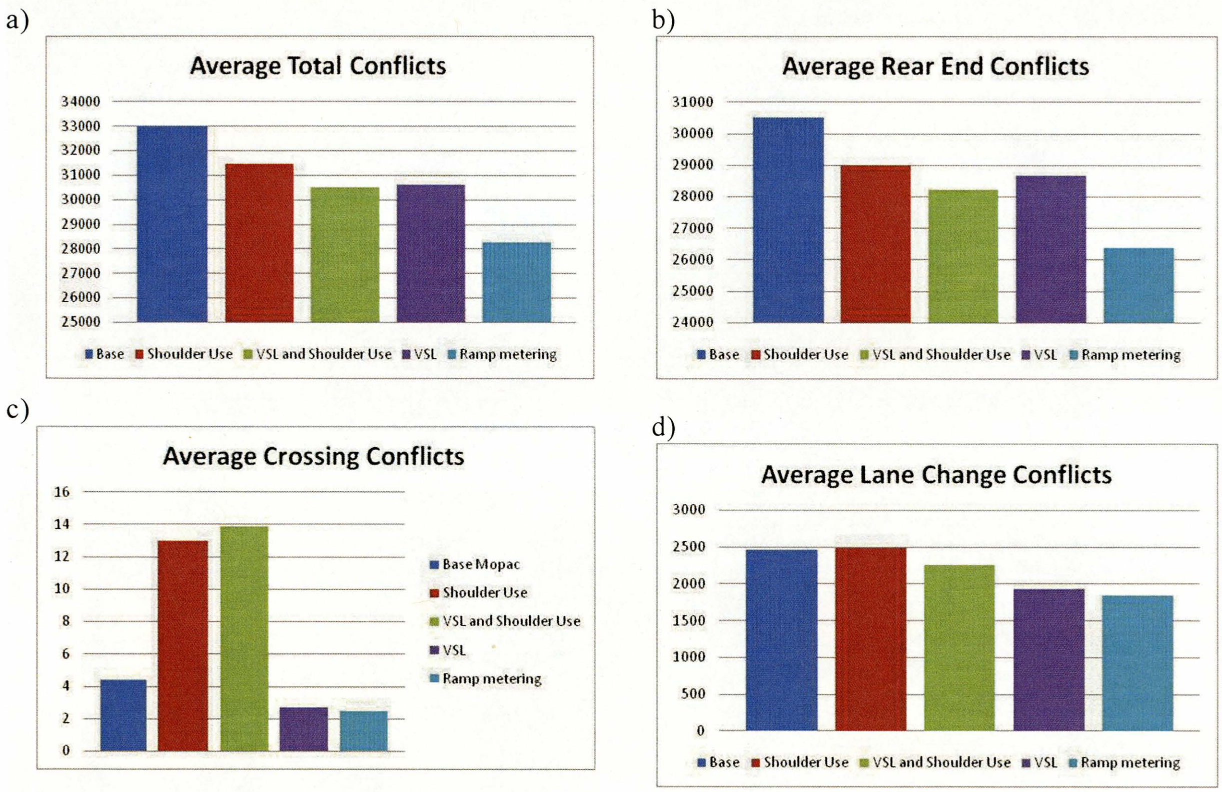


Figure 6.2: *Overall conflict comparison for each scenario*

Figure 6.2 displays that all the ATM strategies decreased the number of total conflicts, rear-end and lane-change conflicts (Figures 6.2a, 6.2b, and 6.2c). Both the strategies including shoulder use increased crossing conflicts (Figure 6.2d). This result is similar to the safety analyses in the previous section, which call for caution while implementing peak-period shoulder use.

Similarly, we performed T-test to see if the differences between the number of conflicts for the base case and the ATM scenarios were significant. The results are shown in Table 6.6. The results show that VSL significantly decreased lane-changing conflicts when compared with the base case scenario. This proves that under VSL vehicles have more harmonious speed and fewer drivers have incentive to change their lane to pass other vehicles. Thus VSL created a safer driving condition. In both the scenarios involving shoulder use, crossing conflicts increased significantly. When both VSL and shoulder use strategies were implemented simultaneously, the benefit of VSL decreasing lane-changing conflicts is offset by shoulder use which increases lane-changing conflicts. Again, ramp metering had a similar effect on safety as VSL, which reduced lane-changing conflicts significantly.

Table 6.6: T-test results for conflict numbers

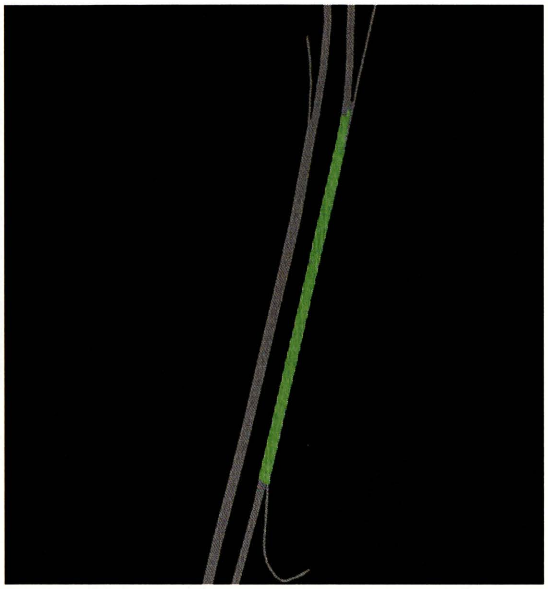
Conflict Types	t-critical	Base vs. VSL			Base vs. Shoulder use			Base vs VSL & Shoulder use		
		t value	Signi-ficant	Mean Difference	t value	Signi-ficant	Mean Difference	t value	Signi-ficant	Mean Difference
Crossing	3.922	-2.146	NO	-1.7	5.944	YES	8.6	5.175	YES	9.5
Rear-end	3.922	-1.107	NO	-1820.2	-0.914	NO	-1,515.3	-1.485	NO	-2,276.2
Lane Changing	3.922	-6.345	YES	-523.9	0.378	NO	25	-3.079	NO	-201.6
Total	3.922	-1.367	NO	-2345.8	-0.863	NO	-1,481.7	-1.55	NO	-2,468.30

Conflict Types	t-critical	Base vs. Ramp metering		
		t value	Significant	Mean Difference
Crossing	3.922	-2.097	NO	-1.9
Rear-end	3.922	-2.988	NO	-4107.074
Lane Changing	3.922	-7.643	YES	-610.675
Total	3.922	-3.273	NO	-4719.648

6.2.1 Visual Analysis of Conflicts

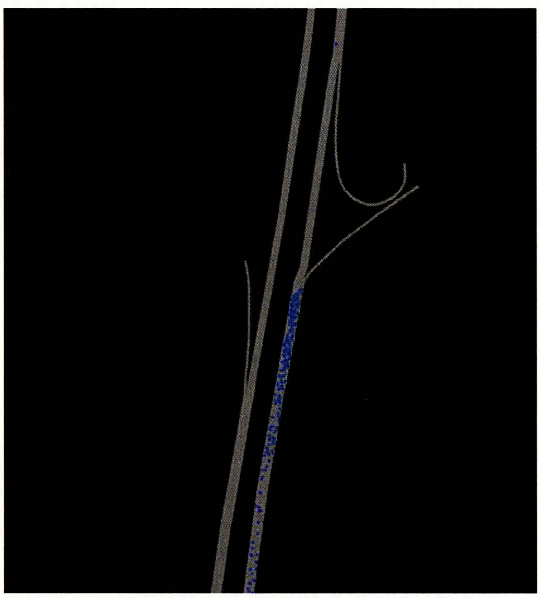
SSAM can visually display the safety implications of various traffic control strategies on the network. Figure 6.3 provides two such examples comparing locations and frequency of conflicts between ATM scenarios.

Visual analysis provides a more focused safety analysis at specific locations in the network, and highlights the safety impacts that may not be easily discernible from network-wide average safety metrics. For example, T-test results in Table 6.4 show that VSL & Shoulder use did not significantly decrease rear-end conflicts for the entire network. However the speed harmonizing effect of VSL & Shoulder is quite significant in the middle of the VSL section as seen in Figures 6.3a and 6.3b. This combined ATM strategy resulted in a significant decrease in rear-end conflicts. Figures 6.3c and 6.3d present a comparison between lane changing conflicts toward the end of the shoulder-use segment under VSL and shoulder use scenario. We can see that shoulder use increased lane-change conflicts between the off-ramp and on-ramp, and spread this conflict to a longer stretch of the freeway. This again, illustrates possible safety issue caused by peak-period shoulder use.

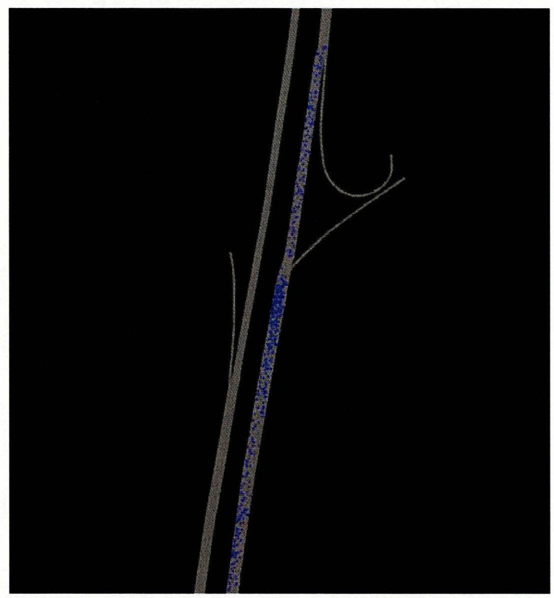


(a) Rear-end Conflicts (Base case, middle of VSL section)

(b) Rear-end Conflicts (VSL & Shoulder Use Scenario, middle of VSL section)



(c) Lane-change conflicts (VSL, end of shoulder use section)



(d) Lane-change conflicts (Shoulder Use Scenario, end of shoulder use section)

Figure 6.3: *Conflict comparisons for ATM scenarios*

Visual analysis in the SSAM is an important tool for performing focused safety analysis, and complements the numerical analysis to help draw credible conclusions. It also encouraged a more focused numerical analysis to ascertain the effect of various ATM strategies in different parts of the testbed section. Effects of the ATM strategies on different types of conflicts were studied at three locations: beginning of the VSL section, middle of the VSL section, and end of the shoulder-use section. These results are presented in Tables 6.7 – 6.9. T-test for difference were conducted using ten simulation runs, and the letter Y/N in parenthesis denotes whether the difference is significant at 95% confidence level or not (Y – difference is significant; N – difference is not significant).

Table 6.7: Conflict frequency by type (beginning of the VSL section)

Conflict Types	Base case	VSL	Shoulder Use	VSL & Shoulder use
Crossing	0	0 (N)	0 (N)	0 (N)
Rear-end	579.1	853.8 (N)	0.4 (Y)	0 (Y)
Lane changing	59.8	85.3 (N)	5.5 (Y)	1 (Y)
Total	638.9	939.1 (N)	5.9 (Y)	1 (Y)

At the beginning of the VSL section, VSL did not have significant impact on any conflict type. Shoulder use and VSL & Shoulder use reduced all conflict types significantly.

Table 6.8: Conflict frequency by type (middle of the VSL section)

Conflict Types	Base case	VSL	Shoulder Use	VSL & Shoulder use
Crossing	1.5	1.1 (N)	0.5 (Y)	0.7 (N)
Rear-end	8,869.00	9273.2 (N)	964 (Y)	1344.7 (Y)
Lane changing	454.9	353.2 (Y)	138.7 (Y)	127.8 (Y)
Total	9,325.40	9627.5 (N)	1103.2 (Y)	1473.2 (Y)

In the middle of the VSL section, ATM strategies involving shoulder use reduced all conflict types significantly. VSL significantly reduced lane-changing conflicts leading to a harmonized traffic flow.

Table 6.9: Conflict frequency by type (end of the shoulder-use section)

Conflict Types	Base case	VSL	Shoulder Use	VSL & Shoulder use
Crossing	0.5	0.6 (N)	2 (Y)	1 (N)
Rear-end	6,959.80	6692.5 (Y)	6230.3 (N)	6708.9 (N)
Lane changing	530.2	406.4 (Y)	754.3 (Y)	694.5 (Y)
Total	7,490.50	7099.5 (Y)	6986.6 (N)	7404.4 (N)

At the end of the shoulder-use section, lane-changing conflicts significantly increased due one lane-drop on the freeway. It also leads to a small increase in crossing conflicts. These results reiterate the safety issues raised by the use peak-period shoulder use. The effect of VSL on conflicts was more than positive at the end of the shoulder-use section compared to the middle of the VSL section.

6.3 Results and Conclusions

This chapter investigated the impact of variable speed limits (VSL), peak-period shoulder use, and ramp metering on freeway traffic safety in a testbed of the Missouri-Pacific (Mopac) Expressway in Austin, Texas. A microscopic simulation model was developed in VISSIM to implement the various active traffic management (ATM) strategies. Safety analysis of the ATM strategies was performed using the Surrogate Safety Assessment Model (SSAM), which uses the vehicle trajectories exported from the microsimulation model.

VSL harmonized traffic flow, reduced lane-changing conflicts, and created safer driving condition. VSL reduced both the likelihood and severity of conflicts. On the other hand, shoulder-use increased the likelihood and severity of conflicts, and leads to increase in crossing conflicts. Shoulder use also decreased speed at the end of the shoulder-use segment due to bottleneck creation as a result of one-lane drop. However, shoulder use improved traffic conditions in the middle of the shoulder-use segment by reducing traffic density and increasing operating speed.

