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16. Abstract

This report summarizes the methodology developed for the evaluation of excess capacity available on arterial streets and frontage roads for use during diversion of freeway traffic to alleviate recurring and non-recurring congestion. An evaluation of the IH-10/Fredericksburg Road corridor in San Antonio was performed to determine the excess capacity available for diverted traffic during the peak and off-peak periods. A simulation study was also performed to investigate the amount of diversion and its benefits to the corridor during the recurring P.M. peak hour congestion in the IH-10/Fredericksburg corridor in San Antonio. An investigation of guidelines for the use of Changeable Message Signs on arterials and past experience with their use was also performed in this study.

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REAL-TIME ASSESSMENT AND USE OF ARTERIAL STREET CAPACITY FOR FREEWAY DIVERSION ROUTING

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IMPLEMENTATION RECOMMENDATIONS

With the installation of Intelligent Transportation Systems in Texas cities, such as TransGuide in San Antonio, the Texas Department of Transportation is investing considerable effort and resources in improving traffic management capabilities to mitigate recurring and non-recurring congestion on freeways. Diversion of traffic from freeways to alternate routes is one of the strategies being investigated. The findings of this study will be helpful to the department in evaluating the existing traffic and geometric conditions on arterials and frontage roads to determine the feasibility of using them as diversion routes. The findings of this study are immediately useful for the IH-10/Fredericksburg Road corridor in San Antonio, Texas where the department is currently investigating the feasibility of implementing diversion strategies to alleviate recurring and non-recurring congestion on IH-10. The following implementation recommendations are made.

1. The methodology developed to evaluate available capacity of a facility could be used to evaluate potential diversion routes. This will help avoid implementing expensive systems in the field if sufficient/reasonable additional capacity is not available.

2. The methodology can be used to identify bottlenecks, and can be used for planning capacity enhancements, construction zone planning/design and managing major events.

3. The Changeable Message Signs (CMSs) implementation and design guidelines could be used to plan placement of CMSs for diversion routes.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views of policies or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.

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SUMMARY

The implementation of ITS systems in Texas, including the TransGuide center of San Antonio, increased the interest in corridor traffic management strategies. One of the strategies being considered in Texas is the use of excess capacity available on arterial streets and frontage roads to divert traffic from freeways in order to relieve recurring and non-recurring congestion. This study investigates the feasibility of using integrated corridor traffic management strategies for relieving recurring and non-recurring congestion in the IH-10/Fredericksburg Road corridor. Although the study corridor was used as a test site for the application of the methodologies developed in this study, the methodologies are applicable to any arterial or frontage road.

A methodology for the estimation of excess capacity available at signalized intersections and interchanges for diverted traffic was developed. This methodology was applied to determine the excess capacity in the outbound direction of Fredericksburg Road and the IH-10 frontage road during the peak and off-peak periods.

It was found that during the A.M. and P.M. peak periods, the section of Fredericksburg Road south of IH-410 has some excess capacity for outbound traffic diverted from IH-10. The section north of IH-410, however, was found to have no excess capacity due to the large cross street and turning movement volumes. During the off-peak period, however, both sections of Fredericksburg have excess capacity for diverted traffic.

The outbound IH-10 frontage road was found to have excess capacity during the off-peak period. During the A.M. peak period, Callaghan, and Huebner interchanges do not have any excess capacity. Vance Jackson, Crossroads, and Hildebrand have a small amount of excess capacity for diverted traffic. Hildebrand, Vance Jackson, and West Avenue were also found to be critical during the P.M. peak period. Geometric improvements at Hildebrand, Fresno, and West Avenue were found to increase the capacity for the outbound through movement.

For the interchanges, it was found that the three-phase timing plan for diamond interchanges results in higher capacity for the frontage road through movement than TTI-Lead phasing. This is because in TTI-Lead phasing a large amount of green time is allocated to the interior movements in order to avoid interior storage of vehicles. It was also observed that with three-phase timing plans long cycle lengths are not possible due to storage problems.

A simulation study, using INTEGRATION, was conducted to study the amount of diversion to the frontage road and Fredericksburg when timing plans to create excess capacity for the outbound traffic are implemented. Although the model needs further calibration, several observations were made. The simulation revealed that despite the P.M. hour

congestion experienced by the outbound IH-10 traffic, the travel time on the frontage road as well as Fredericksburg is higher. Therefore, no significant traffic diversion was observed.

Although the simulation logic did not result in any diversion, IH-410 traffic was diverted to Fredericksburg extraneously, to study if the timing plans provided on Fredericksburg could actually accommodate any excess diverted traffic. It was found that, as estimated, the section of Fredericksburg south of IH-410 can accommodate more traffic with the proposed timing plans for the arterial. As projected, Crossroads intersection and the IH-410 interchange were found to be critical in determining the amount of traffic that can be diverted.

It was found that weaving at IH-410 entrance and exit ramps resulted in increased travel time for the outbound traffic. Diverting IH-410 bound traffic from IH-10 to Fredericksburg reduced weaving activity in the vicinity of IH-410 entrance and exit ramps. This resulted in better speeds and travel time for the outbound freeway traffic upto the IH-410 interchange. New weaving problems upstream of Heubner exit occurred in the absence of the weaving problems at IH-410.

The simulation and PASSER II/PASSER III analysis showed that it may not be beneficial to divert traffic to the frontage road and Fredericksburg to alleviate recurring peak hour congestion on IH-10 due to the lack of additional capacity and long travel times. However, during the off-peak period, diversion may be feasible because of the availability of excess capacity both on the frontage road as well as Fredericksburg Road. Diversion to the frontage road may be more beneficial due to the fewer number of signalized intersections on the frontage road.

Researchers conducted an investigation of the past experience with the use of changeable message signs (CMSs) on arterials. Guidelines for the use of CMSs on arterials is also presented. Because it was found that diversion to Fredericksburg is not beneficial during the peak periods, and diversion to frontage roads may be more beneficial during off-peak periods, the use of CMSs on Fredericksburg Road may not be warranted. However, if the excess capacity in the south section of Fredericksburg is used to divert IH-410 bound traffic, one CMSs, on the east side of Fredericksburg interchange with IH-10 is recommended. On the frontage road, one CMS, at IH-410/Cherry Ridge interchange is recommended to provide trail blazing information to IH-10 outbound traffic diverted to the frontage road.

CHAPTER I.
INTRODUCTION

Transportation professionals are rethinking traffic management since the enactment of the Intermodal Surface Transportation Efficiency Act (ISTEA). ISTEA envisions a transportation system that provides seamless mobility to users across jurisdictional boundaries and different transportation modes. In response to this vision, various states, including Texas, have started implementing Intelligent Transportation Systems (ITS).

One of the ITS systems implemented in the state of Texas is San Antonio's TransGuide system. The system is currently online and capable of detecting, confirming, and guiding motorists away from incidents on freeways.

San Antonio's TransGuide is the first of its kind in the country utilizing high speed computerized communications technologies to immediately respond to traffic congestion and inform drivers of changing road conditions. The first phase of the TransGuide system (encompassing 42 kilometers of highway near the downtown area of San Antonio) includes 50 variable message signs and 359 lane control signals . The system also uses 52 cameras to help identify trouble spots and dispatch the appropriate assistance. The system will ultimately cover 308 kilometers of highway in Bexar County.

The TransGuide system provides facilities that can be utilized to alleviate recurring and non-recurring congestion through integrated corridor traffic management strategies. This study investigates the issues involved in traffic diversion from freeways to parallel arterials and frontage roads to alleviate congestion, with particular emphasis on the IH-10/Frederickburg corridor in San Antonio.

INTEGRATED TRAFFIC MANAGEMENT

Although integrated traffic management has commanded considerable amount of interest from transportation professionals in the last few years, it is now close to being a reality due to the development of facilities such as TransGuide, with their centralized monitoring and control capabilities. TransGuide also brings several agencies in charge of operating and maintaining the transportation infrastructure under one roof, further facilitating integrated management of facilities under different jurisdictions.

The objective of integrated corridor traffic management is to obtain an optimum use of capacity during incident-induced and/or recurring congestion within a freeway corridor consisting of the freeway, its frontage roads, and other parallel arterials (*1*). This objective is accomplished through coordination of various control and driver information systems to facilitate the movement of traffic between the freeway and adjacent urban arterial streets. This type of system contains traffic responsive capabilities and the ability to evaluate and implement operational control strategies that will optimize traffic flow within the corridor.

As TransGuide is now operational, plans are being made to use frontage roads and arterial streets as alternative routes for traffic diverted during incidents and recurring congestion. In order to realize this, it is essential to develop methodologies for evaluating the capacity of the arterial streets and frontage roads to handle traffic diverted from freeways during incidents and recurring congestion.

ALTERNATE ROUTES

Large urban areas, such as San Antonio, have very complex and extensive systems of arterials and freeways. It is, however, not possible to use every arterial as an alternate route to major freeways to alleviate recurring and non-recurring congestion. Potential alternate routes need to have certain geometric characteristics with respect to the freeway facility for which they function. Some of these characteristics and examples of arterials in San Antonio that possess these characteristics are discussed below.

There are several arterials that can serve as alternate routes to freeways in San Antonio. Arterials that form a "D" with a freeway are preferable as alternate routes because motorists who are diverted to the arterial can remain on the arterial and get back to the freeway at the other end. Some such arterials in San Antonio include : Fredericksburg (IH-10), Laredo Highway (IH-35), and WW White Road (IH-410).

Some arterials can be used as alternate routes when incidents result in the closure of interchanges between major freeways. These arterials form a triangle with the two freeways involved. Some such arterials include: Vance Jackson (IH-10 and IH-410), West Avenue (IH-10 and IH-410), Nacogdoches (IH-410 and US 281), Pearsall/Military Drive (IH-410 and IH-35), Zarzamora (IH-35 and US 90), and Culebra Road/Bandera Road (IH-10 and IH-410).

Some major arterials parallel to freeways can also be used as alternate routes although the arterial does not form a "D" or a triangle. These arterials are usually major arterials. Such arterials include: Wurzbach Avenue (IH-410 North Loop), Commerce (US 90), and Presa (IH-37).

Frontage roads, which are parallel to freeways, are an obvious and easy choice for alternate routes due to their proximity to the freeway. In most cases conditions on the frontage road are visible from the freeway and vice versa, and even unfamiliar motorists often use them as alternate routes without any external guidance through CMS's. The signal timing at interchanges, however, is generally not designed for this diverted traffic and may result in congestion on the frontage road as well. Implementation of appropriate signal timing plans to facilitate movement of main lane traffic on the frontage roads during incidents may alleviate congestion without diversion to another arterials.

STUDY SITE

Among the several potential alternate routes in San Antonio identified above, the IH-10/Fredericksburg corridor has been selected as a candidate site for this study. The selection of the corridor was influenced by the existing infrastructure and the recurring congestion experienced in the corridor during the A.M. and P.M. peak hours. The current extent of the ITS system permits the use of freeway changeable message signs (CMS) to convey diversion information to motorists in the outbound direction of IH-10 freeway. Therefore, the focus of this study is the IH-10 outbound traffic. Due to the recurring congestion in the corridor it has been decided to investigate the P.M. peak hour congestion.

Fredericksburg Road is maintained by the city of San Antonio and is part of the city's computerized system. The city mainly uses Type 170 controllers at most signalized intersections, including those on Frederickburg. These controllers are capable of being monitored and operated through modems and telephone lines. The presence of the city personnel at TransGuide Center now further facilitates the use of the arterial as an alternative to IH-10 outbound traffic during incident induced congestion.

STUDY SCOPE

This study investigates the feasibility of using integrated traffic management strategies for relieving recurring and non-recurring congestion in the IH-10/Frederickburg corridor. Although the study corridor was used as a test site for the application of the methodologies developed in this study, the methodologies are applicable to any arterial or frontage road.

Prior to diverting traffic from a freeway to an alternate arterial route, it is essential to know if enough excess capacity is available on the arterial to accommodate the additional traffic. In the absence of any excess capacity, diversion of additional traffic may aggravate the congestion problem by spreading it to other parts of the network. This study investigates methods to estimate the additional capacity available at signalized intersections and interchanges in the direction of the diverted traffic.

The methodology for estimating the excess capacity was applied to the study corridor to estimate the excess capacity available on Fredericksburg Road and the IH-10 outbound frontage road. This study estimates the excess capacity for the outbound direction for different times of day.

A simulation study, using INTEGRATION, investigated the feasibility of diversion of outbound traffic to Fredericksburg Road and the frontage road to alleviate the recurring P.M. peak period congestion on the freeway. Arterial signal timing strategies that yield the maximum amount of excess capacity for the outbound traffic were tested using the simulation.

Finally, this study also investigates the use of arterial changeable message signs (CMS) for corridor traffic management. Although CMS's are being extensively used on freeways to convey information on downstream conditions to motorists, there has been a limited use of CMS's on arterials. An investigation of their past use and guidelines for placement has been studied as part of this study.

ORGANIZATION OF THE REPORT

This report is divided into six chapters. Following this introduction, Chapter II outlines the traffic theory behind the estimation of excess capacity at a signalized intersections or diamond interchanges. Chapter III describes the findings of an analysis of the excess capacity available in the study corridor for the peak and off-peak periods. Chapter IV describes the simulation study and its results and findings. This is followed by a discussion of the use of changeable message signs on arterials and the guidelines for their use and placement in Chapter V. Finally, the conclusions and recommendations are presented in Chapter VI.

CHAPTER II.
ARTERIAL CAPACITY FOR TRAFFIC DIVERSION

Diversion of freeway traffic to an alternate arterial route is beneficial only when it is established that the alternate route has sufficient additional capacity in the direction of the diverted traffic. Generally, the signalization schemes implemented on arterial streets are designed to accommodate the existing traffic volumes. Small variations in traffic are accommodated by these schemes. However, in a diversion scenario, large increases in traffic are expected in a particular direction, and the existing timing may not be appropriate. Hence, new signalization schemes need to be implemented to accommodate the excess diverted traffic through the arterial, if the diverted traffic is expected to be heavy.

Before any diversion strategies can be implemented, it is essential to analyze the existing traffic patterns to determine if additional traffic demands can be satisfied through appropriate signal timing. Traffic detection systems monitor traffic conditions on the arterial. The conditions on the street are a function of the current signal timing on the arterial. An inappropriate signal timing scheme for a traffic situation can lead to severe queues and congestion. While a detection system would detect a slow traffic situation, it may be possible to implement a better signal timing scheme for the same traffic conditions and achieve better throughput from the system.