Simultaneous implementation of VSL and shoulder use consistently harmonized the traffic flow and improved travel condition by reducing speed variability, traffic density, and stop-n-go condition (by reducing the number of stops per vehicle). It did not have significant impact on throughput, as is reported in previous studies on VSL. It had an overall positive effect on speed harmonization for the traffic, reduced the severity of conflicts, but increased the likelihood of conflicts. It was found that VSL and shoulder use decreased delay per vehicle, and overall these strategies created safer driving condition by reducing speed variability. Shoulder use implementation results point to a need for comprehensive safety evaluation before real-life implementation, especially towards the end of the shoulder-use segment.

Chapter 7. Recommendations on ITS, Enforcement, and Potential Impediments

In this chapter we collect a number of recommendations regarding ITS (Section 7.1), enforcement (Section 7.2) and present a discussion on potential impediments (Section 7.3) regarding the actual implementation.

7.1 Recommendations on ITS

This chapter summarizes activities conducted to survey the existing ITS technology implemented in Europe and the US, and provides some recommendations on ITS devices. ITS devices are used for traffic monitoring, information dissemination, and enforcement. We have surveyed the existing ITS technology implemented in speed control and temporary shoulder use throughout Europe and the US. The results are summarized in this section.

7.1.1 Traffic Monitoring

United Kingdom, M25

In the U.K., Automatic Number Plate Recognition (ANPR) cameras have been used on M25 between junctions 28 and 27. The ANPR cameras are used for gathering flow and journey time information. The set-up of the system is as follows: two or more cameras are set up at each end of a road segment under consideration. At each station the plate recognizer unit (running Talon ANPR software) and camera collect plate reads from passing traffic. This information is then sent to a traffic management center for processing.

Netherlands, A2 Motorway

In the Netherlands, loop detectors were located every 500 m (0.3 mile) over a 12.4-mile stretch of the A2 Motorway. The posted speed is determined by a system control algorithm based on one-minute averages of speed and volume across all lanes.

Washington IH 90

The Washington State Department of Transportation (WSDOT) is operating a VSL system on IH 90 across Snoqualmie Pass. The VSL systems are Intelligent Transportation Systems that utilize traffic speed and volume detection, weather information, and road surface condition technology to determine appropriate speeds for drivers. Information for setting speed limits and for the message signs is gathered from a variety of sources. Wide aperture radar tracks speeds for feedback to the control system. All roadside data collection and control is processed through roadside cabinets.

Germany, Bavaria A7

The Line Control System operates at A7, which is a north-south axis extending from the Danish border through Germany and ending near Füssen. For traffic measurements, traffic volume and vehicle speed are detected at every observation point by inductive loops or radar sensors along the 15 km (9.3 miles) between Nesselwang and the tunnel Füssen (border with Austria).

7.1.2 Recommendation Regarding Traffic Surveillance Equipment

Based on the previous review, a monitoring surveillance system is recommended for the use of speed harmonization and peak-period shoulder usage to manage urban freeway congestion. For monitoring purposes, camera detectors are placed at an average of 1 mile, at most, for incident detection purpose. Loop detectors were used along with closed circuit camera detection in case of bad weather like fog, heavy rain, etc.

Based on former experience, if the peak-period shoulder is used at a certain segment of highway, the camera should be installed at a distance of at most 1 mile to encompass traffic detection. To help the operator, fixed movement sequences are programmed into the pivoting cameras:

- During the scanning operation, the cameras are controlled so that even small objects can be detected.
- During the release process, the Traffic Center regular carries out video monitoring of the hard shoulders.
- Unlike with the scanning operation, the cameras use the "waggle program" to show large sections of the released hard shoulder for a few seconds.
- If the operator detects a broken down vehicle during the release process, the release is cancelled for the period of time concerned.
- Shoulder won't open during bad weather, such as fog, heavy rain, etc.

7.1.3 Information Dissemination

In most cases, speed information is displayed using dynamic message signs, which can be switched off or used for other display purposes. In the United States, the variable speed limit information has been typically displayed on the roadside using VMS or portable message signs.

United Kingdom, M25

On the M25 the Journey Time Management System (JTMS) is built with Talon Journey Time Analysis (JTA) and Average Speed (AS) software, which is used with the Talon ANPR engine to provide real time travel and speed information to vehicles travelling along open roads and highways and through urban and city center areas. As a vehicle is matched at the end of a road segment or outstation, the individual journey time of that vehicle is calculated.

Netherlands, A2 Motorway

The standard posted speed limit is 120 km/h (75 mph), and the variable posted speeds are 50, 70, and 90 km/h (31, 43, and 56 mph). The posted speed is determined by a system control algorithm based on one-minute averages of speed and volume across all lanes. The system covers 20 km (12 miles) with VSL signs spaced approximately every one km (0.6 mile). If an incident is detected, a speed of 50 km/h (31 mph) is displayed. If the speeds are posted with a red circle, they are enforced by photo radar. If posted without the circle, they are advisory.

Germany, Bavaria A7

The main Line Control Systems (LCS) in Germany consists of a successive installation of OP (Observation Points) and VMS (variable message signs) panels. The system includes altogether 14 display gantries, 4 variable direction signs, and 26 measurement sites outside the tunnels. Special attention must be given to an integrated technical concept between the tunnel systems and the LCS. If there is an emergency, the switching instructions will be generated by the OCT (operations control technology) and will be transmitted via the sub-center to the control units of the LCS for execution.

Washington IH 90

All roadside data collection and control is processed through roadside cabinets. Communications from the mountaintops to the control center are transmitted by microwave. All of this collected information goes to a central computer, which processes the data and determines the "safe speed" for the roadway. This system is monitored from a DOT maintenance office at the pass. Currently, a computer recommends the speed limit and an operator confirmation implements it. The speeds can vary along the corridor, and speed postings for one direction of travel may differ from those for the other direction. Inherent in the system's design is the capability for expansion, and there has been some planning to lengthen the VSL to cover more of IH 90. VSL is also being planned for portions of US-2, Stevens Pass, which also crosses the Cascades north of IH 90.

New Mexico IH 40

The project on IH 40 used variable speed signs and hazard warning signs on the right side of the road. The speed limit varied with changing conditions. VSL typically covers longer stretches of roadway, and incorporates a broad range of input criteria for speed limit decision (traffic speed, volume, crashes, congestion, construction, ice, snow, fog, etc.), which is restrained by the NMSL (National Maximum Speed Limit).

The system used a look-up table to generate the posted speed limit (see Box 1). The limit was based on the smoothed average speed plus a constant based on the environmental conditions. Negative constants were used to keep the posted speed below the 89 km/h (55mph) maximum speed limit cap.

Box 1: New Mexico's Automated Speed Control Logic

Smooth Mean Speed +/-

	Max (mi/h)	Min (mi/h)
Dry Day	+ 6.5	- 6.0
Dry Dark	+ 5.0	- 5.0
Wet Day	+ 2.5	na
Wet Dark	+ 0	na

Table 7.1 summarizes the ITS configurations detailed in this section.

Table 7.1: ITS configurations

Location	Analyze Information	Output Information
United Kingdom	Journey Time Management System (JTMS) built with Talon Journey Time Analysis (JTA) and Average Speed (AS) software	roadside portable Variable Message Signs (VMS)
Netherlands	The posted speed is determined by a system control algorithm based on one-minute averages of speed and volume across all lanes.	VSL signs spaced approximately every one km
Germany	The control and the switching of all traffic relevant systems normally take place out of the sub center LCS A7. If there is an emergency, the switching instructions will be generated by the OCT and will be transmitted via the sub center to the control units of the LCS for execution.	The system includes altogether 14 display gantries, four variable direction signs and 26 measurement sites outside the tunnels.
Washington	All of collected information goes to a central computer, which processes the data and determines the "safe speed" for the roadway. Currently, a computer recommends the speed limit and an operator confirmation implements it.	VSL (Variable Speed Limit) signs including hazard warning
New Mexico	After obtaining inputs (which have a broad range of criteria), the system used a look-up table to generate the posted speed limit.	VSL (Variable Speed Limit) signs and hazard warning sign

7.1.4 Recommendations Regarding Information Dissemination Devices

The combination of VMS and Portable Changeable Message Sign (PCMS) systems is recommended for information dissemination. The sign is recommended to be displayed on overhead gantries, located each mile.

7.1.5 Enforcement: Photo-radars to Detect Speed-limit Violators

Normally speeding is controlled by the police; however, when the temporary shoulder opens, there will be no space for police officers to enforce the speeding. Photo-radar systems will provide great assistance in the use of speed harmonization and peak-period shoulder usage in managing urban freeway congestion. Table 7.2 summarizes the use of photo-radars in the Netherlands and UK.

Table 7.2: Use of photo-radars in the Netherlands and UK

Location	Tolerance/Enforcement
The Netherlands	In the Netherlands drivers can get a fine for driving 4 km/h (2.5 mph) over the speed limit, after applying a 3 or 4 km/h (1.9 or 2.5 mph) correction factor to compensate for measuring errors. Police officers are usually not allowed to use their discretion when setting the speeding threshold during enforcement activities by photo radar.
United Kingdom	In the United Kingdom, Association of Chief Police Officers (ACPO) guidelines recommend a tolerance level of the speed limit "+10% +2 mph" (e.g., a tolerance level in a 30 mph (50 km/h) zone of 35 mph). However, each police force or safety camera partnership has the ability to use its discretion when setting the levels at which drivers will be prosecuted. Photo radar uses 35mm photos.

7.1.6 Recommendation

Enforcement devices are also recommend to be placed on overhead gantries used to display VMS. These will allow violators to be automatically notified; otherwise the system will record the license plates of vehicles.

7.2 Recommendations on Enforcement

Enforcement is considered a critical piece of a successful variable speed limit program and the effective use of the shoulders as travel lanes. The primary purposes are clear: 1) to ensure compliance with the posted speed; and 2) to ensure vehicles are using the shoulders as travel lanes only when authorized. Variable speed limits appear to present a larger challenge in implementing and enforcing compared to peak period shoulder use. There are two primary ingredients for effective enforcement of variable speed limits. The first is having state law that supports variable speed limits as a regulatory speed limit for which citations can be written, if violated (as opposed to an advisory speed). The second ingredient is consistently enforcing the variable speed limit posted.

This section summarizes why enforcement is necessary for a successful variable speed limit program and peak period shoulder use. It also discusses the two key legal considerations for effectively implementing variable speed limits and liability concerns regarding variable speed limits. Recommendations based on the material reviewed to-date are provided. Finally, a set of survey questions is presented; these survey questions were distributed to jurisdictions in the United States with experience in variable speed limits and/or peak period shoulder use. Key information from the survey responses is incorporated into the recommendations related to enforcement and noted following the survey questions.

7.2.1 Why Enforcement is Necessary

Traditionally, traffic law enforcement relies heavily on deterring traffic violations through fines, which can lead to varying degrees of penalties against an individual's license (e.g., suspension) and/or criminal record (e.g., driving under the influence). An individual's choice to exceed the speed limit and exceed the limit by a certain amount is often made based on the perceived risk of being caught and ticketed. In the absence of consistent enforcement, motorists travel at speeds consistently higher than the speed limit. A study initiated in 2001 and conducted in Washington, D.C. and Baltimore, Maryland illustrated this behavior.

Washington, D.C. implemented automated enforcement for speeding on several surface streets. The automated speed enforcement primarily consisted of cameras triggered to take a photograph when the associated Doppler radar speed sensor indicated a vehicle was traveling faster than a preset speed (Retting and Farmer, 2003). A set of comparable sites in Baltimore, Maryland was left untreated. The study found that in Washington, D.C. the mean speed dropped 14% and vehicles exceeding the speed limit by more than 10 mph dropped 82%. The sites in Baltimore, Maryland experienced no significant change in mean speed or the percent of vehicles exceeding 10 mph (Retting and Farmer, 2003). Clearly, enforcement makes a difference in driver behavior.

The ability to ensure motorists are traveling at the posted speeds when variable speed limits are being used to increase capacity (i.e., manage congestion) is paramount to its success. A primary purpose of using the variable speeds during congested periods is to create a traffic flow with as little turbulence or friction between vehicles as possible; the ideal is to have all vehicles traveling at a constant and consistent speed maximizing capacity for the facility under the given traffic and weather conditions. Variable speed limit installations in the Netherlands and Finland found camera speed enforcement critical to lowering speeds to the desired level under congested as well as adverse weather conditions (Sisiopiku, 2001).

Enforcing the appropriate use of peak period shoulder lanes tends to be more critical when the shoulders are not open or authorized for use. Safety concerns and conflicts could arise from motorists using the shoulders as lanes when travel speeds on the mainline are too high to warrant their use. Conflicts could also occur if vehicles are using the shoulder as a lane when the appropriate controls have not been set at the on- and off-ramps. During peak period shoulder use operations, motorists must also follow the signs and traffic control devices indicating a shoulder is no longer a useable travel lane. Failure to obey these devices when approaching a narrow bridge deck or other similar physical obstruction could easily create conflicts and disruptions in the traffic flow as vehicles try to move abruptly back into a mainline lane.

The need and value of enforcement is clear for both the variable speed limit and peak period shoulder use treatments.

7.2.2 Legal Framework Necessary for Posting Variable Speed Limits

In the context of this project, the purpose of posting variable speed limits is to reduce congestion and/or delay the onset of highly congested periods on freeways. Therefore, the speed limits posted need to be enforceable; they should be able to hold up in the court of law. In general, state law allows state or local officials to decrease speed limits if they determine the absolute speed under existing law is greater than reasonable or safe (Steel et al., 2005). As a result, state or local officials have the ability to set revised regulatory posted speed limits applicable to all times or varying conditions. This general allowance provides for interpreting variable speed limits as regulatory speed limits; however, Hines and McDaniel (2002) recommend several elements of variable speed limit legislature be in place prior to implementing variable speed limits on the roads. The purpose of these elements is to make enforcement possible and is to ensure variable speed limits will survive challenges of constitutionality that could arise. The key elements outlined by Hines and McDaniel (2002) are summarized here.

1. The statutory purpose should allow a change in speed limit to protect public safety and permit the legislature to delegate to an agency power to prescribe details after they have fixed a primary policy or standard.

2. The law should require the change in the speed limit to be based on engineering and traffic investigations; in the context of variable speed limits, these would show the need for and benefit of variable speed limits under certain situations.

3. The statute must require posting for the new limit to be effective.

4. The statue must require posting of advance warning that the legal speed limit is changed ahead.

5. The law must require any information or charging documents include the existing speed limit and speed at which it is alleged the charged driver's vehicle was traveling.

6. The law might prohibit automatic enforcement within a certain distance of the new limit to allow reasonable time for driver's to adjust their speeds.

7. The law should provide broad discretion to administrative agency for enactment of regulations and sub-delegation of decision-making power.

8. Either laws or regulations should provide for certain evidence by affidavit. This means where the speed limit is decreased due to temporary hazards (e.g., traffic, weather) evidence of the reasons and the specific speed limit on the highway where the violation allegedly occurred must be presented.

The legal elements listed should be no different than the legal issues considered by courts where violations of fixed maximum speed limits occur (Hines and McDaniel, 2002).

7.2.3 Enforcing Variable Speed Limits

The two basic types of enforcement are manual enforcement and automated enforcement. Manual enforcement via police presence is generally costly and therefore sporadic; it is not a particularly effective means to consistently enforce traffic laws (Retting and Farmer, 2003). There are also logistical issues during highly congested periods or adverse weather, which makes manual police enforcement challenging and sometimes risky. For these and related reasons, automated enforcement is becoming increasingly popular among jurisdictions. Automated enforcement in Europe and Australia tends to be ahead of applications in the United States. The United States is faced with a different legal context, which can make automated enforcement difficult to uphold when challenged in court.

The first breakthrough in automated enforcement application in the United States has been focused around the use of red-light running cameras at signalized intersections. The state of

Texas is one of the states in which red-light running cameras are in use at intersections with a previously high occurrence of red-light running incidents and/or high-speed angle crashes (which were the result of a motorist running a red-light). Many of the concerns related to automated enforcement for red-light running cameras apply to automated enforcement in general. Therefore, many of the legal concerns related to using cameras to enforce variable speed limits and/or the appropriate use of the shoulder have already been partially addressed.

Manual Enforcement

Manual enforcement is the traditional approach to enforcing traffic laws. It can take form in a variety of ways with the simplest setup being one officer with radar of some sort to detect the speed of vehicles on the facility. This includes stationary marked or unmarked vehicles by the side of the road with radar or vascar technology or a stationary officer with detectors located across the road enabling the officer to be farther from the road. Other variations are a stationary police car plus a chase car (one officer operates the radar and the other chases down the violator), moving police vehicle using moving radar, and air patrol with an air observer and police chase cars on the ground. These strategies can be challenging to operate in periods approaching high congestion and/or adverse weather conditions. Both of these situations make it difficult for officers to chase down the vehicles and find a location to the pull them over safely. In periods approaching high congestion, a traffic stop can create the traffic flow disturbance and shockwaves that the variable speed limit program is trying to eliminate, thereby being counterproductive.

Automated Enforcement

Automated enforcement is becoming increasingly popular in the United States. Many of its uses are currently to prevent red-light running; however, it has been used to enforce speed limits, as demonstrated by the Washington, D.C. study. European countries and Australia tend to have the most experience with automated speed enforcement (Retting and Farmer, 2003). Automated enforcement of speeds is considered common practice in European countries particularly when using variable speed limits (Sisiopiku, 2001). Cameras are mounted on the back of overhead freeway signs above each travel lane and photograph vehicles identified by speed detectors as speeding. Speed data is collected via a variety of methods including loop detectors, overhead radar, and closed circuit television (Sisiopiku, 2001). The same type of technology is available in the United States; however, the legal atmosphere poses additional challenges to implementation.

Kraus and Quinoga (2003) identified four critical legislative issues with regards to automated enforcement. Their paper focuses on legislative issues as related to red-light running cameras, but the same principles apply to automated enforcement of traffic laws, in general. The four critical issues identified are as follow:

1. Criminal vs. civil offense;
2. Liability of the owner or vehicle operator;
3. Privacy rights; and
4. Fine and revenue structure.

The issues identified are consistent with those identified in the Rodier et al. (2007) paper regarding automated speed enforcement in the United States; however, in this paper the issue related to privacy is connected to a broader discussion of constitutionality. Criminal verses civil offense, liability of the owner or vehicle operator, constitutionality, and fine and revenue structure are discussed in this section and related to Texas State Law. Also included is a summary of potential legislation that would enable automated speed enforcement; essentially, help uphold it in a court of law.

Criminal vs. Civil Offense

Criminal and civil offenses are associated with different burdens of proof in a court of law. In a criminal offense, the burden of proof lies with the prosecution; they must prove the defendant guilty beyond a reasonable doubt. In a civil offense, the burden of proof is by preponderance of evidence; if the evidence suggests the defendant is more likely guilty than not the court may find the accused guilty unless the accused can prove innocence. This distinction makes it easier, quicker, and cheaper to try civil offenses, which carry less of a fine/penalty for those found guilty.

The distinction applies to traffic laws because moving traffic violations (e.g., speeding) is considered a criminal offense in many states. Therefore, any enforcement measures for these offenses must positively identify the vehicle and driver (Kraus and Quiroga, 2003). Using manual enforcement in which a sworn officer observes the offense this is not usually an issue. When using automated enforcement, it is now necessary to take a photo of the vehicle license plate and driver; often requiring a set-up in which the front and rear of the vehicle can be photographed. The photographic technology also becomes critical because the quality of the photo must make it possible to definitively identify the driver.

Liability

The liability issue revolves around the ability to identify the driver of the vehicle at the time of the violation. If the driver is identified by an officer or a definitive photograph, then the driver is cited for the violation regardless of car ownership. If the driver at the time of the violation is not identified then a court is able to dismiss the citation in states categorizing moving traffic violations as criminal offenses (Kraus and Quiroga, 2003). This greatly impacts the citation rate for automated enforcement. In Chandler, AZ where red-light running is considered a civil offense and automated enforcement is used, the citation rate is 83% compared to San Francisco where red-light running is a criminal offense and the citation rate is 25% (Kraus and Quiroga, 2003). Automated enforcement is more effective at dispensing citations under a civil offense interpretation of traffic law violations.

Constitutionality

The primary concerns related to violating the constitution are violating the right to privacy and freedom under the First Amendment; protection against illegal search and seizures under the Fourth Amendment; right to due process under the Fifth and Fourteenth Amendment; equal protection doctrine under the Fourteenth Amendment; and the taking clause under the Fifth Amendment (Rodier et al., 2007). However, the Supreme Court has clarified the Fourth Amendment depends on whether the person has a legitimate expectation of privacy in the invaded place and because driving is a public activity the Fourth Amendment does not apply to automated enforcement (Kraus and Quiroga, 2003). A legitimate concern does arise with the use of private information that is pulled as a consequence of the citation; therefore the photogenic evidence needs to remain confidential and limited to authorized personnel.

With regards to the remaining potential constitutional rights, legal scholars generally agree, based on the body of established law, that automated enforcement programs for traffic laws do not violate constitutional rights (Rodier et al., 2007).

Fine and Revenue Structure

Past abuses of automated enforcement by equipment vendors have led to the reputation of automated enforcement as simply a revenue generator. The payment arrangements with vendors and the jurisdictions use of the revenue are related to this reputation. Some jurisdictions set up a payment structure such that vendors were paid for the equipment a flat fee and then paid to operate the system. The payment they received to operate the system was based on the number of citations issued—similar to a commission for a sales person. In San Diego and Denver, this resulted in judges ruling that this payment structure presents a clear conflict of interest to the vendors and was a wrongful delegation of responsibility by the jurisdiction (Kraus and Quiroga, 2003).

Texas State Law

In Texas, a series of attempts to set up a legal structure to support automated enforcement began in 1995 with Senate Bill 1512 progressing to Senate Bill 454 in 2001 and Senate Bill 1184 in 2003. Senate Bill 1512 provided TxDOT with authorization to conduct a two-year study using automated enforcement at 10 highway railroad grade crossings (Kraus and Quiroga, 2003). Senate Bill 454 allows TxDOT and the Texas Turnpike Authority to use automated enforcement systems on toll facilities. The substantial piece of legislature creating the legal structure to use red-light running cameras was Senate Bill 1184 passed in 2003 (Texas House of Representatives House Resource Organization., 2006).

Senate Bill 1184 grants the cities within Texas the power to issue civil citations for violations previously punishable only by criminal offenses (Texas House of Representatives House Resource Organization., 2006). This bill opened the door for red-light running cameras and since then their use has been seen in cities across Texas including Austin. The bill was passed with the amendment granting local authorities the power to regulate roads using "criminal, civil, and administrative enforcement" (Transportation Code, sec 542.202(b) (3)). Based on the language in Senate Bill 1184, it seems possible and reasonable that this could be extended to using automated enforcement for speed limits (variable and static) as well as the use of appropriate use of the shoulder. Review of the law by a lawyer or legislative expert is needed to confirm this deduction.

In addition to state legislation enabling local jurisdictions to prosecute traffic law violations as civil offenses, additional specific elements of legislation are recommended to enable or facilitate the ability of automated enforcement to be upheld the courts. The specific combination of enabling elements of legislation should be determined by the courts, enforcement agencies, transportation departments, motor vehicle departments, and any other agencies whose operations would be affected by the automated speed enforcement program (Rodier et al., 2007). Current states in the United States with enabling legislation for automated speed enforcement are Arizona, Arkansas, Colorado, Illinois, Maryland, Utah, Washington D.C., and Oregon. Potential elements identified by Rodier et al. (2007) to be included in state legislation include these:

- Definition of acceptable automated enforcement devices;

- Any restrictive uses (e.g., man vs. unmanned);
- Description of acceptable photographic evidence;
- Description of admissibility of such evidence;
- A registered owner liability section including provisions for refutable presumptions;
- Description of any required corroborating testimony;
- Provisions for summons by mail;
- Penalty provisions; and
- Specific agency empowered to operate the system.

With the passage of Senate Bill 1184, Texas State Law appears to have the necessary basics to allow for automated enforcement for speed and peak period shoulder use. It would be prudent for TxDOT to have a more formal legal review of the law to confirm this is true. If TxDOT decides to pursue automated speed enforcement, additional detailed consideration to the form and legal language for the appropriate enabling legislation would need to be determined amongst the appropriate transportation, enforcement, and legal experts. The information provided here is a broad overview of issues to be aware of, as well as potential ways to overcome these issues.

7.2.4 Potential Liability Issues

Liability concerns have been raised primarily related to the use of variable speed limits. The basic concern is the transfer of responsibility for determining a safe speed from the driver to the jurisdiction operating the variable speed limit system. Historically, the driver is responsible for choosing a safe speed at which to travel that is at, or if conditions warrant, slower than the posted speed limit. Some believe drivers will become dependent on the variable speed limits and will assume when a speed limit sign displays a typical speed then there is no danger ahead, causing them to disengage mentally from the driving task (Sisiopiku, 2001). Variable speed limits in the United States have not been widely used enough or for a long enough duration to indicate whether or not this perception is true. A review of current literature did not bring to attention any specific cases with respect to liability. The variable speed limits set in this project would remain speed limits, therefore the responsibility of the driver to continue to choose a safe speed at, or if conditions warrant, below the speed limit would still apply.

7.2.5 Recommendations

Based on the material reviewed and information gathered via the survey responses we recommend confirming the elements outlined by Hines and McDaniel (2002) are present in the Texas State Law Transportation Code. While the legal issues related to enforcing variable speed limits should be no different than enforcing fixed speed limits, additional legislative support would be useful if a citation for violating a variable speed limit is challenged in court.

We also recommend using automated enforcement as the primary means for enforcing the variable speed limit and peak period shoulder use. To achieve this, an investment in technology infrastructure will be necessary as well as a more formal legal review of State Bill 1184 to confirm it is applicable to speed enforcement and the proper use of the highway shoulder. We also recommend a more formal legal review of the appropriate enabling legislation specific to automated speed enforcement. Manual enforcement could be used as an interim enforcement measure until the necessary physical and legal pieces are in place; however, manual enforcement as a long-term enforcement plan is likely to be more expensive and less effective than automated enforcement.

7.2.6 Survey Questions

Listed here are a series of survey questions. These survey questions were distributed to jurisdictions in the United States that have or are currently using variable speed limits and/or shoulders as lanes during peak periods.

- Are variable speed limits currently in use in your state? If so, please state the statutory or regulatory authority for those speed limits.
- How many locations are equipped with variable speed limits and/or peak period shoulder use? For what time duration have they been in-place?
- What, if any, changes were made to state laws to allow for or to facilitate using variable speed limits and/or peak period shoulder use?
- Do local jurisdictions have the ability to cite traffic violations as civil offenses when they are considered criminal offenses under state law?
- Has the constitutionality of variable speed limits been challenged? If yes, on what grounds?
- Is automated enforcement (i.e., photo-enforcement) used to enforce the variable speed limits and/or peak period shoulder use?
    - If yes, what basic structure is used to catch violators?
    - If no, what strategies are used with manual enforcement?
- If variable speed limits are used in your jurisdiction, what advance warning is given drivers of the variable speed limit and of enforcement techniques?
  Is there any data or studies indicating the effectiveness of the enforcement approaches?
- Is there any data or studies indicating the cost of the enforcement efforts?
- If automated enforcement is used, has the constitutionality of this type of enforcement been challenged?
    - If yes, on what grounds and what was the result?
    - If there is any case law, please provide a citation or other means of identifying the case.
- Have there been any liability related lawsuits regarding variable speed limits?
    - If yes, what was the claim and outcome? And, what, if any, changes have been made as a result?
    - If no, were there any precautionary or preemptive measures taken to minimize the risk of a liability lawsuit?