Although detection systems cannot be used to determine excess capacity for diversion, they are very useful in monitoring alternate arterial routes for incidents. Before diverting traffic from a freeway, it is important to ensure that the alternate route is incident free.

In attempting to estimate the available capacity, it is not sufficient to know the existing conditions on the arterial. The existing conditions can only indicate the quality of the signal timing scheme currently in use, and not necessarily the actual capacity of the arterial.

It is important to know the maximum throughput that can be achieved at an intersection (arterial) in order to estimate the available capacity. It should be noted that there is an upper limit on the throughput that can be achieved irrespective of the signal control scheme. This upper limit is imposed by the geometry of the arterial and the characteristics of traffic flow. Even in freeway systems it is not possible to achieve unlimited throughput although there is no external control imposed on the traffic.

In order to evaluate potential diversion routes for excess capacity available, it is essential to know the maximum reasonable amount of traffic demand that a signalized intersection can satisfy without causing oversaturation and requiring unduly long cycles. The following sections investigate the relationship between intersection traffic demand and signal timing to determine the demand levels at which the required signal timing ceases to be practical and implementable. Determining this higher limit for an intersection will enable the traffic engineer to evaluate the additional traffic flow that an intersection may satisfy with appropriate signal timing.

ARTERIAL CAPACITY AND SIGNALIZATION

The capacity at intersections is defined as the maximum rate of flow (for the subject approach) which may pass through the intersection under prevailing traffic, roadway, and signalization conditions (*2*). Capacity at signalized intersections is based on saturation flow rates and available green times. Saturation flow is defined as the maximum rate of flow that can pass through a given intersection approach or lane group under prevailing traffic and roadway conditions, assuming that the approach or lane group had 100 percent of real time available as effective green time.

Arterial capacity in any particular direction of interest is a function of several variables including signal control schemes implemented, intersection geometry, driver characteristics etc. The geometry, driver characteristics, and the existing or future traffic volumes are generally known to the traffic engineer. The signal timing scheme is the main factor that is available to the traffic engineer for adjustment to create excess arterial capacity for diverted traffic.

In order to evaluate an arterial for its capacity to accommodate additional diverted traffic from else where in the network, it is important to have an understanding of the impact of arterial signal timing on capacity.

The following paragraphs explore the relationship between arterial capacity and signal control strategies. First the concept of intersection critical flow ratio is discussed. Then the relationship between signalization and loss time is investigated. The theory behind the estimation of cycle lengths, including minimum cycle and Webster's Minimum Delay Cycle is investigated. The objective of this investigation is to determine the maximum amount of traffic that a signalized intersection can satisfy with reasonable cycle lengths.

Intersection Critical Flow Ratio

The intersection critical flow ratio (Y_c) is a measure of the intersection demand levels and the physical capacity available to satisfy the demands. Measuring intersection demand levels purely in terms of vehicular flow rate is misleading. For instance, a two lane approach can serve twice as many vehicles as a one lane approach for the same amount of green time. Hence, green time is generally distributed in proportion to each movement's flow ratio (v/s). Therefore, in the following discussion and analysis, the intersection critical flow ratio has been used as a measure of intersection demand levels.

The intersection critical flow ratio is the sum of the flow ratios of all the critical lane groups. The lane group with the highest flow ratio among all the lane groups served by a signal phase is called the critical lane group for the phase. It should be noted that each signal phase serves exactly one critical lane group. Other non-conflicting lane groups may also be served with each critical lane group, but the phase duration required to serve the other phase groups is shorter than the critical lane group requirement.

Signalization and Loss Time

An intersection signalization scheme determines the allocation of the 3600 seconds in an hour to lost time and to productive green time. Further, it also determines the distribution of the productive green time between the various conflicting streams of traffic intending to pass through the intersection. Signalization always results in a certain amount of unproductive time or loss time per hour. Additional capacity can be created at a signalized intersection by reducing the amount of loss time.

One of the primary signalization parameters that determine the distribution of time between productive green time and loss time is the cycle length. Longer cycles result in fewer cycles per hour and hence in lesser loss time per hour. However, it is inconceivable to have a signalization scheme which results in absolutely no loss time.

The hourly loss time at a signalized intersection depends upon the number of phases per cycle and the number of cycles per hour. Each phase serving a critical lane group or movement(s) results in a certain amount of loss time. This loss time, consisting of startup and clearance loss times, depends on the signal settings, local conditions and driver characteristics. Usually the loss time per phase ranges from 3 to 5 seconds.

The number of phases required at an intersection is dictated by the volumes of various conflicting movements and the geometry of the intersection. Separate green indications for left turning traffic usually result in additional phases and loss time. Once the number of phases is determined, however, the loss time per cycle is fixed. It does not depend on the length of the cycle. Whenever the traffic and geometric conditions permit, the number of phases should be reduced to reduce the loss time per hour.

As the loss time per cycle is fixed for any given phasing scheme, reducing the number of cycles per hour reduces the overall loss time per hour and increases the productive time per hour. The number of cycles per hour depends on the length of the cycle. Longer cycles result in fewer cycles per hour and hence lesser loss time. Several factors limit the length of the cycle. These include:

1. Long cycles lead to long periods of red for some approaches; hence there are higher delays.

2. The long periods of red for certain movements result in storage problems. If the approaches do not have enough capacity to store the vehicles arriving during the long red periods (due to closely spaced intersections), queues may spill back into upstream intersections and lead to grid-lock. This problem is particularly critical at diamond interchanges.

3. Drivers may become impatient with the long red indications which may lead to accidents.

Based on these considerations and others to be discussed below, it is necessary to identify a maximum acceptable cycle length for an arterial intersection.

Cycle Length

Intersections critical flow ratio and the loss time per cycle determines the absolute minimum cycle length. The absolute minimum cycle length determines the maximum amount of time that can be lost per hour and still be able to process the existing demand. This cycle results in a v/c ratio, often referred to as the degree of saturation, of 1 for the intersection. The degree of saturation is a measure of the sufficiency of the capacity provided by the physical design and signal timing (*3*). A degree of saturation over 1 implies that the demand exceeds the capacity provided by the signal timing at the intersection.

The minimum cycle is given by the following relationship:

$$C_{\min} = \frac{L}{1-Y_c}$$

Where,

C_{min}	=	Minimum cycle length in seconds.
L	=	Total loss time in seconds (a function of the number of phases)
Y_c	=	Sum of intersection critical flow ratios = $\sum (v/s)_i$
v_i	=	Actual or projected flow rate for the critical lane group or approach i
s_i	=	Saturation flow rate for the lane group or approach

A minimum cycle is generally not provided. Traffic flow is random and varies from cycle to cycle. Although the hourly volumes may be fairly uniform from day to day, flow rates usually vary from cycle to cycle. The timing plan cannot handle individual cycle flow rates higher than the average hourly flow rates and oversaturation occurs.

In order to overcome this problem, traffic engineers usually provide a cycle length that is greater than the minimum cycle length discussed above. It should be noted that any cycle greater than the minimum cycle results in a degree of saturation less than one. Although any cycle length over the minimum cycle results in a degree of saturation less than 1, long cycles tend to increase the delay. Webster derived an expression for the optimum cycle length to minimize delay. This cycle length is generally referred to as Webster's Minimum Delay Cycle (*4*).

Webster's Minimum Delay Cycle

Webster's Minimum Delay Cycle provides the optimum cycle length to minimize delay. The minimum delay cycle is computed based on the following relation.

$$C_0 = \frac{1.5L + 5}{1 - Y_c}$$

Where,

C_0 = optimum cycle length (sec)

L = Total loss time in seconds

Y_c = sum of the intersection flow ratios as described above

The underlying intersection degree of saturation (denoted by X) can be computed from the following relation:

$$X = \frac{Y_c}{(C_0 - L)/C_0}$$

As C_0 is a function of the sum of critical flow ratios and loss time as shown in the relation above, this relation can also be expressed as:

$$X = \frac{Y_c(1.5L + 5)}{L(Y_c + 0.5) + 5}$$

These two relationships are depicted in Figures II-1 and II-2. The figures depict the relationship for loss times of 8, 12, and 16 seconds. These loss times are based on a 4 seconds/phase loss time and two, three, and four phases respectively.

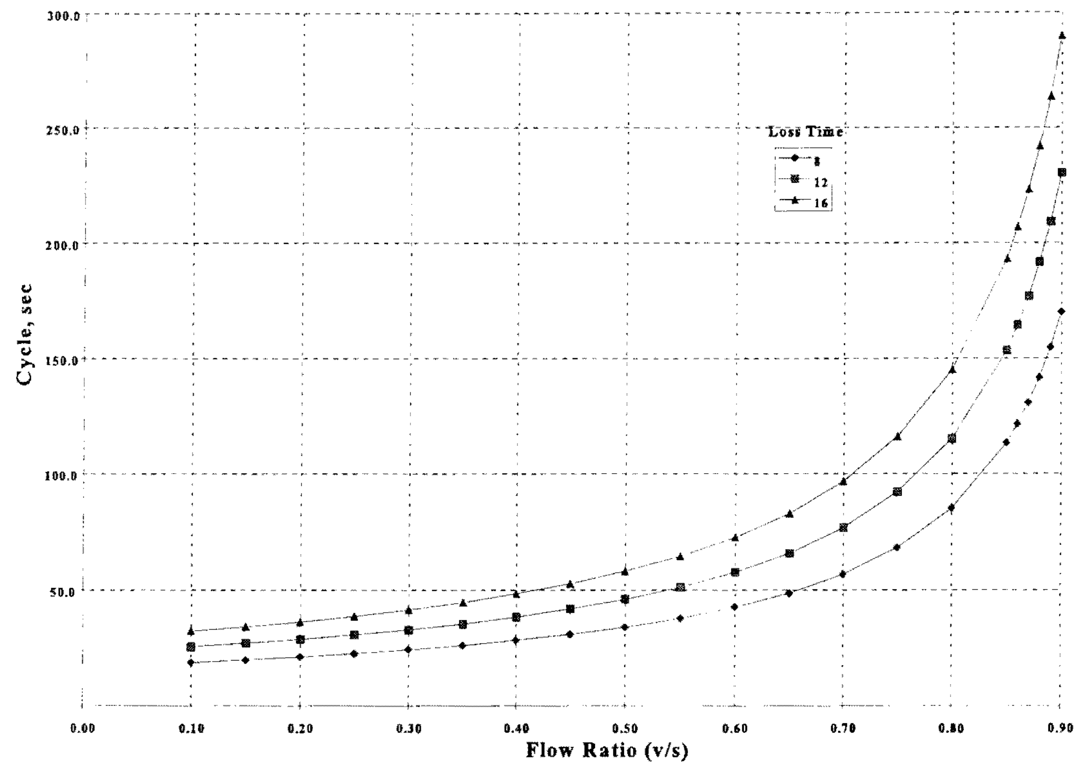

FIGURE II-1. Relationship between intersection critical flow ratio and cycle length

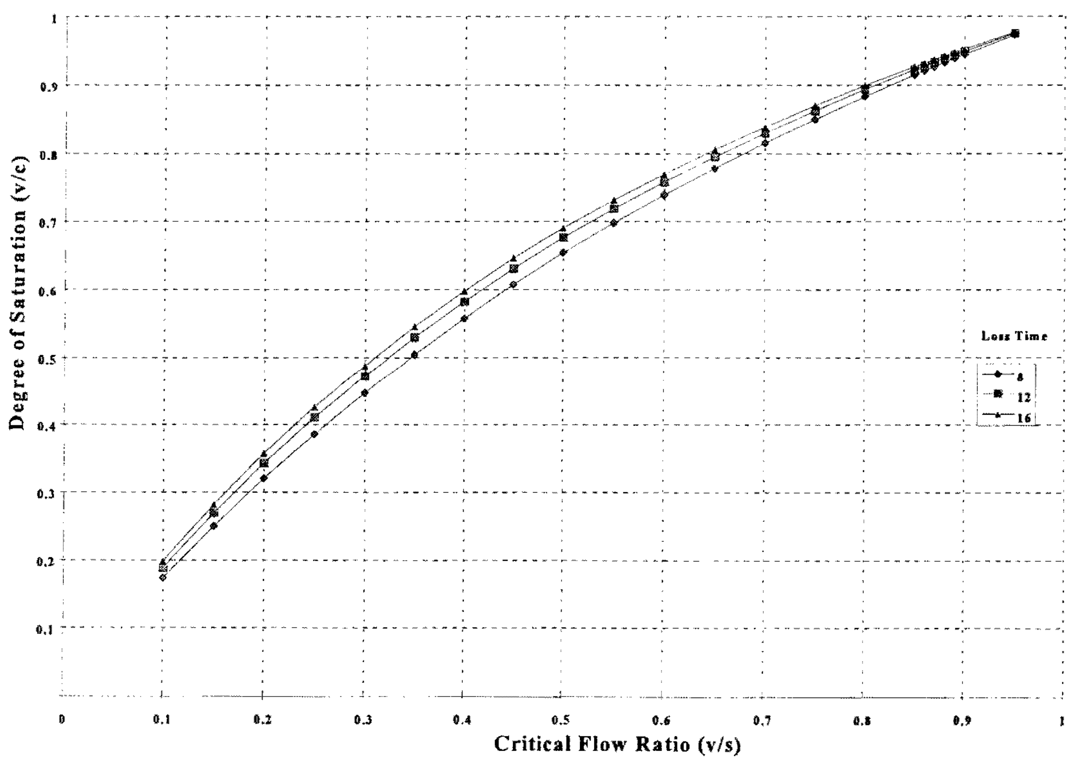

FIGURE II-2. Relationship between intersection critical flow ratio and degree of saturation.

Figure II-1 depicts the relationship between the critical flow ratio and the minimum delay cycle. It can be seen that the rate of increase in the minimum delay cycle length increases rapidly as the intersection critical flow ratio approaches 1. This implies that as the intersection critical flow ratio approaches 1, the required increase in cycle length for a small increase in capacity is very high.

To illustrate this, consider an intersection with a 2-lane approach that is one of the critical lane groups. Assume that the saturation flow rate for the approach is 3600 vph. The minimum delay cycle required for an intersection critical ratio of 0.85, and 4 phases (16 sec loss time) is about 193 seconds. An additional flow rate of 72 vph on this approach would cause the intersection critical ratio to go up by 0.02 (72/3600) to 0.87. The corresponding increase in minimum delay cycle is about 30 seconds. This is a considerable increase in cycle length (hence the delays for other approaches) for a marginal increase in capacity.

It can be seen from Figure II-2 that Webster's Minimum Delay Cycle results in a degree of saturation much below 1 for low intersection critical flow ratios. This implies that the cycle length provided is greater than the minimum cycle length, and additional green time can be provided for each critical movement over the amount necessary to satisfy the existing demands. At higher levels of traffic demands, however, the degree of saturation approaches 1. This implies that for intersections with high levels of traffic demands (resulting in high intersection critical flow ratios) the minimum delay cycle is in the proximity of the minimum cycle. As mentioned above, a minimum cycle ($X = 1$) is not desirable because of the fluctuations in flow rates from cycle to cycle.

The above discussion and Figures II-1 and II-2 show that, if the traffic volumes at an intersection are such that the critical intersection flow ratio is 0.85, then any further increase in the flow rates for the critical movements would require large increases in cycle lengths to satisfy the demand. Also, at this level the intersection degree of saturation is about 0.93 for a 4 phase signal. It is generally not desirable to have a degree of saturation over 0.95 for signalized intersections.

MAXIMUM CYCLE LENGTH

Based on the discussion above, it can be concluded that the maximum demands that a signalized intersection can satisfy with a reasonable cycle length would yield a critical flow ratio of about 0.85 for a four phase signal. Any higher demands would require large cycles (much more than 3 minutes) and are not likely to satisfy the demands despite a large cycle as indicated by the high degree of saturation at this level. For the purposes of determining the maximum capacity available on a signalized arterial, an intersection critical flow ratio of 0.85 has been selected as the upper limit for a four phase intersection. Intersection critical flow ratios of up to 0.87 and 0.9 can be satisfied for intersections requiring three and two phases respectively. It should be noted that the actual volumes (vph) representing these levels of intersection critical flow ratios depend on the intersection geometry and saturation flow rates.

The cycle length at these maximum demand levels is about 190 seconds. Cycle lengths as high as this have been utilized before with success (*5*). As discussed above larger cycles are not likely to increase the intersection capacity proportionately. However, larger cycles add to the delay considerably. Also, larger cycles may cause motorist frustration and accidents.

CHAPTER III.
ANALYSIS OF IH-10 CORRIDOR

Based on the findings discussed in the previous chapter, researchers performed an analysis of the IH-10 freeway corridor to answer the following questions: 1) how much diverted outbound freeway traffic can Fredericksburg (an arterial parallel to IH-10) and the frontage roads accommodate 2) can any geometric improvements be made to increase the number of vehicles that can be diverted. The analysis was performed for peak and off-peak periods. The study site, analysis methodology, and the findings of the analysis are discussed in this chapter. The arterial and frontage road analysis is discussed separately due to some differences in the analysis.

STUDY AREA

As described previously, a section of the IH-10 freeway corridor in San Antonio was chosen as a potential candidate site for application of integrated traffic management strategies using the existing hardware installed as part of the TransGuide system. This corridor experiences severe congestion during the evening peak hours, and if found feasible, a diversion strategy may alleviate some of this congestion.

Figure III-1 shows the location of the study site. This corridor includes the McDermott Freeway (IH-10) and Fredericksburg - a major north-south arterial parallel to IH-10. The freeway has a continuous frontage road in this section, except at the interchange with IH-410. At IH-410 the McDermott freeway frontage road continues as the IH-410 frontage road. A U-turn would be necessary at Cherry Ridge to continue along the McDermott freeway. The study section extends from Woodlawn in the south to Prue Road in the north encompassing 32 arterial signals and 12 interchanges. Several important arterials including West Avenue, Vance Jackson, Callaghan, Wurzbach, and Huebner Road criss cross the two parallel facilities - IH-10 and Fredericksburg. As mentioned previously, Loop 410 also intersects both facilities.

DATA COLLECTION

Data Requirements

The types of data required for this analysis are the following: geometric data including number of lanes, lane widths, allowable movements from each lane, and intersection spacing; and traffic operations data including turning movement volumes, and speeds.

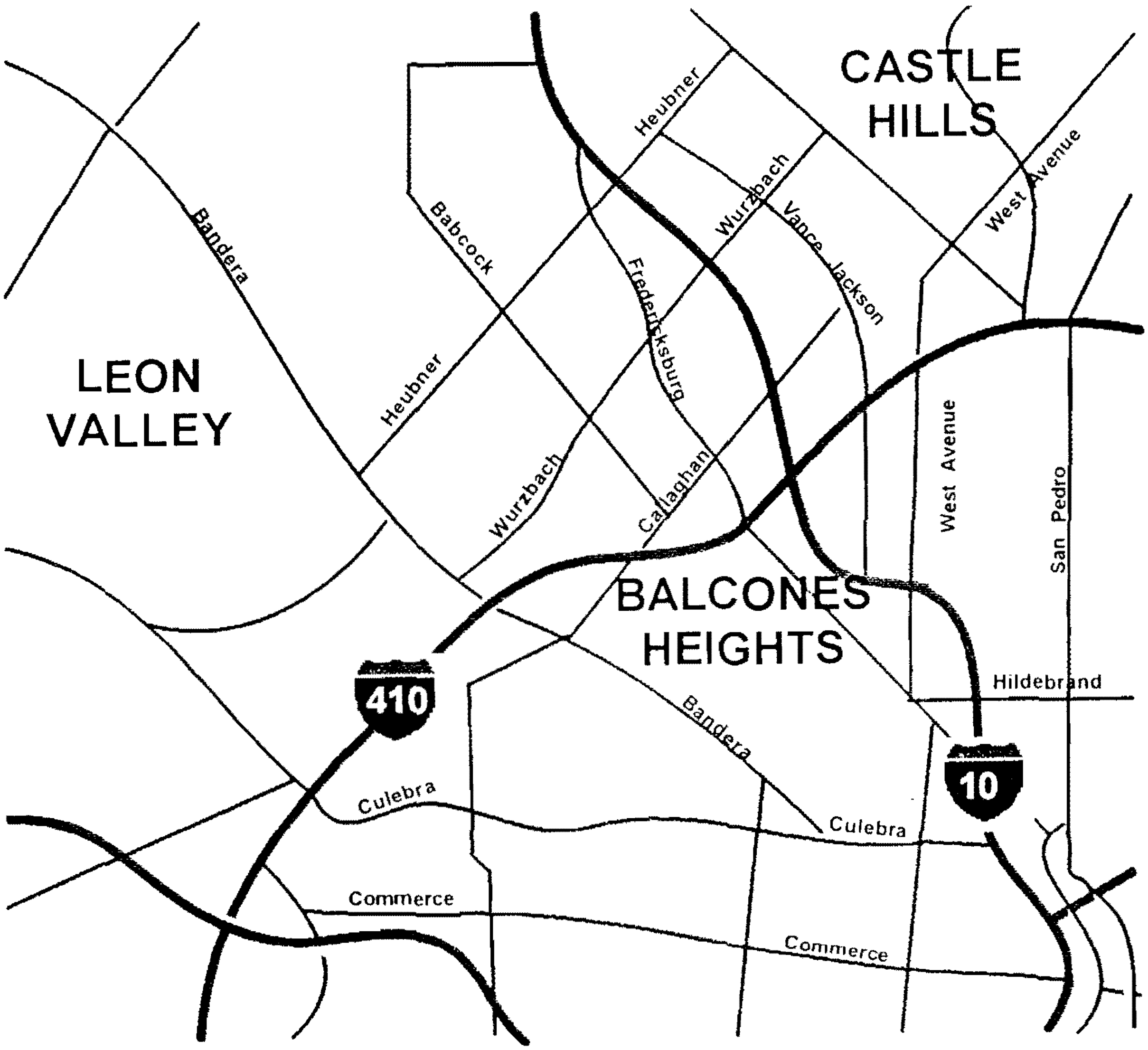

FIGURE III-1. Location of Study Site

Data Acquisition

The district office of the Texas Department of Transportation and the Texas Transportation Institute provided recent and past AM and PM peak hour data for the freeway interchanges. Fifteen minute data was collected for the off-peak period at all the interchanges.

The city of San Antonio contributed traffic volumes for Fredericksburg for peak and off-peak periods. Three intersections along Fredericksburg, namely Balcones Heights, Cross Roads, and Hillcrest are within the jurisdiction of the city of Balcones Heights. These three intersections are not part of the computerized system maintained by the city of San Antonio; hence the data for these intersections was not available through the city. Limited data was collected for these three intersections.

The physical geometry of the network, including the number of lanes, distances, etc. was collected as part of this study. Some of the data was also available from the PASSER II files obtained from the city.

The arterial saturation flow rates were obtained from the PASSER II files obtained from the city. Saturation flow rates for the three intersections mentioned above that were not included in the city's system were obtained using PASSER II.

ARTERIAL ANALYSIS

The methodology adapted for the arterial analysis is discussed first. Then the additional capacity available on the arterial is discussed. Following that, the locations where queue spillback causes potential problems are discussed.

Methodology

Based on the discussion in the previous chapter, an intersection critical flow ratio of 0.85 was assumed to be the maximum demand that can be satisfied at a four-phase intersection with reasonable cycle lengths and splits. The minimum delay cycle for this level of intersection demand would be about 190 seconds assuming 4 phases. Although smaller cycle lengths can be used at intersections with less than four phases, a 190 seconds has been chosen in order to maintain the same cycle length throughout the arterial.

At intersections with fewer phases, the longer cycle can accommodate more demands (a higher intersection critical flow ratio) as discussed below. Table III-1 shows the intersection critical saturation flow ratios and the corresponding underlying degree of saturation for two, three, and four phase signals with a loss time of 4 seconds per phase and a cycle length of 190.

TABLE III-1. Critical Flow Ratio and Degree of Saturation for Minimum Delay Cycle of 190 Seconds

	Four-Phase (16 sec. loss time)	Three-Phase (12 sec. loss time)	Two-Phase (8 sec. loss time)
Critical flow ratio, Y_c	0.85	0.88	0.90
Degree of Saturation, X	0.93	0.94	0.94

As can be seen from the above table, at 190 second minimum delay cycle, intersection critical flow ratios up to 0.9 can be achieved if fewer signal phases are utilized. Fewer phases result in less loss time and hence higher throughput per hour.

Based on the table above, researchers developed a spreadsheet to compute the additional volumes (over the existing volumes) that can be accommodated in the outbound direction on Fredericksburg. The detailed spreadsheet computations are shown in Appendix. As described previously, the intersection critical flow ratio is the sum of all critical movement flow ratios. The additional capacity for the movement serving the diverted traffic is computed using the following equation:

$$AdditionalCapacity(vph) = (Y_t - Y_c) * s_d$$

where,

Y_t = Target intersection critical flow ratio (0.85, 0.88, 0.90 for 2, 3, 4 phases respectively for a 190 second cycle)

Y_c = Intersection critical flow ratio for the demand levels

s_d = saturation flow rate (vph) for the movement serving the diverted traffic.

The same results may be obtained by using PASSER II for arterials. The following procedure may be adapted to evaluate an arterial for its suitability for diversion.

Step 1. Determine the maximum cycle length that is acceptable based on the local conditions. A 190 seconds cycle is recommended for arterial streets based on the analysis discussed earlier in the report. Higher cycle lengths may not proportionately increase the capacity.

Step 2. Use the existing volumes and the maximum cycle length determined in Step 1 to optimize the phase splits and sequence using PASSER II.

Step 3. At each intersection where Step 2 yields a degree of saturation less than 0.95, excess capacity is available. It should be noted that the objectives of providing good level-of-service and creating excess capacity for diverted traffic generally conflict, unless the expected volume of diverted traffic is insignificant. For each intersection where excess capacity is available, increase the volume for the phase serving the diverted traffic until the resultant degree of saturation for the intersection is close to 0.95.

Step 4. The difference between the volumes obtained in the Step 3 and the existing volumes used in Step 2 determines the additional volumes that can be accommodated at the intersection and the particular approach. It should be noted that the additional capacity is different for different movements at the same intersection and for the same time-period.

Step 5. Although the intersection with the least capacity for additional traffic determines how much traffic can be diverted, all excess green time at all intersections should be allocated to the movement serving the diverted traffic in order to ensure minor traffic fluctuations are handled without cycle failure. The timings obtained in Step 5 should be used for diversion scenarios.

Step 6. The long cycle length required for creating additional capacity for diverted traffic may also result in long periods of red for other movements, and some times even for the diverted traffic. Hence, it is essential to ensure that the queues formed during the red periods do not exceed the storage capacity of the link. If the storage capacity is insufficient, then care should be taken to ensure proper coordination for the particular movements. If coordination is not possible, then it may not be possible to have a long cycle length. As mentioned earlier, shorter cycle lengths result in greater amounts of loss time and lesser intersection capacity.

For the purposes of the analysis for additional capacity, researchers divided Fredericksburg into two parts: one to the south of IH-410 and the other to the north of IH-410. The traffic patterns and geometry are different in the two sections. Also, since three of the intersections are under the jurisdiction of the City of Balcones Heights, the city of San Antonio has two separate systems one to the north of IH-410 and the other to the south of IH-410. This division was maintained for the analysis also. However, the three intersections under the City of Balcones Heights were also included in the south section for the analysis.

First, the additional volume that can be added to each intersection so that the target intersection critical flow ratio is achieved is computed. The intersection with the least additional capacity determines the maximum amount of diversion possible on to the arterial. The direction of interest here is the northbound direction, but the methodology can be applied to any direction and any arterial.

The saturation flow rates used were based on the information obtained from the City of San Antonio. The PASSER II files supplied by the city had a different number of intersections for peak and off-peak periods. Since the signal at USAA Gate 2 is not operated during the off-peak hours, it is not included in the off-peak list of intersections. Apart from this, there are other intersections (Gus Eckert, Bluemel, and Mira Mesa) which appear in the PASSER II files for either peak or off-peak periods only. Since these are not critical in any case, they have not been listed in the table. However, with the data available, the excess capacity at these intersections was also computed in order to ascertain they do not constitute critical intersections.

Findings

Table III-2 shows the excess capacity available at each of the intersections in the northern and southern parts of Fredericksburg during the AM, PM, and off-peak periods. The detailed spreadsheets showing the saturation flow rates, volumes, and intersection and critical saturation flow rates are shown in Appendix-A.