These survey questions were sent to the following contacts at various public agencies in the United States.

Ted Trepanier, Washington DOT, email: TrepanT@wsdot.wa.gov
Janice Gipson, Oregon DOT, email: Janice.E.Gispon@odot.state.or.us
Rick Nelson, Nevada DOT, email: rnelson@dot.state.nv.us
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Randal B Thomas, Oregon DOT, email: Randal.B.Thomas@odot.state.or.us
Steve Towen, Arizona DOT, email: stowen@dot.state.az.us
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Oregon and Washington DOTs provided responses from these contacts. The feedback that was provided indicated that the constitutionality of variable speed limits had not been challenged nor have they resulted in liability related lawsuits. Oregon DOT's use of variable speed limits is limited to temporary applications in and around construction zones or in emergency situations (e.g., managing wildfires). Washington DOT has two permanent variable speed limit locations, both of which are on facilities running through a mountain pass; their primary motivation is to post speed limits consistent with the prevailing weather conditions in these passes. One location is IH 90 through Snoqualmie Pass and the other is State Route 2 through Stevens Pass. Both Oregon and Washington use manual enforcement to enforce the variable speed limits; neither has data regarding compliance or effectiveness of this enforcement. Current state statutes in Oregon and Washington support their respective uses of variable speed limits. In Oregon, statues ORS 810.180(8), ORS 810.180(9), and ORS 810.200 allow Oregon DOT to determine the appropriate speed and signs to use for their temporary use of variable speed limits. In Washington, their current state statute RCW 46.61.405 (which can be viewed at http://apps.leg.wa.gov/RCW/default.aspx?cite=46.61.405) provides sufficient support to have permanent variable speed limit locations. Similarly, existing statutes in both states do not preclude the use of shoulders as travel lanes. Washington DOT's report on Active Traffic Management Concept of Operations includes their use of variable speed limits and related measures (Washington DOT, 2008).

7.3 Potential Impediments

The focus of this section is to identify potential impediments to implementing speed harmonization and peak period shoulder use on Texas freeways. The key considerations regarding potential impediments for implementation in Texas are the ITS infrastructure and enforcement mechanisms. The ITS infrastructure is critical for gathering real time information for variable speed limits applicable to current traffic conditions and efficient use of the shoulder during peak congestion periods. The ITS infrastructure can also be beneficial for consistently and effectively enforcing the posted speed limit and appropriate use of the shoulder. To obtain a better and more comprehensive understanding of the potential issues Texas faces, a series of survey questions were developed and disseminated to jurisdictions and agencies (inside and outside of the United States) that have experience with speed harmonization, variable speed limits, and/or peak period shoulder use.

Presented here is an overview of the potential ITS infrastructure and enforcement impediments as well as the survey questions distributed. The survey questions were distributed to a set of potential international contacts found through the literature.

7.3.1 ITS Infrastructure Considerations

Review of the literature regarding previous implementations of speed harmonization and peak period shoulder use indicate there are several functions the ITS infrastructure serves. These are surveillance of current traffic conditions, information dissemination to the motorist, and enforcement.

Technologies used to monitor traffic conditions tend to be a mixture of inductive loop detectors, radar sensors, and cameras of varying sophistications. In the context of speed harmonization and variable speed limits, the inductive loop detectors and radar sensors tend to be used as the key infrastructure providing the data processed by the control algorithm setting the appropriate speed limit. The cameras are used to visually monitor traffic conditions and are particularly useful for ensuring the shoulder is clear for use as a travel lane as well as for identifying incidents aiding incident management efforts.

The loop detectors and radar sensors collecting information about traffic and weather conditions tend to be more densely spaced than the cameras used to visually monitor conditions. Loop detectors and radar sensors spacing depends on the application and location; frequencies of just slightly further than a quarter of mile have been used in the Netherlands. However, there may be facilities on which more or less frequent spacing seems appropriate depending on the potential fluctuations in traffic as well as the degree of saturation for the facility. Computer simulation results indicate the value of effectively implementing speed harmonization before the facility reaches capacity. To aptly time such intervention and to successfully delay the onset of highly congested conditions, a higher density of ITS infrastructure would likely be needed for facilities consistently operating near capacity.

Implementations inside and outside of the United States have consistently disseminated information to motorists via variable message signs and lane use symbols. The variable message signs display the appropriate speed limit for speed harmonization applications. They are either signs placed beside the road or displayed on overhead gantries. Literature review to date indicates the sign spacing is consistent with traditional static speed limit signs. The lane use symbols are used to indicate whether or not the shoulder is available to use as a lane. These are typically placed over the shoulder on an overhead gantry. The literature indicates these overhead gantries tend to be placed roughly every mile. In practice, the spacing will likely depend on the frequency of ingress and egress traffic as well as other roadway geometry conditions such as the inability to use the shoulder due to an approaching physical object.

ITS for enforcement tends to be a combination of radar and cameras to enable automated enforcement for speed harmonization and peak period shoulder use. For speed harmonization, radar tends to be used to measure the vehicle's speed. If motorist is traveling faster than the tolerated speed a camera takes a picture of license plate. One such camera is generally referred to as an Automatic Number Plate Recognition (ANPR) camera. These are used in the United Kingdom. The spacing tends to be based on convenience depending on existing infrastructure conducive to providing an opportunity to deploy such technologies. For example, existing overhead gantries are typically used as a location for the ANPR with radar sensors placed at an appropriate advanced location such that the camera has sufficient time to take a photo of the vehicle violating the speed limit.

Information from survey respondents will help further pinpoint the key ITS requirements and considerations for implementation. This information will be used as a point of reference for Texas freeways.

7.3.2 Enforcement Considerations

As discussed in the previous section, consistent enforcement is considered key for effectively influencing the speeds motorists' drive. In the absence of enforcement, motorists will tend to try to drive as quickly as conditions will permit. In the context of implementing speed harmonization, this is not desirable because such behavior will continue to create turbulent traffic conditions. Turbulent traffic conditions will reduce the speed harmonization's effectiveness at delaying the onset of congestion and improving safety. In the context of peak period shoulder use, consistent enforcement ensures motorists use the shoulder only when permitted and conditions are deemed reasonable for the shoulder lane to be in use.

Manual enforcement and automated enforcement are the two primary means by which to enforce the speed limits and peak period shoulder use. As indicated in Section 7.2, manual enforcement is expensive, time consuming, and less effective than automated enforcement. Automated enforcement offers the benefits of consistency and reduces the necessary manpower to enforce speeds and peak period shoulder use. To be able to enforce the posted speed limits and peak period shoulder use via automated enforcement, Texas would need to pass legislation similar to that passed for enforcing red-light running via automated cameras. Automated speed enforcement is more common outside of the United States; however, applications abroad clearly demonstrate the technology is available and indicate its effectiveness at changing driver behavior to conform consistently to the speed limit and appropriate use of the shoulder lane.

7.3.3 Recommendations

There are two types of impediments to address: global and local. Global impediments are issues relating to the necessary legal framework to be able to implement peak period shoulder use, variable speed limits, and automated enforcement. These potential impediments affect any candidate sites or corridors under consideration. Ensuring existing statutes allow for and support these is a critical up-front task that needs to be addressed. Depending on the existing statutes, following up with additional necessary legislation is another critical task that would need to start early in the process of considering speed harmonization and peak period shoulder use implementation. Local impediments are those specific to a given candidate site or corridor. These potential impediments include considering the existing roadway cross-section, existing density of ITS infrastructure, recurring traffic conditions, traffic mix, and other site-specific characteristics. The degree to which these characteristics coincide with the geometry and ITS recommendations, presented earlier, will determine the feasibility of implementing speed harmonization and peak period shoulder use. We recommend addressing the global impediments noted earlier and then proceeding to address any potential local impediments on a case-by-case basis.

Chapter 8. Feasibility and Operational Deployment Plan

8.1 Feasibility Analysis

This section presents a cost benefit analysis (CBA) framework, which can be applied to assess the potential costs and benefits in implementing speed harmonization and peak period shoulder use on Texas freeways. The ability to assess the relative value of alternative transportation projects is a critical component in making informed decisions for transportation improvements and effective transportation plans for the future. CBA are part of many transportation studies and/or investigations into alternative solutions or projects. The detail to which they are conducted often depends on where in the overall project development process the particular alternative is situated. In earlier planning stages, CBA can be conducted at a rough, sketch-planning level with more comprehensive and detailed CBA following as an alternative enters into the design phases. At each stage of an alternative's development the CBA can serve as one means to screen alternatives and/or modify them to meet the overall project objective or vision. A useful general reference in conducting CBA is the American Association of State Highway Officials' *A Manual of User Benefit Analysis for Highways* (also referred to as the AASHTO Redbook; see AASHTO (2003)).

The CBA framework presented here is not specific to any particular geographical area within Texas; it is oriented towards the generic physical context of Texas freeways. In application, the actual values of the benefits and costs considered in the CBA will depend on the site specific physical and topographical characteristics. The CBA framework is discussed in the context of screening sites for implementing speed harmonization and peak-period shoulder use; the benefits and costs presented are those expected to be attributable to implementing speed harmonization and peak-period shoulder use. The overall intent of this section is to provide an overarching framework TxDOT can use as guidance when considering the potential costs and benefits associated with speed harmonization and peak-period shoulder use implementation. CBA methodologies, potential benefits and costs to consider, and a CBA framework are discussed in the following sections.

8.1.1 Cost Benefit Analysis Methodology

The basic approach for conducting a CBA is to assess the relative difference in benefits and costs associated with a given transportation project or initiative. The focus is to address the question of whether or not a transportation project or initiative is worth the monetary investment. In the course of answering or addressing this question there are number of decisions and assumptions an analyst must make that impact how the CBA is performed and its results. These basic considerations are discussed. For any analyst who has conducted a CBA before, much of this information will be familiar.

Time Period of Analysis and Alternative's Design Life

Two initial considerations are the time period for which the analyses will be conducted and the design life of the proposed project. In the context of speed harmonization and peak period shoulder use, the likely daily time period are the daily peak commuting periods. The design life associated with implementing speed harmonization and peak period shoulder use will be up to the engineer's or analyst's discretion. It is likely to depend on how far into the future he or she anticipates speed harmonization and peak period shoulder use will be employed without any other significant changes to the physical freeway or how it is operated. Time period of the analysis and design life are key parameters because the CBA will consider the total annual benefits and costs for the time periods within each year of the anticipated design life.

Converting Annual Benefits and Costs to a Present Value

The analyst or engineer will quantify the benefits and costs associated with the "do-nothing" and implementation scenarios for each year in the analysis period (the analysis period is equivalent to the proposed project's design life). The annual difference between the benefits for the two scenarios will be calculated as will be done for the annual costs. The annual benefits and annual costs will each be converted to a present value. To calculate the present value, the analyst or engineer must determine a discount rate (minimum rate of return); there is often a federal or state specified discount rate when considering government-funded projects. The annual benefits and costs are unlikely to be uniform over the course of the design life; therefore the following equation would be used to convert the non-uniform annual benefits and costs to present values.

$$PV = \sum_{y=1}^{n} [A_y*(1+i)^{-y}]$$

where, PV = present value, A = annual benefit or cost, i = discount rate, y = index for year in design life of alternative, and n = total number of years in design life.

Comparing Benefits and Costs

Once the present values of the benefits and costs are known, the analyst or engineer can compare the benefits and costs for each site via several different CBA methodologies. Potential methodologies include net present value analysis, benefit costs ratio, and cost effectiveness. These three potential approaches are discussed briefly.

Net Present Value (NPV)

The net present value method is also referred to as net present worth method. The NPV method is a simple comparison of the present value of a project's anticipated monetary benefits and costs. Mathematically, the net present value is computed by subtracting the present value costs from the present value benefits, as shown:

$$NPV = PVB=PVC$$

where, NPV = net present value, PVB = present value of benefits, and PVC = present value of costs. If the NPV is greater than zero, then the project is economically justified (i.e., the anticipated benefits are greater than the anticipated costs).

Benefit Cost Ratio (B/C Ratio)

The benefit cost ratio is similar to the NPV method in that the analyst makes use of the present value of the benefits and the costs. The primary difference is the present value benefits are divided by the present value of cost resulting in a ratio, as shown:

$$BCR = PVB/PVC$$

where, BCR = benefit cost ratio, PVB = present value benefits, and PVC = present value cost. When the benefit cost ratio is greater than 1.0, then the proposed project is considered economically justified. The higher the benefit cost ratio the more attractive the project becomes from an economic perspective.

Cost Effectiveness

Cost effectiveness measures the value an investment produces relative to a specific performance measure. It is most useful when focusing on implementing projects targeting a particular performance measure such as travel time. In such an instance, it may be useful to screen projects based on how effectively each reduces travel time for the cost of the project. The equation that would be used is:

$$\text{Cost Effectiveness} = PVC/(TT_{p,y} - TT_{o,y})$$

where, PVC = present value of cost, $TT_{p,y}$ = travel time for proposed alternative in year y, and $TT_{o,y}$ = travel time for "do-nothing" scenario in year y.

Travel time can be replaced by any performance measure to determine how cost-effective an alternative is at improving a specific measure. The obvious drawback is that this method considers only one potential benefit rather than numerous benefits captured with the NPV and B/C Ratio methods. Cost effectiveness is attractive in situations where unit cost values are not available for a performance measure.

The engineer or analyst will need to decide which is the most appropriate comparison based on the project and guidelines TxDOT may have regarding CBA. The following sections of this report discuss the specific benefits and costs to consider with regards to speed harmonization and peak period shoulder use as well as a CBA framework.