AM Peak

It can be seen from the Table III-2 that during AM peak period Wurzbach and USAA Boulevard are the critical intersections north of IH-410. These intersections have heavy cross street and turning movement volumes and do not allow any more green allocation to the northbound direction as during the PM peak period. It should be noted that Medical, Heubner, and Callaghan also have relatively small amounts of excess capacity and may also be problem areas for diversion.

The one lane exit ramp at Woodlawn is the critical intersection during the AM peak for the section south of IH-410. This intersection allows about 900 vph in addition to the existing volumes if appropriate signal timing at 190 second cycle is implemented throughout Fredericksburg Road. It can be seen that all other intersections on Fredericksburg have the additional capacity to accommodate more than 900 vph.

TABLE III-2 . Excess Capacity for Northbound Diversion to Fredericksburg

Intersection	AM Peak	PM Peak	Off-Peak
South of IH-410			
Woodlawn	**910**	884	**1212**
Fredericksburg	1651	1868	2221
Fredericksburg	1737	1941	2146
Buckeye	2299	2424	2661
Fulton/Zaram	1955	2057	2302
Lynwood	2163	2026	2486
West Avenue	1952	1371	2044
Hildebrand	1773	2108	2418
Babcock/Fresno	1745	1316	2540
Vance Jackson	2921	1460	2585
De Chantle	2209	1909	2322
Williamsburg	2340	1847	2295
Balcones Hts.	2013	1379	1971
Crossroads	1854	**549**	1804
Hillcrest	3291	2498	3192
410 East int.	1677	902	1333
410 West int.	2232	803	1513
North of IH-410			
Woodlake	2585	2152	2677
Lakeridge	2737	2297	2910
Magic	3078	2582	3202
Callaghan	446	475	**1666**
Mockingbird	3034	2463	3224
Chamber	2905	2780	3183
Louis Pasteur	2050	1757	2196
Medical	260	0	2124
Data Point	2477	1844	3087
Wurzbach	**139**	**0**	2106
Cinnamon Creek	894	487	2966
USAA Blvd	**0**	1448	3449
USAA G2	2534	2473	N/A
Huebner	432	**70**	2618

PM Peak

The volume of traffic that can be diverted depends on the most restrictive intersection, which is the intersection with the least amount of excess capacity. As can be seen from Table III-2, during the PM peak period, there is no excess capacity in the section of Fredericksburg to the north of the IH-410 interchange. High levels of demand on major cross streets including Huebner, Wurzbach, Medical, and USAA boulevard make it impossible to allocate more green to the northbound approach of Frederickburg to facilitate diversion without starving either the cross streets or turning movements. It should be noted that due to high cross street volumes, starving them (oversaturation) may result in severe congestion throughout the network and possibly grid-lock.

The section of Fredericksburg to the south of IH-410 can accommodate up to 550 more vehicles at 190 seconds cycle. Although this is not sufficient to significantly reduce congestion in the westbound direction on IH-10, it may be used to divert IH-10 traffic exiting at IH-410. This is one of the scenarios tested in simulation and has been discussed in the following chapter.

Off-Peak

Substantial amount of diversion can be made from IH-10 westbound to Fredericksburg during the off-peak period. This can be used for non-recurring congestion mitigation. It can be seen from Table III-2 that about 1200 vph outbound vehicles can be diverted to Fredericksburg. Again, the one-lane exit-ramp from IH-10 at Woodlawn limits the amount of diversion possible. It should be noted that all intersections along Fredericksburg, including those to the north of the IH-410 interchange, have at least 1200 vph additional capacity at the 190 second cycle length and appropriate splits.

The additional capacity available during the off-peak period can be used for diverting outbound traffic on IH-10 in the event of a non-recurring incident on IH-10. However, it may be easier to divert traffic to the frontage road instead of Fredericksburg, if additional capacity can be created on the frontage roads. The following section discusses the additional capacity available on the frontage road for peak and off-peak periods.

Storage Problems

Analysis was performed to determine any storage problems that might result from the large cycle lengths proposed. It was found that queuing is not a problem on Fredericksburg. Among the cross streets, however, West Avenue and Hildebrand may experience some storage problems. These two cross streets intersect with each other, just before their intersection with Fredericksburg road, forming a triangle. Storage problems at these two cross streets may lead to spill back to their mutual intersection. This may be avoided by providing proper coordination between the intersection of West Avenue at Hildebrand and the intersections of West Avenue and Hildebrand with Frederickburg.

FRONTAGE ROAD ANALYSIS

Researchers conducted the analysis of the frontage road for additional capacity on similar lines as the arterial. However, due to the special nature of interchanges and their traffic flow characteristics, it was necessary to use PASSER III to ensure that the interior storage is not a problem while maintaining the degree of saturation below 0.95.

Several phasing strategies are used at diamond interchanges. The basic phase configurations are two-phase, three-phase, and four-phase. The number of basic phases and the method by which one calculates green splits determines two-phase, three-phase, and four-phase control. It should be noted that there is a difference is calculation green splits between different phasing schemes. At other types of intersections, generally, Webster's method is used to determine the phase split for critical movements.

For two-phase and three-phase control schemes, the diamond interchange is treated as two separate intersections, each having either two or three phases respectively. Based on the volumes for different movements, the two intersections are coordinated such that the more critical (higher volume) movement can go through the interchange without stopping in order to avoid interior storage problems.

For four-phase control, the diamond interchange is treated as a single intersection that has four basic phases. The four basic phases are the two arterial phases and the two frontage road or ramp phases (*6*). Protected left-turn phases for the interior movements are provided; however, their duration is dependent upon the two exterior movements that contribute to the volume of the interior. A subset of the four-phase control is the TTI-Lead phasing.

Because four-phase control minimizes the number of vehicles stopping within the interchange, it is generally recommended for isolated diamond interchanges with short interior storage length. Generally, four-phase with overlap control works well for heavy unbalanced ramp traffic (*6*).

When traffic is diverted from the freeway to the frontage road, the diverted traffic goes through at the interchange and does not cause any interior storage problems if they do not already exist due to normal traffic demands and interchange geometry. Therefore, for diversion timing four-phase control is generally not suitable. However, for some traffic conditions it may be more appropriate than a three-phase scheme even for traffic diversion.

Methodology

Considering the fact that in some cases TTI-four phase with overlap control schemes works better, and that the methodology used for determining the splits is different between three-phase and four-phase control, the spreadsheet methodology used for arterial analysis has not been used for the analysis of frontage roads for excess capacity. Instead a methodology similar to the arterial analysis methodology discussed above using PASSER II was used. Instead of PASSER II, the diamond interchange timing program PASSER III was used. It should be noted that if three phase control scheme is used always, the spreadsheet methodology can be used for interchange analysis also as can be seen from the spreadsheets included in Appendix-B.

A 190 second cycle was used for arterials as the maximum cycle length. This cycle could accommodate demand levels resulting in intersection critical flow ratios of 0.85 at a four-phase intersection. For similar demand levels, it can be seen from Figure II-1 that a 150 second cycle would suffice at a 3-phase intersection. At 190 second cycles, as can be seen from Table III-1, higher demands up to intersection critical flow ratios of 0.90 can be accommodated. At diamond interchanges, however, large cycle lengths lead to interior storage problems. TTI-Lead phasing needs to be used to overcome this problem. TTI-Lead phasing, however, allocates large green times to interior movements in order to avoid queue storage in the interior and is not very favorable for the through traffic on the frontage road. Also, PASSER III does not optimize three-phase control for cycle lengths above 150 seconds. In view of all these factors, a 150 second cycle was used with three-phase control, a 150 and 190 seconds cycle was also tested with TTI-Lead phasing, and the phasing that resulted in the maximum additional capacity was chosen. In all cases, except one, a 150 second cycle with three-phase control was found to result in more capacity for through movement than 190 seconds cycle as can be seen from the tables in Appendix-B.

The following procedure can be adapted to estimate the excess capacity available at a diamond interchange for the frontage road through movement.

Step 1. Determine the maximum acceptable cycle length. A 150 second cycle is recommended for diamond interchanges because a) interior storage may be a problem for larger cycle lengths, and b) TTI-Lead phasing may be used with larger cycle lengths, but this phasing scheme allocates large green intervals for the interior movements in order to avoid interior storage problems, and may not result in any additional capacity for the diverted traffic, and c) PASSER III does not optimize internal offsets for three-phase control schemes for cycle lengths over 150 seconds.

Step 2. Use the existing traffic levels, geometry, and lane assignments to determine the optimum phasing and splits for the cycle length determined in step 1. If the storage ratio and the degree of saturation is below 1.0 for all movements on both sides of the interchange, there is excess capacity. It should be noted that even when only one side of the interchange is oversaturated, it may not be possible to divert any traffic to the other side. The movements from the undersaturated side to the oversaturated side will be affected and lead to congestion on both sides.

Step 3. For each interchange, increase the volume for the phase that serves the diverted traffic which is usually the frontage road through movement. The volume should be increased until either the interior storage ratio or degree of saturation exceeds 0.95.

Step 4. The difference between the volumes obtained in Step 3 and those used in Step 2 determines the excess capacity available for that particular movement at the interchange.

Step 5. The interchange with the least additional capacity for excess traffic determines the amount of traffic that can be diverted. In case of diversion to the frontage road, it should be noted that several exit ramps provide the opportunity for motorists on the freeway to use the frontage road. Also, the conditions on the frontage road are generally visible to the motorists on the freeway and no additional information is required. Hence, providing a control scheme that results in maximum capacity for the diverted traffic under the prevailing conditions at each interchange is recommended. The timings obtained in step 4 above provide the maximum amount of green for the diverted traffic and may be used for diversion scenarios. Providing a degree of saturation close to 1.0 ensures that all slack green is allocated to the phase serving the diverted traffic. The timing plan results in a low level-of-service, but as mentioned previously, traffic diversion and level-of-service are conflicting objectives and they cannot both be fulfilled at the same time, unless the volume of existing and/or the diverted traffic is small.

Findings

Table III-3 shows the excess capacity for the westbound direction available at each of the IH-10 interchanges in the study section for different times of day. The methodology discussed above has been used to compute the values shown in the table.

The geometric improvements suggested involve changing the lane assignment at the following interchanges: Fulton, Hildebrand, Fresno, and West Avenue. The existing lane assignment allows for one exclusive left-turn and one shared left-and-through lanes. Based on the volumes, it was found that providing just one exclusive left turn lane (no shared lane for left turns) on the frontage road and the interior increases the excess capacity for the through movement. The same methodology discussed above was used to compute the excess capacity with geometric improvements.

TABLE III-3 . Excess Capacity for Westbound Diversion to Frontage Road

Interchange	Off-Peak (vph)		AM Peak (vph)		PM Peak (vph)	
	Existing	GI*	Existing	GI*	Existing	GI*
Woodlawn	1000	1000	750	750	600	600
Fredericksburg	950	1025	950	900	1000	1125
Fulton	1225	1225	1000	1025	1050	1075
Hildebrand	1150	1475	350	990	225	700
Fresno	1800	2025	1250	1575	775	1200
West Avenue	950	1350	825	1250	0	450
Vance Jackson	1025	1200	425	675	115	0
Crossroads	1920	2145	425	0	625	0
Callaghan	1950	1950	0	0	1150	1150
Wurzbach	500	500	850	850	550	550
Huebner	1525	1525	0	0	400	400

*GI= Geometric Improvements

AM Peak

Two interchanges to the north of IH-410, namely Callaghan and Huebner do not have any additional capacity for the outbound through movement on the frontage road. During the AM peak, the inbound direction is the peak direction. Due to heavy left turning volumes from the westbound frontage road and also due to heavy traffic on the eastbound frontage road, the left side is oversaturated at these interchanges. Although the westbound traffic does not use the left side, the heavy left turning movement from the right-side frontage road is likely to be affected and hence excess capacity for diversion may not be available. Based on the existing traffic patterns, a lane reassignment at the three interchanges to the north of IH-410, namely, Callaghan, Wurzbach, and Huebner is not likely to generate any additional capacity for the diverted traffic.

Among the interchanges to the south of IH-410, Hildebrand, Vance Jackson, and Crossroads offer the least additional capacity for diverted traffic. Hildebrand has only 350 vph additional capacity in the outbound direction, while Vance Jackson and Crossroads have slightly higher amount at 425 vph. This is not likely to be sufficient in case of a major incident on the freeway.

The proposed geometric improvements in terms of changing the existing lane assignment as mentioned above may result in some additional capacity for the through movement, as can be seen from the Table III-3. The lane reassignment will be beneficial at Fulton, Hildebrand, Fresno, and West Avenue. The improvements do not yield similar benefits at Vance Jackson and Crossroads interchanges.

PM Peak

During the PM peak it can be seen from the table that excess capacity is available at all interchanges to the north of IH-410. Huebner has the least additional capacity of 400 vph and restricts the amount of diversion from the freeway during the PM peak hour.

Among the interchanges to the south of IH-410, West Avenue does not offer any additional capacity for diverted traffic without geometric improvements. Hildebrand and Vance Jackson also severely restrict the amount of traffic that can be diverted. Woodlawn and Fredericksburg are not recommended for Frontage Road diversion because the frontage road reduces to a one-lane section between Frederickburg and Fulton and is likely to be a bottleneck area. Although Fresno and Crossroads have larger amounts of capacity for additional westbound frontage road traffic, it cannot be used because of the lack of capacity at the above mentioned interchanges.

Off-Peak

During the off-peak period, Wurzbach has the least capacity (500 vph) for additional traffic in the westbound direction. In fact, this is the least for the entire study section in the westbound direction for the off-peak period. As can be seen from Table III-3, much higher capacity can be made available at the other two interchanges in the study section to the north of IH-410. About 950 outbound vehicles can be diverted to the section of the frontage south of IH-410. West Avenue has the least additional capacity.

As can be seen from Table III-3 and the above discussion, the greatest amount of additional capacity is available during the off-peak period. Although large volumes can be accommodated at most interchanges in the outbound direction, one or two interchanges limit the amount that can go through the section.

Geometric Improvements

In order to increase the capacity for additional diverted traffic, geometric improvements as mentioned earlier were also analyzed. Based on the existing traffic conditions, improvements are feasible only at interchanges to the south of IH-410. Geometric improvements can yield benefits in terms of additional capacity for the outbound traffic at

Hildebrand, Fresno, and West Avenue mainly. At other interchanges, the improvements are not beneficial.

CONCLUSIONS

Based on the discussion in the previous chapter, researchers developed a methodology for estimating the available capacity for traffic diversion in a particular direction for arterial streets as well as frontage roads. A maximum cycle length of 190 seconds was used for estimating the excess capacity available for diverted outbound traffic on Fredericksburg Road and the IH-10 westbound frontage road for different times of day.

The analysis revealed that diversion is possible on Frederickburg during the off-peak period. During the AM and PM peak periods, however, it was found that the section of Frederickburg to the north of IH-410 does not have any additional capacity for diversion. The section to the south of IH-410 has additional capacity, although not as much as during the off-peak period, during the AM and PM peak periods also. Link storage was not found to be a problem on Fredericksburg. However, West Avenue and Hildebrand may experience some queuing and spillback problems.

The frontage road can also accommodate additional westbound traffic during the off-peak period. Wurzbach was found to be the critical interchange for the off-peak period, with additional capacity for only 500 vph in the outbound direction. The interchanges to the south of IH-410 all have additional capacity during the off-peak period. During the AM peak period, diversion may not be possible through interchanges to the north of IH-410. Those to the south of IH-410 can accommodate some additional diverted traffic at a 150 second cycle. During the PM peak period, again, the available capacity is very limited.

Geometric improvements in the form of lane reassignment at Hildebrand, Fresno, and West Avenue may result in increased amounts of excess capacity for the diverted traffic. Based on the existing traffic conditions, similar improvements at other interchanges do not yield any benefits.

One of the main problems faced was the use of saturation flow rates computed by PASSER III. PASSER III estimates saturation flow rates based on lane-group analysis. For example, when the left turning movement has an exclusive left turn lane and a shared lane with the through movement, PASSER III assigns a saturation flow rate that is less than one-lane saturation flow rate based on the volumes. This is unrealistic. It is particularly of concern because in this analysis the through movement volumes are increased in order to determine the excess capacity, and more capacity than is actually available is allocated to the frontage road through movement.

CHAPTER IV.
SIMULATION STUDY

Researchers conducted a simulation study in order to determine the amount of diversion that can actually be achieved from a congested freeway using the timing plans discussed in the Chapter III. PM peak conditions were studied in order to study the possibilities for diversion during the recurring PM peak hour congestion in the IH-10 corridor. The simulation study and the results and findings are discussed in this chapter.

STUDY AREA

The study corridor for the simulation included small sections of Interstate-10, Fredericksburg Road, and the system of roadways connecting the freeway and Fredericksburg Road in San Antonio, Texas (Figure III-1). Fredericksburg Road includes thirty signalized intersections and two interchanges (IH-10 and IH-410). The section of IH-10 within the study area includes nine signalized interchanges, including the interchange on Fredericksburg Road, and one stop sign controlled interchange.

STUDY OBJECTIVE

The study objectives were to: 1) assess the use of alternate routes for diversion through simulation methods, and 2) compare and substantiate the results of the analysis for excess capacity discussed in Chapter III.

Several alternate signal timing strategies, based on the discussions in the previous chapter, were developed to study the actual diversion and capacity that would result. The alternate routes were made more attractive by optimizing signals and increasing main street green, resulting in increasing bandwidth, improving travel time, and increasing capacity on the alternate route. The simulation methodology, strategies tested, and the results are presented in the following sections.

MODELS FOR CORRIDOR SIMULATION

Several simulation models exist with a wide variety of capabilities. Most simulation models are capable of either simulating a freeway or arterial network. Corridor simulation models are like other simulation models, but they are capable of simulating networks consisting of freeways and signalized arterials in an integrated network. Corridor simulation models generally also have a traffic assignment capability (*1*).

Several corridor simulation models are available currently and much research is going on to develop models capable of evaluating advanced traffic management systems and advanced traveler information systems. Among the currently available models, INTEGRATION and CORFLO are widely used in the United States. Other models in use mainly in Europe include CONTRAM and SATURN. The CORFLO and INTEGRATION models are briefly discussed in the following paragraphs.

The CORFLO model is a part of the TRAF family of models developed by the Federal Highway Administration. It consists of a macroscopic arterial simulation model (NETFLO), a macroscopic freeway simulation model (FREFLO), and an equilibrium traffic assignment model (TRAFFIC) (*7*).

In order to estimate the amount of diversion that different signal timing strategies could produce, it is necessary to have a simulation model that is capable of reassigning traffic based on the current travel times and speeds in the network. Past studies on traffic diversion using CORFLO involved an external specification of the amount of diversion and the diversion route (*5*). This is one of the drawbacks of the model. Instead a model that is capable of dynamically assigning traffic in response to emergent traffic conditions during the simulation is required for assessing diversion scenarios. INTEGRATION models meets this requirement for dynamic assignment.

The INTEGRATION model is a microscopic simulation model developed by the Transportation Systems Research Group of Queen's University, Canada. It is one of the few models available today which was specifically developed for IVHS applications. The model combines traffic flow simulation and traffic assignment functions. The approach is fully dynamic, as routes, flows, demands, and controls are continuously updated. The behavior of traffic flow is considered in terms of individual vehicles that have self-assignment capabilities (*8*).

INTEGRATION is a fully microscopic model, as it tracks the lateral as well as longitudinal movements of individual vehicles at a resolution of up to one deci-second. This microscopic approach permits the analysis of many dynamic traffic phenomena, such as shockwaves, gap acceptance, and weaving, that are usually very difficult or infeasible to capture under non-steady state conditions using a macroscopic rate-based model (*9*).

The microscopic approach also permits variations in traffic conditions. INTEGRATION permits the density of traffic to vary continuously along the link. The model can consider virtually continuous time varying traffic demands, routings, link capacities, and traffic controls, without the need to predefine an explicit common time-slice duration between these processes (*9*).

One of the objectives of the simulation was to analyze if the alternate routes were beneficial for diversion. Thus if the freeway was congested and the alternate routes were beneficial, INTEGRATION would automatically divert traffic due to its dynamic nature. Hence, due to the dynamic nature of INTEGRATION, it was considered as a better option for this type of analysis.

BUILDING THE NETWORK

The fundamental model input requirements to simulate a network using INTEGRATION are the node characteristics, link characteristics, signal timing plans, and the origin-destination traffic demands. A link-node network representation was developed to understand the network and data requirement. The network contains a total of 342 nodes, 539 links, and 50 signals. Figure IV-1 illustrates the link-node representation of the network.

The data required for specifying the node and link characteristics, such as link length, number of lanes, type of control, etc., were obtained from the department plans and field data collection.

Traffic volume data is required for generating signal timing for the arterials and frontage roads. Some of the signal timing data for intersections on Fredericksburg Road was available in the PASSER-II files provided by the City of San Antonio. The data that was not available in the PASSER-II files was collected through field studies.

Traffic volumes (peak periods) and geometry at interchanges in the study corridor were available through the Texas Department of Transportation. Off-peak volumes that were not available were collected through field studies. Since only volumes were available, PASSER-III was used to determine the optimum signal timing plans for the interchanges.

Apart from node, link, and control information, INTEGRATION also requires information on network traffic volumes for simulation. This information is provided in the form of an origin-destination table for the study area. Some simulation models that are not capable of dynamic traffic assignment require link volumes as input. INTEGRATION, however, requires origin-destination information because it dynamically assigns traffic to the links in the network based on the existing traffic conditions in the network. Since only link and turning movement data was available for the study area, it was necessary to generate a synthetic origin-destination table for the network. A proper origin-destination table is crucial to the success of any simulation study using INTEGRATION. The origin-destination table represents the existing traffic conditions in the network. Generating the synthetic origin-destination data is a complex and iterative process and is described in the following section.

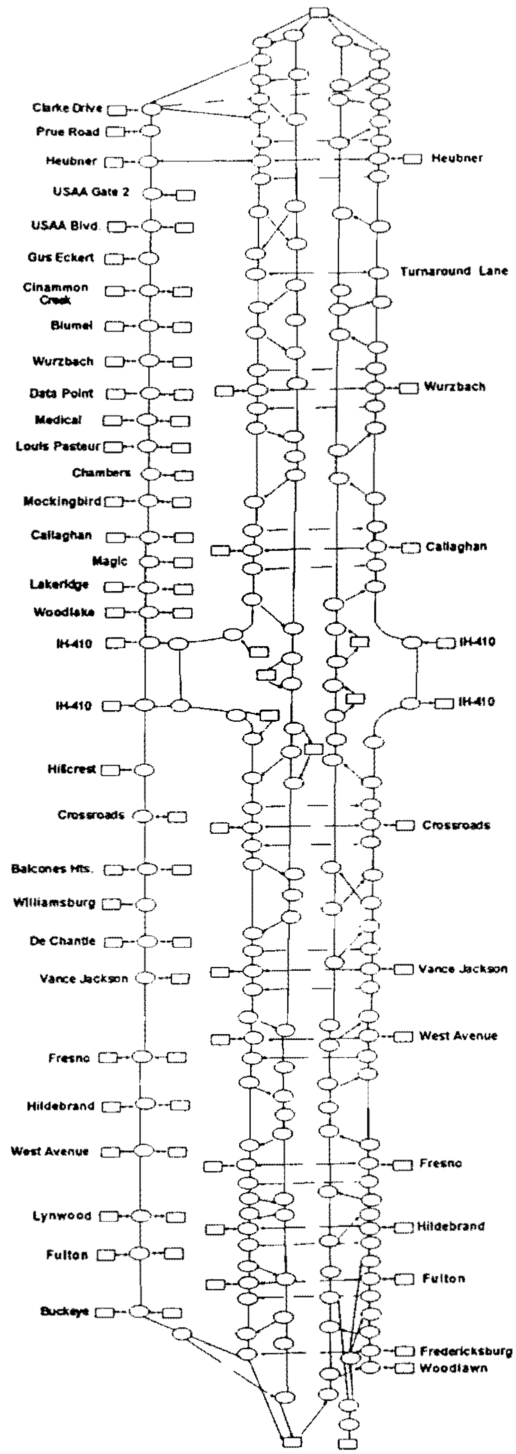

FIGURE IV-1. Link-Node Representation of the Network

ESTIMATING ORIGIN-DESTINATION TRAFFIC DEMANDS

The software program QUEENSOD generated the synthetic origin-destination (OD) data. The Canadian developers of INTEGRATION at Queens' University also developed the program. QUEENSOD works in conjunction with INTEGRATION sharing the same network data files. QUEENSOD generates a synthetic OD table based on the observed link volumes.

Apart from the link and node characteristics, QUEENSOD requires information on the minimum paths utilized by traffic between various OD pairs and the observed link traffic flows for the network to be able to estimate static OD traffic demands. The peak hour volumes for the links on Fredericksburg were calculated from the PASSER-II files provided by the City of San Antonio. For the freeway links, peak hour volumes were estimated from planning models and traffic counts on frontage roads.

The minimum path trees required by QUEENSOD were obtained from INTEGRATION output. An initial minimum path tree was obtained from INTEGRATION by loading the network with a single OD pair. A seed origin-destination demand file was also prepared in order to minimize errors between the resulting link flows and observed link flows. The seed demand matrix is an important data input to QUEENSOD model as it provides a starting point for the solution search. As mentioned earlier, this is an iterative process. QUEENSOD requires minimum path trees from the simulation and the simulation requires origin-destination information from QUEENSOD.

Using the minimum path trees and the seed OD information, QUEENSOD provided the estimated OD demands in a format compatible with INTEGRATION. The estimated OD demands were used in INTEGRATION to obtain trees again which could be different from the trees used in the earlier iteration. It should be noted that several minimum path trees are utilized within INTEGRATION during the simulation period because minimum path trees for each OD pair are computed at a user specified interval. At any given time the simulation uses up to five different minimum paths between each OD pair. As the network loading changes due to changes in the OD trip data, new paths may be used between the same OD pair. These new paths may in-turn result in a different OD solution from QUEENSOD.

The new path trees from INTEGRATION were used along with the link volumes, seed OD, and other input files to estimate new OD demand. This iteration was repeated several times to achieve an acceptable OD table.

One of the problems faced during this process was that INTEGRATION assigns traffic dynamically; therefore, paths changed every time the OD changed. Static OD demands were estimated in QUEENSOD because dynamic volumes were not available. Hence, selection of the appropriate tree is essential to minimize the errors between link flows observed in INTEGRATION, link flows estimated by QUEENSOD, and actual link flows from the field. The process discussed above is illustrated in Figure IV-2..

The iterative process depicted in Figure IV-2 could not be continued until an OD table that results in simulated link flow rates comparable to the observed link flow rates was obtained. The OD table used for evaluating the scenarios resulted in about a thirty to forty percent difference between the observed and simulated link flows. On some links the difference was higher. This level of accuracy may not be sufficient for studying traffic diversion. However, due to time constraints, it was decided to use the OD table to study some scenarios nevertheless.

DESCRIPTION OF SCENARIOS

A number of scenarios were created to achieve the objectives of the study discussed earlier. The description and development of these scenarios are discussed in this section. Table IV-1 gives a short description all the scenarios created.

Scenario 1 (Base Case Scenario)

Scenario 1 represents the existing conditions in the study area. As mentioned earlier, the simulation experiment was set up to study freeway traffic diversion to Fredericksburg Road and the IH-10 westbound frontage road during the PM peak traffic conditions. This scenario is the base against which traffic flows and speeds resulting from control strategies represented in other scenarios are compared.

The simulation duration was divided into three time periods. At the beginning of simulation, a 15 minute period of off-peak loading was used. The demand for this initial period was assumed to be 40 percent of the peak hour demands. Following this initialization period, the peak hour traffic was loaded for one hour. The OD table estimated as discussed in the previous section was used for the peak one hour. A third period of 15 minutes was used for clearing the network and for the completion of peak hour trips en route. Traffic demands during the third time period were also assumed to be 40 percent of the peak hour demands.

PASSER II and PASSER III were used to obtain the signal timing plans for signalized intersections and interchanges. On Fredericksburg Road, all intersections south of the IH-410 interchange were coordinated at a cycle length of 100 seconds. Signal splits for all intersections north of the IH-410 interchange, including the interchange, were optimized for a cycle length of 120 seconds. The phasing sequences used were the same as those used by the City of San Antonio. A total of 30 signalized intersections and 2 interchanges exist on Fredericksburg Road. All the interchanges on the frontage road of IH-10 were optimized for the minimum delay cycle. TTI-Lead phasing was used for all the interchanges.