8.1.2 Benefits and Costs

Potential benefits and costs associated with speed harmonization and peak period shoulder use are discussed here. At various stages in the project development process, it may not be feasible to quantify each of the benefits and costs noted. In such instances, it reasonable to screen and/or compare alternatives as long as each CBA has been conducted to an equivalent level of detail. As alternatives progress through the project development process, it is likely to become feasible to quantify each of the benefits and costs discussed and perhaps additional ones as well. Throughout the project development process, alternatives analysis, and/or site screening process, conducting the CBA to the same level of detail for each alternative and/or site is important for equitable consideration of each alternative.

Benefits

Benefits are often used to describe measures or characteristics one hopes to improve with a proposed alternative or project. For example, travel time is a common metric used to represent congestion or delay. Transportation professionals often target reduced travel time as a primary goal or motivation for a transportation project. It is not always feasible to reduce travel time and as a result, a project or proposed alternative may result in a detriment associated with travel time (i.e., an increase in travel time). The relative change for each potential benefit is measured as the difference between a "do-nothing" scenario and the anticipated proposed alternative's performance at some pre-specified future year or over the course of an alternative's anticipated design life.

Table 8.1 presents a summary of potential benefits resulting from speed harmonization and peak period shoulder use.

Table 8.1: Summary of quantitative potential project benefits

Measure	Description
Travel Time	Changes in travel time for network users due to speed harmonization and peak period shoulder use (as compared to "do-nothing" scenario).
Travel Time Reliability	Change in travel time reliability due to speed harmonization and peak period shoulder use (as compared to a "do-nothing" scenario).
Emissions	Change in emissions due to speed harmonization and peak period shoulder use (as compared to a "do-nothing" scenario).
Safety	Change in crash potential due to speed harmonization and peak period shoulder use (as compared to a "do-nothing" scenario).
Fuel Consumption	Change in fuel consumption for system users due to speed harmonization and peak period shoulder use (as compared to a "do-nothing" scenario).

The measures noted in Table 8.1 are focused on characteristics that can be assessed quantitatively and converted to a common monetary unit. There are qualitative measures or non-monetary benefits the analyst may wish to consider during the site screening process; however, such considerations are beyond the scope of this CBA framework. Each potential benefit noted in Table 8.1 is discussed in further detail here.

Travel Time

Travel time savings is a potential benefit to implementing speed harmonization and peak period shoulder use. Travel time serves as a surrogate measure for reducing traffic congestion and improving mobility on the transportation system. To quantify the potential travel time savings, the analyst will need to conduct traffic analysis simulation for the "do-nothing" scenario and the proposed implementation scenario. The analyses will need to be conducted for the peak periods (morning and evening) within each analysis year. A set of simulation runs can be conducted to represent each year and then aggregated up to a total annual peak period travel time per user (see the project handbook for guidance on how to conduct such simulation). The difference between the total annual peak period travel time per user for the "do-nothing" and implementation scenario is the annual potential travel time savings. This difference can be converted to a monetary value using a value of travel time based on the amount a traveler is willing to pay per hour of travel time savings.

Many research studies have looked into the value of travel time. A recent study by Levinson and Tilahun (2006) found travelers to value travel time at $7.44 per hour. The study is based on stated preference data and was used to estimate the value of travel time and travel time reliability in the context of route choice. This value of travel time is presented here as a default value that can be used within the CBA framework for speed harmonization and peak period shoulder use. The study's framework of quantifying the monetary value of travel time within a route choice context makes the value of travel time reasonably applicable for screening cites for speed harmonization and peak period shoulder use. It reflects the intuitive reasoning of the additional value (or benefit) a route with speed harmonization and peak period shoulder use can provide to travelers.

Other studies have found the value of travel time and the value of travel time reliability to vary based on trip purpose, traveler's household income, and in some instances gender and trip duration (Steimetz et al., 2005; Small et al., 2005; Lam and Small, 2001; Gulipalli and Kockelman, 2006; Pinjari and Bhat, 2006). However, the level of detail necessary to decipher between different driver socio-demographics, trip purpose, and trip duration is beyond the scope of this CBA. The CBA framework presented here targets a sketch-planning or screening level of analysis. If the analyst has access to travel time unit cost values specific to the proposed project area and/or additional information regarding travelers' socio-demographics and trip characteristics, more detailed and/or site specific travel time monetary values can be used in replace of the $7.44/hour found by Levinson and Tilahun (2006).

Travel Time Reliability

Travel time reliability measures the variability in travel time; it is typically quantified as the standard deviation from the average travel time. To quantify travel time reliability, the average travel time for the "do-nothing" and implementation scenarios per year will need to be quantified. This can be achieved through the traffic analysis discussed in the travel time subsection of this report. More guidance on quantifying such values can be found in this project's handbook (0-5913-P1). Similar to travel time, the annual difference in travel time reliability can be found by comparing the "do-nothing" and implementation scenarios. A unit cost value for travel time reliability can then be used to convert the estimated changes in travel time reliability to monetary benefits (or detriments).

Similar to the value of travel time, multiple studies have been conducted and produced varying monetary value for travel time reliability. Levison and Tilahun (2006) found travel time reliability to valued at $7.11/hour. As noted above, this value is presented as a default value for converting travel time reliability to a monetary value. This value is considered reasonably applicable in the context of speed harmonization and peak period shoulder use because the value is based on traveler's route choice decisions based on route performance.

Also similar to travel time, other studies have found the value of travel time reliability to vary based on trip purpose, traveler's household income, and in some instances gender and trip duration (Steimetz et al., 2005; Small et al., 2005; Lam and Small, 2001; Gulipalli and Kockelman, 2006; Pinjari and Bhat, 2006). As noted, the level of detail necessary to decipher between different driver socio-demographics, trip purpose, and trip duration is beyond the scope of this CBA. The CBA framework presented here targets a sketch-planning or screening level of analysis. If the analyst has access to travel time unit cost values specific to the proposed project area and/or additional information regarding travelers' socio-demographics and trip characteristics, more detailed and/or site-specific travel time reliability monetary values can be used in replace of the \$7.11/hour found by Levinson and Tilahun (2006).

Emissions

Emissions impacts can be quantified by pollutant for each analysis year and converted to an annual monetary value using the average effective speed, vehicle mix, and information regarding average ambient temperature and humidity. The average effective speed can be obtained from the traffic analysis discussed in the travel time and travel time reliability subsections. The analyst can use MOBILE6.2 to estimate the amount of volatile organic compounds (VOCs), carbon monoxide (CO), nitrogen oxides (NOx), carbon dioxide (CO_2), and particulate matter (PM10) emitted during the "do-nothing" scenario compared to the implementation scenario on an annual basis. Educated assumptions may need to be made regarding future vehicle mix, ambient temperature, and humidity. The total change in emissions per pollutant can be calculated using the emissions factors provided by MOBILE6.2, which are a function of roadway types, temperature, relative humidity, average effective speed, and vehicle type. The total emissions per pollutant are arrived at by multiplying the rates per vehicle type by the VMT for the vehicle type.

The difference in total annual emissions per pollutant between the "do-nothing" scenario and the implementation scenario can be converted into monetary values using unit cost estimates for each pollutant shown in Table 8.2

Table 8.2: Emissions unit cost values

Pollutant (tons)	Cost Estimate (per ton)	In 2009 dollars	Source
VOC	\$4,400	\$5,504	Ozbay and Berechman (2001)
NOx	\$10,300	\$12,884	Ozbay and Berechman (2001)
CO	\$15	\$19	Ozbay and Berechman (2001)
PM10	\$133,000	\$166,366	Ozbay and Berechman (2001)
CO_2	\$50	\$53	Fischer et al. (2007)

Notes: Ozbay and Berechman's work is cited as the most relevant for the unit cost per pollutant because it is the most recent study found available and is cited via the Bureau of Transportation Statistics website (available at: http://www.bts.gov/publications/journal_of_transportation_and_statistics/volume_04_number_01/paper_06/html/table7.html)

Safety

The anticipated safety performance of implementing speed harmonization and peak period shoulder use can be quantified via crash potential models as discussed in Chapter 6. Using the methodology presented in Chapter 6, the analyst can estimate the potential change in the probability of crashes occurring each year. The analyst can use the traffic analysis procedures mentioned in the travel time subsection to quantify the three traffic flow metrics used to calculate crash potential. Those three metrics are the coefficient of variation of speed (i.e., speed variation within each lane), spatial variation of speed (i.e., difference in average speed at upstream and downstream locations), and covariance of volume (i.e., difference in average covariance of volume between adjacent lanes at upstream and downstream locations). These three metrics used in conjunction with the crash potential model lead to an estimated probability of crashes occurring over a specific time period. This estimated probability can be converted a number of crashes by multiplying the probability crashes occurring by the total number of vehicles on the freeway segment for the given analysis period. The analysis periods should be determined such that traffic flow conditions are relatively similar within in each period. The potential number of crashes for each analysis period can then be aggregated up to an annual estimate.

The potential change in number of crashes can be converted to a monetary value using the unit crash costs shown in Table 8.3.

Table 8.3: Unit crash costs by severity

Severity	In 2009 Dollars	Unit Cost[1]	Source
Fatality	$4,259,339.64	$4,100,000	NSC (2009)
Incapacitating Injury	$216,603.00	$208,500	NSC (2009)
Non-incapacitating Evident Injury	$55,267.53	$53,200	NSC (2009)
Possible Injury	$26,283.24	$25,300	NSC (2009)
Property Damage Only (PDO)	$2,389.39	$2,300	NSC (2009)

[1] All unit costs are comprehensive costs rounded to the nearest hundred dollars. Comprehensive costs incorporate the loss of quality of life and are considered the most appropriate unit costs for calculating the value of reducing crash occurrence in the future.

The crash potential prediction model applied in Chapter 6 does not predict the severity of crashes; the model predicts the probability of crashes occurring. As a result, to convert the estimated change in the probability of crashes occurring to a monetary value, the analyst may choose to find a weighted average the values shown in Table 8.3 or may choose to assume the percent of crashes of certain severity remains unchanged in the future (i.e., the percent of PDO crashes in existing conditions is equivalent to the percent of PDO crashes under future conditions). There are obvious drawbacks to both approaches; the analyst will need to use his or her discretion to determine which is the most reasonable based on the level of detail at which the analysis is being conducted. As with all CBA, when screening multiple projects, the same assumption should be used to produce a consistent comparison.

Fuel Consumption

Fuel consumption is an out-of-pocket cost to the user and can serve as a surrogate for indicating the smoothness or turbulence of traffic flow. One of the potential benefits seen from speed harmonization is the reduction in stop and go traffic conditions. Fuel consumption for each analysis year can be calculated using the average speed during the peak periods (found via traffic analysis discussed in the travel time subsection) and the fuel consumption rates shown in Table 8.4. The analyst will need to know the basic traffic mix (light duty versus heavy trucks) for existing conditions as well as each of the future analysis years.

Table 8.4: Fuel consumption rates for light duty vehicles and heavy trucks

Average Speed	Fuel Consumption Rate (miles per gallon)[1]	
	Light Duty Vehicle[2]	Heavy Truck (FHWA Class 8)[3,4]
15	24.4	3.73
20	27.9	4.11
25	30.5	4.41
30	31.7	4.40
35	31.2	4.75
40	31.0	5.06
45	31.6	5.43
50	32.4	5.77
55	32.4	6.26
60	31.4	6.63
65	29.2	7.01
70	26.8	7.53
75	24.8	9.71

[1] Source: Davis et al. (2008)

[2] Light-duty vehicles include passenger cars, sports utility vehicles, pickup trucks and minivans.

[3] Fuel consumption is for dual tires on a tractor and trailer. Fuel economy improves when singlewide tires are used instead (Davis et al., 2008)

[4] Class 8 Heavy Duty Trucks are over 33,000 pounds (15,000 kg) as defined by the Federal Highway Administration.

The difference in fuel consumption between the "do-nothing" and implementation scenarios is converted to a monetary value for each analysis year being considered in CBA. The fuel consumption rate per average speed is converted to a monetary value by multiplying the total fuel consumption (based on average speed, vehicle type, and VMT per vehicle type) by the price of fuel. Due to continuing volatility in fuel costs, the appropriate cost of fuel to be used in the analysis will be determined by the analyst.

Costs

The costs associated with implementing speed harmonization and peak period shoulder use can be categorized into three basic groups. These are capital costs, other potential initial start-up costs, and operations and maintenance. Table 8.5 summarizes the potential costs within each of the groups.

Table 8.5: Summary of quantitative project costs

Item	Description
Capital Costs	
Right-of-Way Acquisition	Costs incurred while acquiring additional right-of-way (if necessary).
Geometric Changes to Facility	Design and construction costs associated with changes to the horizontal and vertical geometry of the facility.
Signing and Pavement Marking Modifications	Design and implementation costs for modifications to upgrade or change existing signing and/or pavement markings.
ITS Infrastructure	Costs incurred to design ITS layout, purchase ITS components, and install ITS system.
Other Potential Initial Costs	
Initial Education Public Education Program	Includes costs for initial public information campaign to inform motorists of new operating procedures during congested periods.
Initial Enforcement Campaign	Costs incurred to ensure consistent, effective enforcement at onset of new operations.
Operations and Maintenance	
Monitoring ITS System Operations	Cost of monitoring system performance in real-time (while under operation).
Evaluating System Effectiveness	Cost incurred to evaluate the effectiveness of the system on a routine basis and to identify potential improvements.
Maintaining ITS Components	Includes costs for routinely maintaining and as necessary, replacing ITS components.
Maintaining Integrity of Physical Road Structure, Signs, Pavement Markings	Cost incurred to maintain the physical integrity of the facility including pavement structure, overhead structures, bridges, signs, and pavement markings.
Continuing Enforcement	Costs incurred to ensure a consistent, effective level of enforcement.