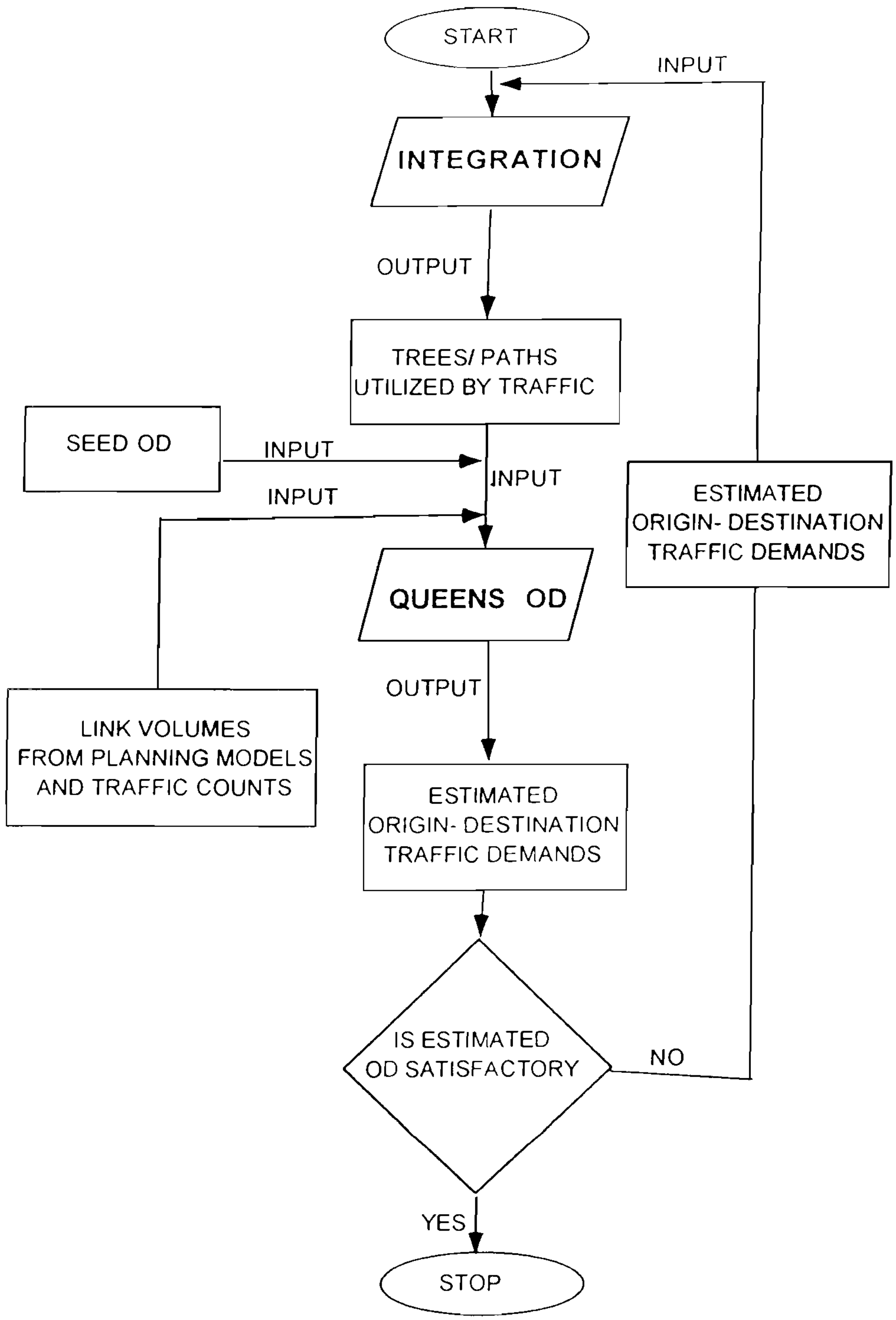

FIGURE IV-2. Process Used for Obtaining Origin-Destination Traffic Data

TABLE IV-1. Description of Scenarios

Scenario Number	Description of Scenario	Objective of the Scenario
Scenario 1	Existing conditions (Base Case Scenario), All interchanges isolated.	To simulate existing conditions
Scenario 2	Existing conditions + Interchanges coordinated	To analyze the impact of frontage road coordination on the system for existing conditions.
Scenario 3	All signals on Fredericksburg Road, South of the IH-410 interchange, coordinated with a 190 sec. cycle length.	To test for diversion on Fredericksburg Road for links south of the IH-410 interchange for trips on IH-10 to IH-410.
Scenario 4	Scenario 3 + Interchanges coordinated.	Analyze the impact of frontage road coordination.
Scenario 5	All signals on Fredericksburg Road coordinated with a 190 sec. cycle length.	To test Fredericksburg Road as diversion route for trips going through IH-10.
Scenario 6	Scenario 5 + Coordinated Interchanges.	Analyze the impact of frontage road coordination.
Scenario 7	All interchanges coordinated at 190 sec. cycle length.	Frontage road tested as an alternate route for diversion.
Scenario 8	Scenario 3 + Forced trips to IH-410 on Fredericksburg Road.	To analyze the impact of diversion on Fredericksburg Road.
Scenario 9	Scenario 8 + Additional 750 trips forced on Fredericksburg Road.	To analyze the maximum volume that Fredericksburg Road can handle.

Scenario 2

For Scenario 2 all the conditions were the same as in Scenario 1 except that the interchanges were coordinated for frontage road progression. The interchanges south of IH-410 were coordinated at a cycle length of 95 seconds and the interchanges north of IH-410 were coordinated at a 110 second cycle. This Scenario was created to analyze the diversion impact of just coordinating the frontage road signals.

Scenario 3

In this Scenario, all the intersections south of the IH-410 interchange on Fredericksburg Road were coordinated at a cycle length of 190 seconds. To obtain the optimal signal timing plan for this case, the northbound through volumes on all intersections of Fredericksburg Road were increased so that the resulting intersection critical flow ratio was 0.85. This was done so that the cross street gets just enough green to dissipate queues within a cycle and the rest of the green would be allocated to Fredericksburg outbound direction. The objective here is to study the diversion of IH-10 outbound traffic to Fredericksburg that may result due to the additional capacity created on Fredericksburg through allocation of more green and proper coordination. If an intersection was already oversaturated at 190 seconds cycle, the northbound through movement volumes were not altered. Fredericksburg Road, south of IH-410, was tested as an alternate route in this scenario. It should be noted that the southern section of Fredericksburg could only serve as an alternate route for IH-10 outbound traffic exiting at IH-410.

Scenario 4

This Scenario was the same as Scenario 3 except that the interchanges were coordinated as in Scenario 2. The aim of creating this scenario was to observe if the coordinated interchanges would affect the performance if there was diversion on to Fredericksburg.

Scenario 5

The signal timing plans for all the intersections on Fredericksburg Road were changed. A 190 second cycle was used for all intersections. The intersections south of the IH-410 interchange were coordinated in one system, and the others north of it were treated as another system. All IH-10 interchanges were treated as isolated to obtain signal timing plans as in Scenario 1. The objective in this scenario is to study the feasibility of Fredericksburg as an alternate route to IH-10 outbound traffic going to destinations beyond Huebner.

Scenario 6

In Scenario 6 all the interchanges were coordinated as in Scenario 2, and the rest of the operational content was the same as in Scenario 5. Here again, the scenario was created to analyze the impact of frontage road coordination on the system.

Scenario 7

The aim of this Scenario was to test frontage road as an alternative route for diversion. All frontage road signals were operated at a cycle length of 190 seconds. Again, to achieve the most optimal signal splits, the volume for westbound frontage road through movements was increased. For interchanges that were already over saturated (Callaghan, Wurzbach, Crossroads, & Heubner) the outbound volumes were not changed. All interchanges south of IH-410 were treated as one system, and the remaining interchanges north of IH-410 were treated as a separate system for coordination. This was done due to frontage road discontinuity. The Fulton interchange was also signalized, although it is currently stop controlled, to provide progression and also because it is infeasible to divert large amounts of traffic through a stop controlled interchange. The origin-destination traffic demands were the same as in Scenario 1.

Scenario 8

The operational content of Scenario 8 was the same as Scenario 3 except that the trips on the IH-10 West freeway to IH-410 were extraneously diverted onto Fredericksburg Road. This was done by creating a new origin node and assigning the trips to IH-410 from downtown to the new origin. These vehicles were defined as a new vehicle type and were prohibited to enter the freeway or travel on the frontage road of IH-10. The aim of this scenario was analyze the performance of Fredericksburg Road under higher volume conditions. A forced diversion scenario had to be created because the scenarios discussed above, when simulated, did not result in any significant diversion of the outbound traffic from IH-10 to Fredericksburg. The lack of diversion may be due to the difference in outbound average travel time on Fredericksburg and the IH-10 freeway which does not favor diversion.

Scenario 9

In order to study if the signal timing strategies used in Scenario 3 would result in higher capacity for the outbound traffic, an additional 750 vph was forced on Fredericksburg Road. Of these additional trips, 500 vph were added to the new origin created in Scenario 8. These trips were also forced to use Fredericksburg Road to reach IH-410. The other 250 trips were added to the cross streets south of the IH-410 interchange to ensure that the simulated volumes on the cross streets were comparable to the observed volumes at some of the cross streets. The rest of the operational content was the same as in Scenarios 3 and 8.

MEASURES OF EFFECTIVENESS

The measures of effectiveness (MOEs) selected for the analysis of the scenarios were system/network-wide delay in vehicle-hours, deferred trips, average travel time in minutes, and average network speed in kmph.

Deferred trips are the total number of vehicles parked at the end of simulation. A vehicle is considered to be in parked mode if it cannot enter the network although it is scheduled to enter the network due to queue spill back from the entry link. These deferred departures will remain in the parked mode till they are able to enter the network.

The system wide delay in vehicle-hours was calculated by finding the difference in average travel time and free flow travel time at each link at the end of simulation multiplied by the total number of vehicles that traversed that link. Average network speed and deferred trips were generated by INTEGRATION as output from simulation.

RESULTS AND FINDINGS

Table IV-2 summarizes the results from a series of INTEGRATION runs for the various scenarios discussed above. Several comparisons and discussions with these results were made.

Analysis of the Base Scenario

Scenario 1 was created to simulate existing conditions, and the results obtained from this scenario were to be used to make comparisons. As mentioned earlier, a satisfactory OD matrix could not be obtained, and the resulting simulated link flows were inaccurate. The link volumes obtained from the simulation were compared with the observed volumes. The primary focus of comparison was on Fredericksburg outbound links as it is the alternative route to be tested for diversion

It was observed that the average peak one-hour volumes for all outbound links on Fredericksburg Road, were 28 percent lower than the actual volumes. For the northbound links on Fredericksburg Road, south of the IH-410 interchange, the peak one-hour volumes were 13 percent lower. However, the peak 15 minute volumes were 34 percent higher. Although a better OD matrix is required to obtain more accurate results, due to time constraints it was not possible. Although the average peak 15 minute volumes were higher,

TABLE IV-2. Summary of Results

Scenario	Deferred Trips	System Wide Delay (veh-hrs)	Average Travel Time on Fredericksburg (min.)	Average Travel Time on IH-10 West Freeway (min.)	Average Network Speeds (Kmph)
Scenario 1	1927	2913.38	32.3	16.0	40.0
Scenario 2	4047	2808.60	31.2	17.9	36.6
Scenario 3	3895	2745.57	26.4	17.5	37.6
Scenario 4	5346	3041.33	27.5	20.0	35.4
Scenario 5	7278	2686.86	26.2	15.9	35.8
Scenario 6	7079	3038.46	24.6	19.2	33.9
Scenario 7	2528	2929.34	28.7	17.9	39.1
Scenario 8	1869	2671.16	22.4	18.4	40.0
Scenario 9	3806	3343.24	34.4	18.7	34.0

it was found that on majority of the links, peak 15 minute flow rates compared well with the observed flow rates. Hence, it was decided that simulations can be performed with the available OD data in order to study the feasibility of diversion and identify any likely problems in achieving the objectives.

Comparison of Average Flows and Travel Time

In order to measure any diversion and its impact on the alternate route, an analysis of the changes in traffic flows and speeds under various control scenarios was performed. As surrogate measure for the average speed in a particular section, the average travel time was used. This analysis was performed for the outbound direction of Fredericksburg, the IH-10 frontage road, and IH-10 main-lanes. The analysis was performed through a comparison of the link flows and travel times in each scenario with those observed in Scenario 1.

Fredericksburg Northbound - South of IH-410 Interchange

Table IV-3 shows the percent change in average flows and travel time under different scenarios with respect to Scenario 1, for the outbound Fredericksburg links south of IH-410.

It can be seen that for all scenarios, except scenarios 8 and 9 where some IH-10 outbound traffic was extraneously diverted, the peak one-hour volumes actually reduced. This implies that overall fewer vehicles traversed the links than in Scenario 1. This is counter to the expected diversion from the freeway. The travel times, however, decreased on these links due to the lower link flows. The average simulated travel time of northbound links on Fredericksburg Road up to the IH-410 interchange is 15.2 min/veh in Scenario 1.

Although the difference between Scenario 2 and the base scenario is just the coordination of frontage road signals, it can be noticed that the link flows on the arterial are lower. This difference, however, is not very high and may be attributed to the randomness in the simulation process.

TABLE IV-3. Average Volume and Travel Time Changes for Northbound Links on Fredericksburg Road, South of IH-410

Scenario	Volume Change (Peak 1 hour)	Volume Change (Peak 15 min.)	Difference in Average Travel Time
Scenario 2	-5%	-1%	-0.1
Scenario 3	-27%	10%	-2.9
Scenario 4	-17%	10%	-3.9
Scenario 5	-25%	1%	-4.3
Scenario 6	-1%	8%	-4.2
Scenario 7	-20%	4%	-3.1
Scenario 8	38%	8%	-4.6
Scenario 9	72%	30%	5.3

The reduction in link flows on Fredericksburg is higher for Scenarios 3, 4, and 5 than other scenarios. This may be due to the long cycle length used in these scenarios for the arterial. It should be noted that the number of deferred trips increased also increased in these scenarios with respect to the base scenario. Much of the deferred trips were from the cross streets on the arterial and the frontage roads. As mentioned earlier, deferred trips are those

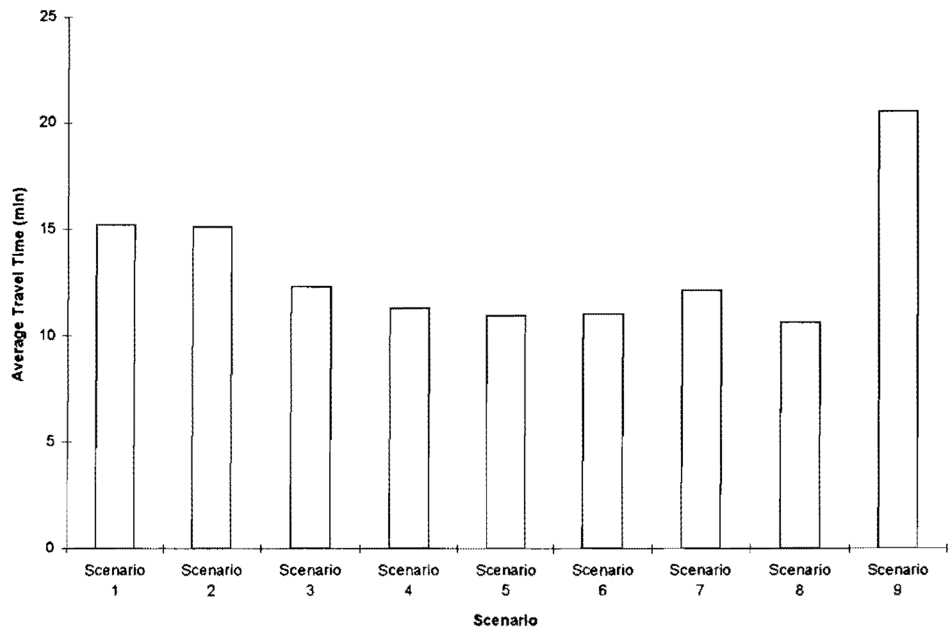

FIGURE IV-3. Average Travel Time on Northbound Links on Fredericksburg Road, South of IH-410

that could not enter the network past their scheduled departure time due to queue spill back. This implies that the long red periods resulting from the long cycle lengths might adversely affect the cross streets. Since the cross street trips could not enter the network, the link flows on the arterial reduced. Again, due to lower arterial flows, the travel time on the arterial decreased.

Although the difference between Scenario 6 and Scenario 5 is just the coordination of the frontage road signals, the link flows on the arterial are more in agreement with the base scenario. The reason for this is not apparent. The difference in link flows is too high to be attributed to the randomness in the simulation process.

In Scenario 8 there is a 38 percent increase in peak one-hour volumes and a 4.6 minute decrease in average travel time. It can be seen that the higher volumes as compared to Scenario 3 did not result in any corresponding drop in speeds. As mentioned earlier, the difference between Scenarios 3 and 8 is that in the latter, IH-10 outbound trips ending at IH-410 were extraneously diverted to the arterial. The lack of difference in speeds despite the higher volumes implies that the timing plan provided in Scenarios 3 and 8 is capable of accommodating higher traffic volumes.

The simulation logic did not automatically divert any additional traffic from the freeway, despite the additional capacity created through the arterial timing plan provided in Scenario 3. The absence of diversion may be due to the large differences in travel time between the freeway and the arterial. Despite the congestion on the freeway, the arterial travel time is higher due to the large number of signalized intersection. The difference in travel times renders the arterial unattractive for diversion.

It can also be seen from Table IV-2 that the overall system wide delay, deferred trips, and travel times are the lowest in Scenario 8 as compared to others. This is mainly due to the improved travel time on the arterial. It should be noted that the diversion of some traffic from IH-10 did not result in an improvement in the freeway speeds because the weaving problem moved from the vicinity of IH-410 to a downstream location. This is further discussed later in this section of the report. The reason for the improvement in travel times on the arterial despite the additional traffic flows and the same signal timing plans as in Scenario 3 is not readily apparent.

In Scenario 9, in order to estimate the actual capacity available for the outbound through movement, additional traffic was extraneously introduced in the outbound direction of Fredericksburg. The IH-410 interchange and Crossroads intersection were oversaturated due to the high increase in volumes. The peak 15 minute volumes at Crossroads was comparable to the capacity estimated in Chapter III. Although the maximum flows estimated in Chapter III for the outbound movement were not exactly matched by the flows observed in the simulation, the simulation showed that the signal timing strategy does actually result in additional capacity for the outbound through movement on Fredericksburg, and also that the critical intersection would be Crossroads. Since the maximum capacity of the arterial was reached, queue spill back resulted on Fredericksburg Road, and the travel time increased by 5.3 minutes. Figure IV-3 compares the average travel time of all vehicles northbound on Fredericksburg Road, up to the IH-410 interchange.

Fredericksburg Northbound - All links

In this section, the MOEs observed for the north and south sections of Fredericksburg together is discussed. This analysis will show the diversion impacts of the timing strategies used in various scenarios on both sections of Fredericksburg combined.

Figure IV-4 depicts the average travel times for all the scenarios. Table IV-4 shows the average change in link flow rates and travel times for each scenario with respect to Scenario 1 for all northbound Fredericksburg links combined.

Scenarios 2, 3, 4, 5, and 6 do not involve any changes to the control strategies at Fredericksburg signals north of the IH-410 interchange. It can be seen from Table IV-4 that the average percent difference in link flow rates is lower than for the south section alone. This implies that the flow rates in the north section of Fredericksburg either did nor change or increased with respect to Scenario 1. This is as would be expected. Travel time also decreased as in the south section.

In Scenario 6, however, the percent difference in link flow rates is much higher for the two sections together than for the south section alone. This implies that all the difference is due to the change in link flow rates north of the IH-410 interchange. This is again due to the large cycle length which resulted in more cross street deferred trips, lower link flow rates, and improved travel time.

In Scenario 7, although no changes were made with respect to Scenario 1 on Fredericksburg, a drop in link flows was observed. This may be due to the spill back from Huebner. The IH-10 interchange at Huebner may be affected due to the large cycle length on the frontage road. The queues on Huebner may spill back and affect the turning movement from Fredericksburg thus affect the flow on Fredericksburg. It should be noted that Huebner is the only arterial coded in the simulation network connecting IH-10 and Fredericksburg. Other cross streets were not coded entirely in order to reduce the complexity of the model.

TABLE IV-4. Average Volume and Travel Time Changes for Northbound Links on Fredericksburg Road

Scenario	Volume Change (Peak 1 hour)	Volume Change (Peak 15 min.)	Difference in Average Travel Time
Scenario 2	-4%	34%	-1.1
Scenario 3	-17%	36%	-5.9
Scenario 4	-15%	40%	-4.8
Scenario 5	-22%	23%	-6.1
Scenario 6	-11%	36%	-7.7
Scenario 7	-14%	33%	-3.6
Scenario 8	8%	29%	-9.9
Scenario 9	20%	45%	2.1

Scenarios 8 and 9 resulted in smaller percent change in link flows again because there is no impact on the northern section of Fredericksburg in these scenarios. These scenarios involved an extraneous diversion from the freeway to Fredericksburg. These extra outbound

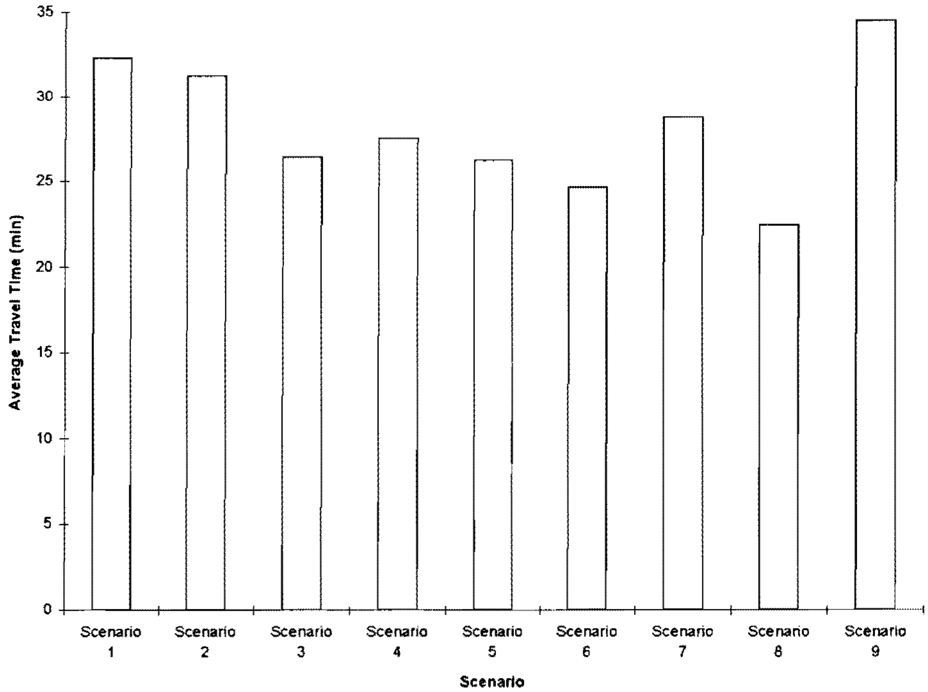

FIGURE IV-4. Average Travel Time on All Northbound Links on Fredericksburg Road

trips on Fredericksburg, however, do not go beyond IH-410 and have no impact on the northern section.

It can be seen from Table IV-2 that the average travel time on Fredericksburg is highest in Scenarios 1 and 9. The lowest travel time is in Scenario 8. The reason for this is not apparent. The only difference between Scenario 3 and Scenario 8 is that in the later an additional 500 vph have been diverted on to Fredericksburg.

Table IV-2 also shows the average travel time for the IH-10 freeway and Fredericksburg. It can be seen from the table that the best travel time on the arterial is higher than the worst travel time on the freeway. As mentioned earlier, it may be due to this difference in travel times that there was no automatic diversion from the freeway to the arterial. The large number of signals on Fredericksburg add significantly to the total travel time on the arterial and make it less attractive than the freeway despite the congestion on the freeway.

IH-10 Frontage Road Westbound

Table IV-5 below shows the changes in volumes and travel times on the westbound frontage road. Figure IV-5 shows the average travel time on the westbound frontage road for the various scenarios.

TABLE IV-5. Average Volume and Travel Time Changes for IH-10 Westbound Frontage Road links

Scenario	Volume Change (Peak 1 hour)	Volume Change (Peak 15 min.)	Difference in Average Travel Time
Scenario 2	-8%	31%	0.4
Scenario 3	-9%	35%	0.7
Scenario 4	-8%	31%	1.0
Scenario 5	-7%	35%	0.6
Scenario 6	-11%	25%	3.3
Scenario 7	-10%	49%	3.1
Scenario 8	7%	42%	4.1
Scenario 9	1%	42%	8.1

It can be seen from Table IV-5 that the frontage road volumes under the different scenarios are lower than in Scenario 1. The travel times are also higher in all scenarios as compared to Scenario 1. It was observed that the volumes on the freeway main lanes were also lower for the different scenarios tested. This implies that there was no sustained diversion from the freeway to the frontage road. Any reduction in flow rates may be due to the deferred trips. It can be seen, however, that the peak 15 minute volumes registered a substantial increase with respect to Scenario 1. This implies that there may be some diversion from the freeway to the frontage roads for a short duration. This diversion may have led to the rise in average travel time on the frontage road.

All IH-10 diamond interchanges in the study area were provided TTI-Lead phasing for all the scenarios. It was found that TTI-Lead phasing is not favorable for frontage road through movements as it allocates large amounts of green time to the interior movements in order to avoid storage of vehicles in the interior. The provision of TTI-Lead phasing may have led to a lack of capacity for the frontage road through movement and to a lack of diversion.

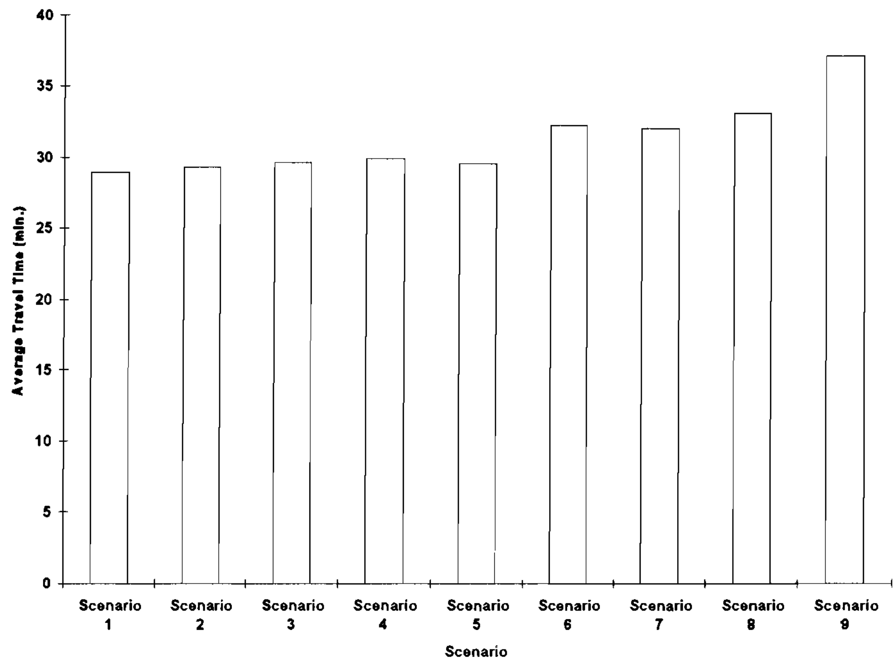

FIGURE IV-5. Average Travel Time on IH-10 Westbound Frontage Road Links

Figure IV-5 also shows the travel time on the frontage road. It can be seen that the best travel time on the frontage road among all scenarios is still higher than the travel time on the freeway main lanes. This may be a reason for the lack of diversion of freeway main lane traffic to the frontage road.

IH-10 Main Lane Westbound

The main problem section on the freeway in Scenario 1 was the interchange of IH-10 and IH-410. Other short weaving areas apart from the IH-410 interchange also exist in the study section. The average travel time for westbound links on the IH-10 freeway were compared to analyze the impact of diverting the outbound traffic on the performance of the freeway.

Figure IV-6 compares the westbound travel time on IH-10. The average travel time on IH-10 West is sixteen minutes in Scenario 1. As can be seen from Figure IV-6, the travel time on the freeway increased with respect to the base scenario in all scenarios except Scenario 5. The variation, however, is not substantial.