The costs summarized in Table 8.5 serve as a likely list of potential costs associated with speed harmonization and peak period shoulder use implementation. Table 8.5 is not an exhaustive list of potential costs but rather an overview of the type of costs to consider and for which to plan. Some of the costs noted may not be applicable based on the candidate site and there may be additional costs due to unique characteristics at another candidate site. Discretion should be used by the analyst when considering the potential costs and screening sites for speed harmonization and peak period shoulder use.

To remain consistent with how benefits are quantified, the costs should be quantified for the "do-nothing" and implementation scenarios on an annual basis over the course of the design life being used in the analysis.

8.1.3 Cost Benefit Analysis Framework

The project team reviewed the Federal Highway Administration's Intelligent Transportation Systems Deployment Analysis System (IDAS) software to determine if it is applicable for screening potential sites for speed harmonization and peak period shoulder use. We determined IDAS is not applicable. IDAS is currently the most pertinent software package for evaluating the impacts of ITS deployment. The software was developed for use at a sketch-planning level or screening level. It is designed to post-process information directly output by a region's travel demand model. The outputs include each of the potential benefits noted in Table 8.1. However, speed harmonization and hard shoulder running (i.e., peak period shoulder use) are not included as ITS treatments in the current version of IDAS. Therefore, IDAS is not feasible to use as a CBA or screening tool when considering speed harmonization and peak period shoulder use. An alternative framework is outlined in this section. The framework provides an overarching order in which the benefits and costs discussed above can be calculated and compared.

Consistent with many CBA, the basic approach to conducting CBA is to quantify the potential changes in performance measures (i.e., potential benefits) under a "do-nothing" scenario and an alternative "build" or implementation scenario. The difference in performance is converted to a monetary value and compared to the cost of the proposed alternative. As noted, the method for comparing the benefits and costs can be a net present value analysis, benefit cost ratio, cost-effectiveness evaluation, or another similar method. The comparison will indicate whether or not the proposed alternative is economically valid (i.e., whether or not the monetary benefits are anticipated to sufficiently outweigh the costs). A framework for conducting such analysis as related to speed harmonization and peak period shoulder use is presented here.

1) Identify candidate sites for evaluation.
2) Conduct preliminary analyses for "do-nothing" and implementation scenarios per site.
3) Identify design life to be considered in CBA.
4) Identify discount rate (minimum rate of return) to use for CBA.
5) Identify CBA comparison methodology or methodologies (e.g., NPV, B/C ratio).
6) Identify benefits to quantify.
7) Conduct more focused analyses for "do-nothing" and implementation scenarios per site to quantify annual potential benefits over the course of the design life.
8) Use outputs for "do-nothing" and implementation scenarios per site to quantify difference in performance per year of design life.
9) Convert anticipated difference in performance per year to monetary values per year of the design life and convert annual monetary benefits to a total present value.
10) Estimate difference in costs for "do-nothing" and implementation scenario per year of design life and convert annual costs to a total present value.

11) Compare present value monetary benefits and costs via chosen methodology.

This framework can be modified to fit within the standard TxDOT CBA procedures. The critical considerations with regards to speed harmonization and peak period shoulder use are quantifying the related benefits and costs. The challenge in quantifying such benefits is that a sketch-planning tool does not currently exist for such screening. Therefore, to quantify the potential benefits the analyst will need to follow the guidance in the handbook (deliverable from Task 13, 0-5913-P1) related to conducting traffic simulation analysis to obtain the necessary metrics for quantifying travel time, travel time reliability, emissions, safety, and fuel consumption. The analyst will then be able to refer to the section earlier regarding benefits to obtain guidance on how to convert the metrics to monetary values and/or software available for additional post-processing of outputs (e.g., MOBILE6.2 to convert average effective speeds to emissions).

8.1.4 Concluding Remarks

The CBA framework and considerations presented in this section are focused on methodologies, benefits, and costs applicable to implementing speed harmonization and peak period shoulder use on Texas freeways. The framework and level of detail for the analyses target a sketch-planning level or screening level evaluation suitable for identifying candidate sites likely to benefit from speed harmonization and peak period shoulder use. Potential benefits to consider include travel time, travel time reliability, safety, emissions, and fuel consumption, which collectively capture such system performance characteristics as reduced congestion and less turbulent traffic flow. Costs for consideration include initial capital costs associated with construction, right-of-way, and/or initial ITS infrastructure investments. Other initial costs for consideration are public education campaigns and focused enforcement. Finally, also considering the operations and maintenance costs over the design life of the speed harmonization and peak period should use alternative provides a solid foundation for considering the total potential costs for speed harmonization and peak period shoulder use.

Overall, this section provides an overarching framework TxDOT can use as guidance when considering the potential costs and benefits associated with speed harmonization and peak-period shoulder use implementation. Unfortunately, a sketch-planning tool or software package capable of conducting a CBA analysis for speed harmonization and peak period shoulder use does not currently exist. IDAS does not consider speed harmonization or peak period shoulder use within its suite of ITS deployment packages. As a result, the analyst will need to refer to the handbook produced as part of this project (0-5913-P1) for guidance on how to conduct the appropriate traffic analysis simulations to quantify the travel time and speed metrics necessary for assessing the potential benefits of speed harmonization and peak period shoulder use. More detailed or additional guidance regarding CBA can be found by referring to AASHTO's *A Manual of User Benefit Analysis for Highways* (also referred to as the AASHTO Redbook).

8.2 Operational Deployment Plan

This section presents an operational and deployment strategy for speed harmonization and peak period shoulder use. The strategy builds on the cost benefit analysis (CBA) framework discussed in the previous section. The operational and deployment strategy is intended to work with the CBA framework to provide a consistent means for identifying and deploying speed harmonization and peak period shoulder use to promising candidate sites. The CBA framework provides an opportunity to assess the economic validity of deploying speed harmonization and peak period shoulder use to a site. The operational and deployment plan provides information to develop the appropriate control scheme for a site, estimate a site's potential performance, identify infrastructure upgrades, create enforcement and education plans, and consider potential community impacts not directly quantifiable. Each of these elements of the operational and deployment plan is discussed below.

8.2.1 Operational and Deployment Overview

The purpose of the operational and deployment strategy is to intelligently apply speed harmonization and peak period shoulder use as a combined traffic control strategy that delays the onset of severe congestion and increases throughput. Previous deployments in other states and countries as well as traffic simulations run for this research project indicate this combined strategy is also likely to improve travel time reliability, improve safety, reduce emissions, and reduce vehicle fuel consumption. The three key pieces to realizing such benefits are to identify the sites with the most promise for improvement, develop the appropriate speed harmonization and peak period shoulder use operational scheme, and modify the existing traffic control devices and roadway geometry to support and enforce the operational scheme.

To identify sites with the most potential for success, an initial round of candidate sites or corridors will likely be identified based on the severity of the reoccurring congestion during peak commuting hours. This initial group of candidate sites can then be screened and simultaneously prepared for deployment through the following approach.

1) Develop speed harmonization and peak period shoulder schemes applicable to the candidate site based on prevailing traffic conditions and geometric data.
2) Estimate the potential performance for the candidate site; use this output to inform the CBA methodology presented in Section 8.1.
3) Identify the necessary infrastructure improvements to make deployment feasible; use this information to derive a cost estimate for deployment to be incorporated as input to the CBA methodology presented in Section 8.1.
4) Create an enforcement strategy and public education plan to complement the operational scheme developed; estimate the costs of the desired strategy and plan and include in the CBA presented in Section 8.1.
5) Consider the qualitative benefits or disbenefits of deployment to the surrounding community as well as the degree of community support.
6) Based on the outcomes of the CBA, qualitative assessment of other potential benefits and/or disbenefits and degree of community support prioritize the candidate sites.

The following sections provide additional information and guidance to make each of these steps feasible.

8.2.2 Estimating Potential Performance

Travel time, travel time reliability, safety, emissions, and vehicle fuel consumption are all performance measures for implementing speed harmonization and peak period shoulder use. As discussed in Section 8.1, each of these are potential benefits (or detriments) based on the candidate site's estimated performance without and with speed harmonization and peak period shoulder use . An appropriate implementation scheme must be developed to deploy a speed harmonization and peak period shoulder use strategy that is effective at improving the performance measures noted. The following sections discuss how to design an effective speed harmonization and peak period shoulder use scheme as well as how to quantify each of the performance measures noted. Once these performance measures are quantified they can be integrated into the CBA analysis discussed in Section 8.1.

Designing Appropriate Speed Harmonization and Peak Period Shoulder Use Schemes

Traffic simulation plays a crucial role in the design of speed harmonization and peak period shoulder use schemes. Hence, after the selection of a potential corridor, the first step of the analysis is to build a detailed simulation model, both of the local network (for microsimulation purposes), as well as for the "global" network (for mesoscopic simulation purposes).

Depending on the availability of sufficient ITS technologies, there are two forms of speed harmonization: online and offline. If ITS deployment is sufficiently dense, then the online version is preferred. When there is not sufficient ITS, offline algorithms are used. Many control algorithms have been proposed in the literature; however, as we have argued in Chapter 4, simple control strategies are preferred. For completeness, the online and offline control strategies are restated next (for more details we refer to Chapter 4).

Online Algorithm Speed Harmonization

Input

- $(\bar{u}_s, q)$-curves for each of the n road segments. Note that we can extract the maximum capacities $c_0(k)$, $k = 1, 2, .., n$ of the road segments from these curves.
- Current speed limits $s_0(k)$, $k = 1, 2, .., n$ of the road segments.
- The minimum intervention duration T_{min} , *i.e.*, the minimum time interval in which the speed limit remains constant.

Output A set of dynamically changing speed limits for each of the road segments.

INITIALIZATION $c(k) \leftarrow c_0(k), s(k) \leftarrow s_0(k)$

FOR $k = n, n\text{-}1,.., 2$
    IF $q(k) \approx c_0(k)$
        FOR all road segments $r = k\text{-}1, k\text{-}2, \ldots, 1$
            DO select a speed $u(r)$ for segment r using the online VSL algorithm.
            END DO
        END FOR
    END IF
    set $c(r) \leftarrow c_0(r)$
END FOR
Display new speed limit vector $s(r)$
Wait for T_{min} time units, set $s(r) \leftarrow s_0(r)$ and repeat the algorithm.

Recall that temporary shoulder use should always be utilized in conjunction with speed harmonization.

Online Control Temporary Shoulder Use

Step 1. Check if shoulder lane is free of objects. If the shoulder lane is free, go to Step 2; otherwise, repeat Step 1 after some time.

Step 2. Open shoulder lane for traffic.

Step 3. If the average flows on the lanes are less than a pre-specified value, then close the shoulder lane.

After the execution of these algorithms, local performance can be evaluated (see below). Furthermore, based on the above results one can adjust model parameters in the mesoscopic simulation model (see Chapter 3) to obtain the network impacts, if any.

Travel Time

One of the performance measures is travel time (saving). One can focus on the travel time between specific origin-destination pairs or the network-wide travel time (global). Moreover, one can also purely examine the change in travel time on the corridor itself (local). Next we briefly indicate how the travel time savings can be measured.

Local: Run microsimulation to evaluate the total travel time before and after speed harmonization and peak-period shoulder use are applied. The difference amounts to the savings in travel time.

Global: Run a mesoscopic simulation of the entire network and evaluate the total travel time. Adjust parameters in the network-level model to reflect the changes due to the advanced traffic management strategies (see Chapter 3) and evaluate the new total travel time. The difference amounts to the saving in travel time.

Travel Time Reliability

Travel time reliability is a crucial element in the route choice process. Hence it is natural to consider it as a measure of performance. To evaluate this measure, we perform:

Local: Run microsimulation as described (i.e., under the heading "Travel Time"). Instead of evaluating the average total travel time, now the variability of the travel time should be evaluated. This can be accomplished by the calculation of the sample variances before and after the implementation of the traffic management strategies.

Global: Same as above. However, now we use the travel time data obtained from the mesoscopic simulation model to estimate the variance of travel time.

Safety

Safety is an important consideration in transportation systems. Unlike the measures already described, safety is typically a local performance measure. One should not expect to find measurable changes in safety at the network level. To measure safety, we suggest calculating several crash precursors and comparing the change in their values across different scenarios (see Chapter 6).

Ideally, a crash potential function $p(x)$ is estimated based on the specific corridor's crash history. The evaluation of safety then simply reduces to the real-time evaluation of $p(x)$ as a function so the real-time prevailing traffic conditions x.

Emissions and Vehicle Fuel Consumption

Emissions and vehicle fuel consumption are important environmental measures to be considered. Conveniently, these data are standard output in virtually all simulation packages. There are also programs such as MOBILE and MOVES developed by the Environmental Protection Agency; these can be used to supplement simulation outputs. Key inputs for MOBILE and MOVES can be obtained from the simulation outputs discussed.

Local: Run microsimulation "before and after" and examine the differences in emissions/vehicle fuel consumption.

Global: Run mesoscopic simulation "before and after" and examine the differences in emissions/vehicle fuel consumption.

8.2.3 Identifying Infrastructure Improvements

The following sections discuss what is recommended or what has been used in the past for each infrastructure element necessary to deploy speed harmonization and peak period shoulder use. Each of these topics has been covered in additional detail in previous chapters. A synopsis is provided for ease of reference and to help guide the review of each candidate site; essentially, the analyst or engineer will compare the existing features of the candidate site to the desired features. The more the existing features match or coincide with the desired features the more attractive the site becomes based on the infrastructure present. Ultimately, the information here can be used to identify the necessary capital and operational/maintenance costs necessary for the site to be successful; this cost information feeds into the cost benefit analysis methodology presented in Section 8.1 aiding in the overall candidate site screening process.

Intelligent Transportation Systems (ITS)

As discussed in the Section 7.1, ITS is critical for providing accurate information to motorists, collecting information regarding the traffic flow, and enforcing the traffic operation controls in place. In deploying speed harmonization, ITS provides the information necessary to set the appropriate speed limit given the traffic conditions, to communicate that speed limit to motorists and to consistently enforce the speed limit. Similarly, when deploying peak period shoulder use, ITS provides information on when it is best to open and/or close the shoulder to traffic, to communicate whether or not the shoulder is open to motorists, and to consistently enforce the appropriate use of the shoulder.

ITS technologies previously used in speed harmonization and peak period shoulder use can be summarized into three categories of traffic surveillance, information dissemination, and enforcement. Table 8.6 summarizes the recommendations made in the Section 7.1 regarding each of these functions.

Table 8.6: ITS infrastructure recommendations

ITS Function	Recommendation
Traffic Surveillance	Place camera detectors at 1-mile intervals to detect incidents on the main line and shoulders. Place loop detectors at 1,500 to 2,000 foot intervals to gather data regarding traffic flow characteristics.
Information Dissemination	Place variable message signs at 1-mile intervals preferably on overhead gantries.
Enforcement	Place photo radar sensors and cameras at approximately 1-mile intervals. Take care to provide enable the system to provide motorists with ample time to respond to changes in the posted speed limit before enforcing it.

For additional details we refer the reader to Section 7.1.

Horizontal and Vertical Roadway Alignment

As noted in Section 5.3, the roadway geometry is most critical for peak period shoulder use; the deployment of peak period shoulder use changes the operational cross-section of the highway or freeway by adding the equivalent of one or two lanes of traffic. The geometric design guidelines focus on providing an overview of the primary horizontal and vertical alignment considerations applicable to deploying peak period shoulder use. The guidelines were developed in consultation with the *TxDOT Roadway Design Manual*, AASHTO's *Policy on Geometric Design*, and AASHTO's *Roadside Design Guide*. A key assumption made while developing these guidelines is the shoulder will be used as a travel lane under conditions in which the freeway operating speed is 35 mph or less.

Table 8.7 summarizes the basic geometric design guidelines and considerations for using the shoulders as travel lanes.

Table 8.7: Roadway geometric design guidelines and considerations

Geometric Characteristics/Considerations	Guidance
Shoulder Lane Width	10 feet with low to no heavy vehicles in shoulder lane. 11 feet to allow for more extensive use of shoulder lane by heavy vehicles.
Acting Shoulder Width	2 feet to 4 feet to provide shy distance and lateral support to pavement.
Pavement	Structural composition consistent with mainline. Cross slope 2.5% or less; maintain driver comfort, control, and ample drainage.
Horizontal Curves	Verify superelevation and width are adequate/appropriate for vehicle use.
Vertical Clearance	Verify 16.5 feet of vertical clearance across shoulder lanes;

	mitigate discrepancies as specified in *TxDOT Roadway Design Manual*.
Horizontal Clearance	Verify appropriate horizontal clearance of 30 feet for mainline travel and 16 feet for freeway ramps. Mitigate discrepancies via appropriate treatments identified in the *TxDOT Roadway Design Manual* and/or AASHTO's *Roadside Design Guide*.
Transition Areas (Closed to Open Shoulder and vice versa)	Open shoulder at a 10 to 1 taper (one lateral foot for every 10 feet traveled). Close shoulder at a 50 to 1 taper (one lateral foot for every 50 feet traveled).
Entrance/Exit Ramps	Implement yield control for traffic entering freeway on an auxiliary lane (see Section 5.3 for details).
Incident Management	Provide emergency vehicle access via a case-by-case review of each site. Options include managing lanes via lane assignment controls, providing median breaks, and/or recoverable areas adjacent to freeway. Provide vehicle refuge areas every 1/3 of a mile; areas of 15 feet in width and 150 feet in length.
Freeway Operations in Dark	Verify traffic control devices in use meet night-time visibility standards outlined in MUTCD.

As is noted in Section 5.3, each candidate site is likely to present unique and challenging characteristics, solutions to which may require variations from the guidance summarized in Table 8.7 and/or presented in Section 5.3. In such situations, engineers should use their best judgment as to the appropriate mitigations. Additional details regarding these design guidelines can be found in Section 5.3.

8.2.4 Planning For Enforcement And Education

Enforcement and education are two key components to successfully implementing new traffic operations schemes. Enforcement is necessary to ensure motorists comply with the posted regulations and education is critical to ensure motorists understand what is expected of them on the roadway. Each of these components is discussed in more detail here.

Enforcement Considerations

Section 7.3 discusses enforcement considerations in particular detail. Past deployments of speed harmonization and/or peak period shoulder use have illustrated that consistent enforcement is significant in ensuring the traffic control strategy's effectiveness via speed limit compliance. Automated enforcement has been particularly effective in positively influencing drivers' tendency to obey the speed limit. Consider the following study initiated in 2001 and conducted in Washington, D.C. and Baltimore, Maryland:

> Washington, D.C. implemented automated enforcement for speeding on several surface streets. The automated speed enforcement primarily consisted of cameras triggered to take a photograph when the associated Doppler radar speed sensor indicated a vehicle was traveling faster than a preset speed. A set of comparable sites

> in Baltimore, Maryland was left untreated. The study found that in Washington, D.C. the mean speed dropped 14% and vehicles exceeding the speed limit by more than 10 mph dropped 82%. The sites in Baltimore, Maryland experienced no significant change in mean speed or the percent of vehicles exceeding 10 mph (Retting and Farmer, 2003).

Clearly, consistent automated speed enforcement can make a significant impact on speed compliance. Speed compliance is critical for deploying a successful speed harmonization and peak period shoulder use traffic control strategy.

While there have been case studies in the United States, automated speed enforcement tends to be less common in the United States than abroad, particularly compared to European countries. Despite its scarce use in the United States, its proven effectiveness makes it a priority recommendation for successfully implementing speed harmonization and peak period shoulder use. Section 7.3 discusses some of the obstacles facing automated speed enforcement in the United States as well as recommendations in developing the legal framework necessary to use automated enforcement techniques in the United States. Listed here are the key elements of variable speed limit legislation recommended by Hines and McDaniel (2002) in their National Highway Cooperative Research Project (NCHRP) publication entitled *Judicial Enforcement of Variable Speed Limits* (NCHRP, 2001):

1. The statutory purpose should allow a change in speed limit to protect public safety and permit the legislature to delegate to an agency power to prescribe details after they have fixed a primary policy or standard.
2. The law should require the change in the speed limit to be based on engineering and traffic investigations; in the context of variable speed limits, these would show the need for and benefit of variable speed limits under certain situations.
3. The statute must require posting for the new limit to be effective.
4. The statue must require posting of advance warning that the legal speed limit is changed ahead.
5. The law must require any information or charging documents include the existing speed limit and speed at which it is alleged the charged driver's vehicle was traveling.
6. The law might prohibit automatic enforcement within a certain distance of the new limit to allow reasonable time for driver's to adjust their speeds.
7. The law should provide broad discretion to administrative agency for enactment of regulations and sub-delegation of decision-making power.
8. Either laws or regulations should provide for certain evidence by affidavit. This means where the speed limit is decreased due to temporary hazards (e.g., traffic, weather) evidence of the reasons and the specific speed limit on the highway where the violation allegedly occurred must be presented.

For additional details and information, please refer to Section 7.3.

Education Considerations

Public education for new operating strategies and traffic control devices can be useful in proactively informing the public of what is expected of them under certain conditions. Deploying speed harmonization and peak period shoulder is likely to result in modifying the character and appearance of the roadway as well as implementing new signs or traffic control devices intended to convey critical information to motorists. In addition to traditional public outreach meetings, simple informational flyers included in utility bills, short T.V. commercials, public announcements via radio, and informational flyers made available for pickup at grocery stores, schools, and libraries can make it easier for motorists to understand the purpose for the changes, what is expected of them, and the benefits intended to come out of the new traffic control strategies. Studies regarding past speed harmonization and peak period shoulder use deployment have not explicitly discussed an education component; however, it seems clearly beneficial to consider such a component when altering some of the basic operational characteristics motorists' may take for granted.

8.2.5 Considering Qualitative Characteristics and Community Support

Thus far feasibility and deployment considerations have been focused on quantifiable benefits and costs; however not all potential impacts can be quantified, but are still worth considering qualitatively. Many of these measures are complex and are related to societal considerations not immediately conducive to representing with a numerical value (e.g., community cohesion). There are a few measures, such as noise, that can be quantified with more detailed analysis; however, this detailed analysis necessary may be beyond the scope of many screening exercises. To be able to capture these measures in some form during the screening and deployment process, the analyst can qualitatively assess them.

Table 8.8 summarizes potential qualitative measures for consideration.

Table 8.8: Potential qualitative characteristics

Measure	Description
Noise	Anticipated change in noise pollution due to change in traffic volume and/or mix to traffic.
Accessibility	Ability to access basic services (e.g., schools), employers, quality of life destinations (e.g., shopping), and local access (e.g., sidewalks).
Community Cohesion	The degree to which existing neighborhoods, communities, and recreational areas remain intact. Considers residents and local businesses necessary to relocate and/or residents and local businesses isolated from the community.
Equity	Distributive effect of the proposed project; what is the investment's impact across societal groups?
Environmental Considerations	Impacts on water resources, wetlands, habitats of endangered/threatened species, and other similar considerations.
Regional Development/ Economic Effects	Assessment of whether proposed project would attract new development or employers to the region.
Aesthetics	Visual impact of proposed project compared to "do-nothing" scenario.

The measures listed are not exhaustive nor will they be applicable for all candidate sites. Table 8.8 is provided as a reference to help guide the conscious and consistent consideration of welfare measures not conducive to quantifying numerically.

In addition to considering the qualitative measures noted, holding public meetings to gather thoughts from the community and gage community support is likely to be particularly useful in identifying candidate sites most conducive to speed harmonization and peak period shoulder use. As with many transportation initiatives, gaining community support can be a powerful catalyst in implementing new traffic control strategies.

8.2.6 Summary

The operational and deployment strategy presented here is intended to guide decisions to deploy speed harmonization and peak period shoulder use to candidate sites most likely to benefit from such strategies. The plan presents a framework for developing effective speed harmonization and peak period shoulder use schemes as well as assessing the potential performance of a candidate site, the necessary infrastructure upgrades, and the corresponding enforcement and education plans. Coupled with the CBA framework presented in Section 8.1, the operational deployment strategy provides a holistic approach to screening candidate sites while simultaneously preparing for successfully implementing speed harmonization and peak period shoulder use.

Chapter 9. Conclusions

Speed harmonization, peak-period shoulder use, and ramp metering are promising active traffic management (ATM) strategies for dealing with the increasing levels of congestion in urban areas around the globe. In this report we investigated their implementations on Texas freeways. To this end, we developed a comprehensive framework to evaluate their traffic operations and safety impacts on Texas freeways. In particular, we implemented efficient control algorithms for these ATM strategies, presented a multi-resolution simulation framework to evaluate their network-level effects, evaluated traffic operations and safety benefits of the ATM strategies through the development of multiple interdependent models, and made recommendations on the ITS devices requirement and enforcement. We also discussed potential impediments in their implementations. A cost-benefit framework has been presented to determine economic viability of these strategies. We have summarized these crucial steps in a comprehensive operational deployment plan.

ATM strategies were implemented in the testbed section under four different scenarios (variable speed limits (VSL), peak-period shoulder use, VSL with shoulder use, and ramp metering) to assess their impact on traffic operations and safety. The results obtained from the implementation of the ATM strategies are summarized below.

VSL harmonized traffic flow, reduced lane-changing conflicts, and created safer driving condition. VSL reduced both the likelihood and severity of conflicts. On the other hand, shoulder use increased the likelihood and severity of conflicts, and lead to significant increase in crossing conflicts at network level. Shoulder use also decreased speed at the end of the shoulder-use segment due to bottleneck creation as a result of one lane-drop. However, shoulder use improved traffic conditions in the middle of the shoulder-use segment by reducing traffic density and increasing operating speed.

Simultaneous implementation of VSL and shoulder use consistently harmonized the traffic flow and improved travel condition by reducing speed variability, traffic density, and stop-n-go condition (by reducing the number of stops per vehicle). Smoother flow of traffic results in less emission, less fuel consumption, and less wear and tear for the vehicles, and leads to safer driving conditions. It did not have significant impact on throughput, as is reported in previous studies on VSL. It had an overall positive effect on speed harmonization for the traffic, reduced the severity of conflicts, but increased the likelihood of conflicts. VSL and shoulder use decreased delay per vehicle, and overall these strategies created safer driving condition by reducing speed variability.

Ramp metering reduced the average number of stops per vehicle and speed variation. It also reduced the freeway corridor delay, but at the expense of overall network delay that worsened due to vehicles queued at on-ramps during peak-hours. Ramp metering's effect on safety was similar to VSL's in that it decreased both the likelihood and severity of conflicts. Overall, ramp metering had a positive impact on safety.

Shoulder use implementation results point to a need for comprehensive safety evaluation before real-life implementation, especially towards the end of the shoulder-use segment. To the best knowledge of the authors, this is the first study that implemented VSL and peak-period shoulder used simultaneously, and used the SSAM for extensive safety analysis. Further work on the related topics is still needed.

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Appendix A: Vehicle Actuated Programming (VAP) Source Code for Variable Speed Limits

```
PROGRAM Spl_VMS; /* Q:\VISSIM\DATEN\_PTV\VBA\Spl_VMS.vv */

VAP_Frequency 1;

CONST
  F = 2.0,
  DT = 1,
  ALPHA = 0.5,
  Qon = 1200,
        Interval = 300; /* time interval at which VSL will check conditions */

/* ARRAYS */

/* SUBROUTINES */

/* PARAMETERS DEPENDENT ON SCJ-PROGRAM */

/* EXPRESSIONS */

/* MAIN PROGRAM */

 IF NOT initialized THEN
        initialized := 1;
        desSpeed := 65;

        /* Initialize speeds at VSL locations */
        set_des_speed(18, 10, desSpeed); set_des_speed(18, 20, desSpeed);
        set_des_speed(19, 20, desSpeed); set_des_speed(19, 20, desSpeed);
        set_des_speed(20, 20, desSpeed); set_des_speed(20, 20, desSpeed);

        SumSpeed :=0;         /*To calculate ave. speed over many loops for a single loop of
"Interval"; AvSpeed = SumSpeed/ NumVeh*/
        NumVeh :=0;

        Set_sg_direct( 1, Off );
        Start( evalInt )
 END;



        Detect:= (Detection( 1 ) + Detection( 2 ) +Detection( 3 ) +Detection( 11 ) +Detection( 12
) +Detection( 13 ) );
```

```
        IF Detect > 0 THEN

        SumSpeed := SumSpeed + (Velocity( 1 )*Detection( 1 ) + Velocity( 2 )*Detection( 2 ) +
Velocity( 3 )*Detection( 3 )+
                Velocity( 11 )*Detection( 11 ) + Velocity( 12 )*Detection( 12 ) + Velocity( 13
)*Detection( 13 ));
        NumVeh := NumVeh + Detect;

        END;

 IF evalInt = Interval*DT THEN

        qCarPrev := qCar; qHGVPrev := qHGV;
        qCar1 := Front_ends( 1 ) * (3600/Interval) / DT;
        qCar2 := Front_ends( 2 ) * (3600/Interval) / DT;
        qCar3 := Front_ends( 3 ) * (3600/Interval) / DT;
        qCar := qCar1 + qCar2 + qCar3;

        qCarZ := (ALPHA * qCar) + ((1.0 - ALPHA) * qCarPrev);
        Clear_Front_ends( 1 ); Clear_Front_ends( 2 );
        Clear_Front_ends( 3 );

        qHGV1 := Front_ends( 11 ) * (3600/Interval) / DT;
        qHGV2 := Front_ends( 12 ) * (3600/Interval) / DT;
        qHGV3 := Front_ends( 13 ) * (3600/Interval) / DT;
        qHGV := qHGV1 + qHGV2 + qHGV3;

        qHGVZ := (ALPHA * qHGV) + ((1.0 - ALPHA) * qHGVPrev);
        Clear_Front_ends( 11 ); Clear_Front_ends( 12 );
        Clear_Front_ends( 13 );

        Qb := qCarZ + F*qHGVZ;
        Q_tot := (Qcar+QHGV)*(Interval/3600)*DT;

        flow := Qb/3; /*flow per hour per lane vphpl */

        Occ := (Occup_rate( 1 ) + Occup_rate( 2 ) +Occup_rate( 3 ) )*100/3;


        IF NumVeh > 0 THEN
                AvSpeed := (SumSpeed/NumVeh)* 2.236936; /*Unit conversion from m/s to
mph */
                Q_5min := NumVeh;
        ELSE AvSpeed :=0; Q_5min := NumVeh;
        END;
```

```
        SumSpeed :=0;
        NumVeh :=0;


        Reset( evalInt ); Start( evalInt );

        /* Calculate average time mean speed */

        IF flow <= Qon THEN
                IF Occ <= 15 THEN
                        desSpeed := 65
                ELSE IF AvSpeed > 50 THEN
                                desSpeed := 65
                                ELSE IF ( (AvSpeed <= 50) AND (AvSpeed >= 40)) THEN
                                        desSpeed := 50
                                ELSE desSpeed := 40
                                END;
                        END;
                END;
        END;

        ELSE IF flow > Qon THEN
                IF AvSpeed > 50 THEN
                        desSpeed := 65
                        ELSE IF ( (AvSpeed <= 50) AND (AvSpeed >= 40)) THEN
                                desSpeed := 50
                        ELSE desSpeed := 40
                        END;
                END;
        END;


END;

Set_des_speed( 18, 10, desSpeed); Set_des_speed( 19, 10, desSpeed);
Set_des_speed( 20, 10, desSpeed);
Set_des_speed( 18, 20, desSpeed); Set_des_speed( 19, 20, desSpeed);
Set_des_speed( 20, 20, desSpeed);
Record_value( 1, flow ); Record_value( 3, Occ);Record_value( 2, desSpeed);
Record_value( 4, SumSpeed ); Record_value( 5, NumVeh); Record_value(6, AvSpeed);
        Record_value(7, Q_tot);Record_value( 8, Q_5min)
.
/*--------------------------------------------------------------------------*/
```

Appendix B: Vehicle Actuated Programming (VAP) Source Code for ALINEA (Ramp Metering)

```
PROGRAM ALINEA; /* E:\Simulation\mopac_rampMetering\ALINEA.vv */

VAP_FREQUENCY 1;

CONST
        MAX_LANE = 3,
        KR = 70,
        OCC_OPT = 0.25;

/* ARRAYS */
ARRAY
        detNo[ 3, 1 ] = [[11], [12], [13]];

/* SUBROUTINES */

/* PARAMETERS DEPENDENT ON SCJ-PROGRAM */
        IF( prog_aktiv = 1 ) AND ( prog_aktiv0vv <> 1 ) THEN
          prog_aktiv0vv := 1;
          DT := 1;
        ELSE IF( prog_aktiv = 2 ) AND ( prog_aktiv0vv <> 2 ) THEN
          prog_aktiv0vv := 2;
          DT := 1;
        END END;

/* EXPRESSIONS */
        Demand := Detection( 1 );

/* MAIN PROGRAM */

S00Z001:  IF NOT init THEN
S01Z001:    init := 1;
S01Z002:    Set_sg( 1 , off )
        END;
S00Z004:  cyc_sec := cyc_sec + 1;
S00Z005:  IF cyc_sec >= cyc_length THEN
S01Z005:    cyc_sec := 0
        END;
S00Z007:  Set_cycle_second( cyc_sec );
S00Z008:  laneNo := 1;
S00Z010:  IF laneNo <= MAX_LANE THEN
S01Z010:    IF detNo[ laneNo, 1 ] > 0 THEN
S02Z010:      oout := oout + Occup_rate( detNo[ laneNo, 1 ]);
S02Z011:      laneNo := laneNo + 1;
          GOTO S00Z010
```

```
        END
       END;
S00Z013:   timer_dc := timer_dc + 1;
S00Z014:   IF timer_dc = (60 * DT) THEN
S01Z014:     timer_dc := 0;
S01Z015:     qRamp := (Front_ends( 2 )); Clear_front_ends( 2 );
S01Z016:     oout := oout / MAX_LANE / (60*DT);
S01Z017:     cqRamp := qRamp + KR * (OCC_OPT - oout);
S01Z018:     cyc_length := 60*DT / cqRamp;
S01Z019:     oout100 := oout * 100; RecVal( 1, oout100 );
S01Z020:     oout := 0
       END;
S00Z023:   IF cyc_length < 4 THEN
S01Z023:     Set_sg( 1 , off )
       ELSE
S00Z024:     IF Demand THEN
S01Z024:       IF cyc_sec = 0 THEN
S02Z025:         Set_sg( 1 , redamber );
S02Z026:         cyc_sec := 0
         ELSE
S01Z025:         IF T_red( 1 ) >= cyc_length-3 THEN
           GOTO S02Z025
          ELSE
S00Z027:           IF Current_state( 1, redamber ) THEN
S01Z027:             Set_sg( 1 , off )
           ELSE
S00Z028:             IF Current_state( 1, off ) THEN
S01Z028:               IF NOT (cyc_length < 4) THEN
S01Z029:                 Set_sg( 1 , amber )
             END
            ELSE
S00Z030:               IF Current_state( 1, amber ) THEN
S01Z030:                 Set_sg( 1 , red )
             END
            END
           END
          END
         END
        ELSE
         GOTO S00Z027
        END
       END;
S00Z032:   RecVal( 2, cyc_length );
S00Z033:   qRampHour := qRamp * 60 / DT; RecVal( 3, qRampHour )
PROG_ENDE:   .
/*------------------------------------------------------------------------*/ .
```

Appendix C: VISSIM Simulations Results for Base Case

		Run Number	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10
		Random Seed Number	1	6	11	16	21	26	31	36	41	46
Performance Measures	Average Value											
Lane Changes	68617.10		69099.00	69789.00	70097.00	70470.00	65063.00	67793.00	70346.00	68122.00	66196.00	69196.00
Travel Time(h)	2236.76		2219.40	2151.76	2309.54	2215.07	2582.53	2136.20	2112.68	2201.20	2180.35	2258.88
# Stop/Vehicle	16.90		16.81	15.99	17.23	16.22	21.13	15.92	15.33	16.21	17.36	16.76
Delay	71.74		71.66	65.80	69.08	66.90	83.48	74.61	65.98	71.03	78.70	70.11
Throughput DC2	8457.50		8444.00	8438.00	8539.00	8503.00	8580.00	8392.00	8427.00	8429.00	8291.00	8532.00
Throughput DC3	10406.10		10360.00	10332.00	10451.00	10507.00	10626.00	10321.00	10375.00	10390.00	10178.00	10521.00
Speed DC2	24.94		23.24	24.69	25.88	24.44	27.11	24.82	24.86	23.85	23.09	27.38
Speed DC3	33.08		34.71	34.70	32.57	33.52	25.12	34.56	34.87	33.84	34.86	32.09
Tot Delay/Veh	362.79		380.00	352.60	320.30	371.50	386.30	384.40	358.90	350.20	415.40	308.30
Stopped Delay/veh	47.30		48.80	44.50	39.40	44.20	49.40	56.60	45.40	43.50	59.70	41.50
CVS (within lane)	0.54		0.55	0.51	0.56	0.59	0.53	0.65	0.50	0.47	0.52	0.55
CVS (across lanes)	0.52		0.62	0.45	0.49	0.53	0.39	0.40	0.56	0.54	0.60	0.65
Density	0.51		0.51	0.49	0.47	0.76	0.46	0.52	0.43	0.33	0.65	0.44

Appendix D: VISSIM Simulations Results for VSL

		Run Number	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10
		Random Seed Number	1	6	11	16	21	26	31	36	41	46
Performance Measures	Average Value											
Lane Changes	48140.70		48418.00	46737.00	49765.00	47873.00	49989.00	44597.00	47543.00	48513.00	49393.00	48579.00
Travel Time(h)	2346.30		2313.87	2268.98	2194.34	2333.34	2740.84	2318.94	2294.07	2215.29	2352.19	2431.17
# Stop/Vehicle	16.70		15.85	15.64	13.43	17.30	21.52	17.50	16.67	14.34	17.17	17.60
Delay	76.89		72.85	73.93	59.24	80.38	93.57	84.19	75.83	66.50	81.14	81.29
Throughput DC2	8189.50		8266.00	8207.00	8377.00	8043.00	8349.00	7998.00	8045.00	8230.00	8219.00	8161.00
Throughput DC3	10104.40		10199.00	10103.00	10291.00	9984.00	10261.00	9903.00	9910.00	10189.00	10093.00	10111.00
Speed DC2	20.45		20.29	20.24	21.29	20.43	20.18	20.29	20.23	20.87	20.51	20.20
Speed DC3	27.20		27.61	27.67	27.67	27.69	23.26	27.73	27.76	27.78	27.24	27.56
Tot Delay/Veh	259.53		256.60	260.40	186.50	287.50	284.60	307.10	254.50	226.50	276.20	255.40
Stopped Delay/veh	51.67		48.60	53.30	32.30	60.80	53.00	67.60	48.90	42.40	57.40	52.40
CVS (within lane)	0.49		0.49	0.43	0.48	0.49	0.34	0.56	0.45	0.49	0.61	0.53
CVS (across lanes)	0.44		0.35	0.42	0.58	0.51	0.38	0.43	0.37	0.46	0.45	0.43
Density	0.48		0.41	0.49	0.50	0.47	0.58	0.45	0.45	0.44	0.47	0.58

Appendix E: VISSIM Simulations Results for VSL & Shoulder Use

		Run Number	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10
		Random Seed Number	1	6	11	16	21	26	31	36	41	46
Performance Measures	Average Value											
Lane Changes	53783.50		52913.00	53462.00	54481.00	53846.00	55481.00	53611.00	53276.00	52025.00	55436.00	53304.00
Travel Time(h)	2331.70		2240.81	2265.16	2148.26	2301.16	2361.97	2349.71	2270.64	2501.66	2341.10	2536.57
# Stop/Vehicle	14.70		12.84	13.83	11.38	14.08	15.30	14.56	13.98	17.24	15.21	18.56
Delay	56.64		49.17	54.78	44.50	53.73	59.58	55.71	54.45	65.91	57.32	71.24
Throughput DC2	8591.10		8592.00	8594.00	8579.00	8574.00	8602.00	8607.00	8597.00	8603.00	8568.00	8595.00
Throughput DC3	10556.00		10608.00	10533.00	10650.00	10621.00	10564.00	10625.00	10492.00	10489.00	10510.00	10468.00
Speed DC2	25.72		25.50	26.80	26.60	25.50	25.70	25.30	26.20	25.50	24.60	25.50
Speed DC3	21.12		21.90	20.80	21.30	21.20	20.90	21.30	21.10	21.00	21.00	20.70
Tot Delay/Veh	151.91		130.20	145.50	118.10	148.90	160.70	153.00	150.90	170.20	171.70	169.90
Stopped Delay/veh	25.24		21.30	24.90	18.90	23.50	27.20	26.70	24.60	28.20	29.00	28.10
CVS (within lane)	0.05		0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.13	0.28	0.01
CVS (across lanes)	0.83		0.41	0.81	0.55	0.95	0.64	0.90	0.62	1.13	1.64	0.65
Density	0.33		0.26	0.35	0.33	0.31	0.29	0.28	0.28	0.45	0.39	0.33

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