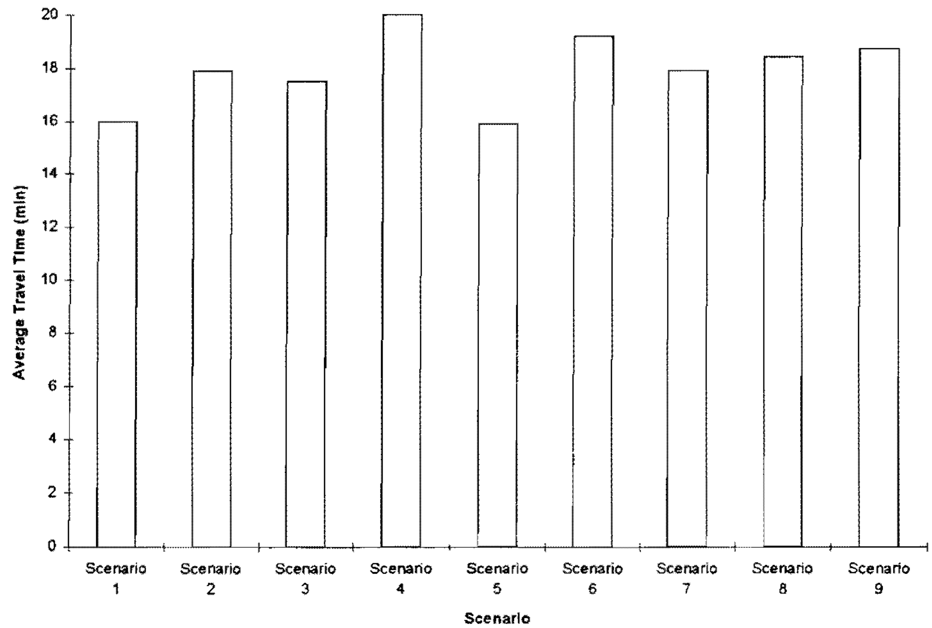

FIGURE IV-6. Average Travel Time on IH-10 Westbound Freeway Links

The reasons for the increase in travel time on the freeway are not very apparent. Despite the forced diversion of IH-410 bound trips (about 500 vph) from the freeway, the travel time for Scenarios 8 and 9 was higher than the base travel time on the freeway. It was observed from the simulation that the performance of the freeway improved until the IH-410 exit and entrance ramps, but the problem area was now shifted to the link upstream of the exit to Heubner. The volumes on the freeway were lesser but the volume on the exit ramp to Heubner was near saturation. The high exit volumes at Huebner could be either because the trips to Heubner were using the freeway instead of the frontage road under this scenario, or because these trips were metered earlier by the bottleneck at IH-410 and no problem was observed. As a result of this new bottleneck, no improvement was observed in the freeway travel times even though some of the traffic was extraneously diverted onto Fredericksburg.

The freeway travel time is higher in all scenarios where longer cycle length than the minimum delay cycle was used. This may be because TTI-Lead phasing was utilized in all the coordinated frontage road plans as well as in Scenario 7 where all interchange signals are operated at 190 seconds cycle. An mentioned earlier, in order to avoid interior queue storage, TTI-Lead phasing allocates a large amount of green to the interior movements at the diamond interchange. This results in lower green times for the arterial and ramp phases. The lower green times reduce the capacity for the frontage road through movement which in turn may result in more vehicles remaining on the freeway rather than using the frontage road, resulting in higher travel times on the freeway. Also, the reduced capacity may lead to spill backs from the exit ramps.

In order to create additional capacity for the frontage road through movement, it was found that three-phase timing is more appropriate (refer Chapter III). With three-phase timing, however, interior storage may be a problem if heavy turning movements also exist, or if long cycle lengths are used. Due to time constraints it was not possible to test other timing strategies for the interchanges.

TABLE IV-6. Average Volume and Travel Time Changes for IH-10 Westbound Freeway Links

Scenario	Volume Change (Peak 1 hour)	Volume Change (Peak 15 min.)	Difference in Average Travel Time
Scenario 2	-16%	15%	1.9
Scenario 3	-22%	3%	1.5
Scenario 4	-19%	0%	4.0
Scenario 5	-18%	7%	-0.1
Scenario 6	-23%	7%	3.2
Scenario 7	-9%	7%	1.9
Scenario 8	-15%	-3%	2.4
Scenario 9	-12%	9%	2.7

It can be seen from Table IV-6 that the peak one-hour volumes reduced on the westbound freeway as compared to the base scenario. This reduction may not be due to diversion, because it was observed that the peak one-hour flows on alternatives to IH-10 West, namely frontage road and Fredericksburg, also reduced. The observed reduction may be mainly due to the deferred trips which failed to enter the network due to spill back.

System Wide Delay

Finally, a comparison of system wide delays was made to get a network wide perspective of all the scenarios. System wide delay is not generated by INTEGRATION and was calculated from the average travel times, free flow travel times and the link flows that

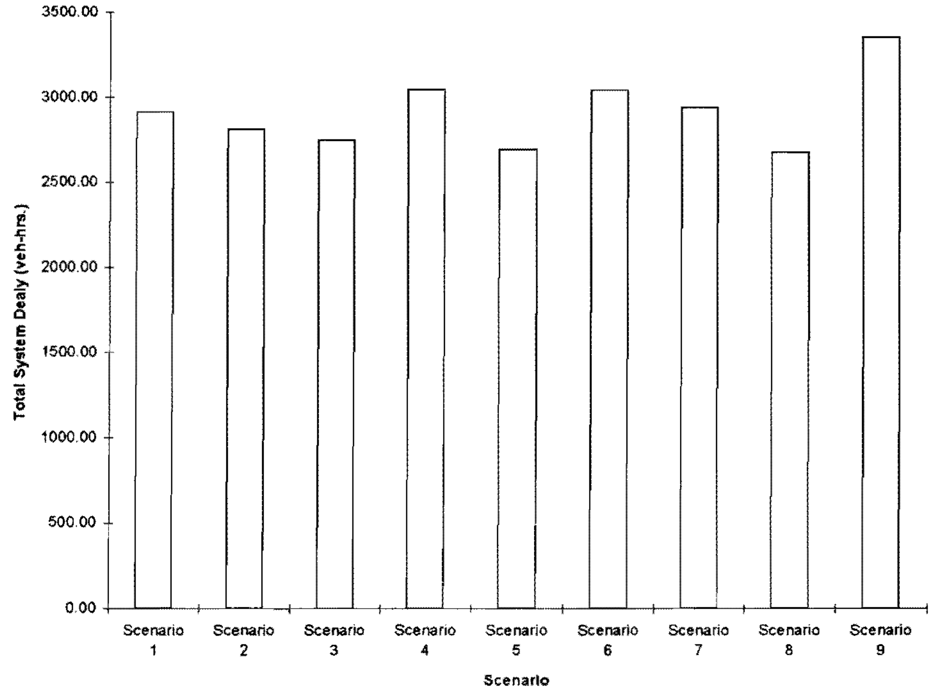

FIGURE IV-7. System Wide Delay

were available in the output file. Figure IV-7 compares the system wide delays for all the scenarios. Apart from travel times, system wide delay is a also a function of the number of deferred trips. Since deferred trips do not enter the network until the end of the simulation period, although they are scheduled to enter, their travel times and speeds are not recorded by INTEGRATION. Hence, the resultant system delay may be lower.

It can be seen from Figure IV-7 that Scenario 8 has the least system wide delay of all the scenarios. It should also be noted that Scenario 8 has the least number of deferred trips (refer Table IV-2). This implies that diversion of IH-410 bound traffic from IH-10 to Fredericksburg will result in an overall system wide benefit although the diverted traffic may not experience any reduction in travel time. The travel time on Fredericksburg reduced in spite of increase in volumes. This is a contributing factor for a lower system wide delay. On IH-10 west, though, there was an increase in travel time as mentioned earlier, and also the volumes were lower; hence this did not affect the system wide delay in Scenario 8. On the other hand, in Scenario 9, since there was increase in travel time as well as volumes on Fredericksburg Road, the system wide delay was maximum.

PROBLEMS ENCOUNTERED

Arriving at a satisfactory OD table was the major problem faced during the process of network building. INTEGRATION did not have any way of computing OD's from link volumes, so QUEENSOD had to be used to obtain the origin-destination information from observed link volumes. Since INTEGRATION was dynamic and static OD estimation was used in QUEENSOD, there was very little control over the errors between actual link flows, link flows estimated by QUEENSOD, and the resulting link flows in INTEGRATION. A seed OD matrix was made to minimize these errors, but there was no way to calibrate it well. An inaccurate OD table then led to a number of other problems.

The link flows in INTEGRATION were often different from the volumes used for signal timing in PASSER II and PASSER III. Also, due to the absence of any representation of midblock access points on the arterial, QUEENSOD assigned all midblock traffic to the cross streets resulting in high volumes for the cross street. Because of this incompatibility, the signals timings may not be optimal for the simulated volumes and may have resulted in a number of deferred trips from the cross streets.

Also due to dynamic assignment, minimum paths would change rather rapidly sometimes depending upon how the link was performing at that time. The rapid change in minimum paths was, however, controlled by reducing the frequency of minimum path tree computations in the model.

INTEGRATION did not divert traffic on to Fredericksburg road as it was expected to. This may be due to the large number of signals on the arterials and the entailing delay. Hence, the traffic had to be forcefully diverted. Simulating and monitoring a huge corridor was quite a difficult task because the model is very sensitive to small changes. Spotting and monitoring problem areas were difficult due to the dynamic nature of the model. It was also not possible to specify an external tree/path for the vehicles.

CONCLUSIONS

Several conclusions may be made based on the simulation regarding the feasibility of diversion of outbound IH-10 main lane traffic to Fredericksburg and the frontage roads. It should be noted, however, that the origin-destination data used for the simulation did not yield the observed link volumes in the study area. All observations should be interpreted in this context. Some observations may hold good despite the origin destination data because the levels of congestion observed on the freeway during field studies were also observed in simulation.

It was found that the travel time on Fredericksburg is much higher than the travel time on the freeway main lanes despite the congested conditions on the freeway main lanes. This may be the main reason for the lack of diversion from the freeway main lanes despite the signal retiming on the arterial. The large number of signalized intersections on Fredericksburg add considerably to the total travel time on the arterial and render it unattractive for IH-10 outbound traffic as an alternate route.

The use of TTI-Lead phasing on the diamond interchanges may have led to the lack of any diversion of the outbound traffic to the frontage road. During the analysis for excess capacity at diamond interchanges TTI-Lead phasing allocates large amounts of green time to the interior movements in order to avoid storage of vehicles in the interior. Therefore, this phasing scheme is not particularly suitable for creating additional capacity for the frontage road through movement. Other phasing schemes were not simulated in this study. Analysis using PASSER III, however, showed that three-phase timing plans are better for creating excess capacity for frontage road through movement, and interior storage does not pose a problem at any of the interchanges in the study section. Cycle lengths above 150 seconds, however, may result in interior storage problems with three-phase timing plans. A simulation analysis using the model developed in this study may be performed to evaluate the usage of three-phase timing plan to facilitate the use of frontage road by the main-lane traffic.

Although the dynamic assignment logic in INTEGRATION did not automatically divert freeway traffic to the arterial despite congestion on the freeway, diversion was extraneously induced to verify whether the signal timing plans proposed for the arterial can actually accommodate the excess traffic. It was found that the diverted traffic can be accommodated on Fredericksburg south of IH-410. As the analysis in Chapter III showed, the Crossroads intersection and the IH-410 interchange were found to be critical in determining the amount of traffic that can be diverted.

During the simulation experiments, it was observed that when traffic is diverted from the main-lanes to Fredericksburg Road, the resulting weaving leads to reduced capacity and speeds on the freeway main lanes upstream of the Fredericksburg/Woodlawn exit. If the existing speeds in this section are not already low, diversion may introduce the problem there.

Outbound trips to IH-410 were extraneously induced to use Fredericksburg instead of IH-10. The removal of these trips from the freeway resulted in reduced weaving activity and improved speeds on IH-10 main lanes in the vicinity of IH-410 entrance and exit ramps. The better flow of traffic in this section resulted in new weaving problems upstream of the exit to Huebner. Further analysis needs to be performed in order to ascertain this in view of the lack of accuracy of the origin-destination data used in this analysis.

The diversion of IH-410 bound traffic from IH-10 resulted in the least system-wide delay. This implies that although the travel time is higher on the arterial for the diverted traffic, the reduced weaving on the freeway results in overall system-wide benefits. Since IH-

410 was not included in the study network, the problems associated with the diverted traffic reentering the IH-410 traffic stream at Fredericksburg interchange could not be studied.

Long cycle length in combination with allocation of all excess greens to the arterial may impact the cross street adversely. This can be noticed from the number of deferred trips in scenarios with long cycle lengths. This problem should be given particular attention during the development of arterial and frontage road timing plans for diversion. Long queues on important cross streets including Huebner, Wurzbach, Callaghan, Data Point, Medical, etc., may lead to problems elsewhere in the corridor.

The simulation study was aimed at studying the possibilities for diversion during recurring PM-peak hour congestion in the study area. Based on the travel times observed, it can be concluded that diversion may not be feasible for recurring congestion. During incidents, however, when there is a substantial reduction in freeway capacity, diversion to Fredericksburg may be beneficial. As the simulation analysis and PASSER II/PASSER III based analysis in the previous chapter showed, capacity is available on Fredericksburg for a limited amount of diversion depending on the time of day diversion is undertaken and the extent of diversion (i.e., diversion up to the IH-410 interchange on Fredericksburg, or diversion all the way up to the Fredericksburg/IH-10 interchange north of Heubner).

As the base case scenario could not be calibrated further, it was not possible to make more concrete conclusions and study other viable alternatives and options. The simulation results, however, are in conformance with observations made in the study area through field trips and other analysis methods discussed in the previous chapters. The dynamic nature of the INTEGRATION model and its sensitivity to small changes in geometry and control schemes, although desirable, make it very difficult to calibrate the model more accurately.

CHAPTER V.
ARTERIAL CHANGEABLE MESSAGE SIGNS

Real-time motorist information displays, particularly changeable message signs(CMSs), are playing an increasingly important role in attempts to improve highway safety, operations, and use of existing facilities. For over 30 years, CMSs have been used in urban environments as a means of relating real-time information about the driving environment to individual motorists. The flexibility of CMSs allows the display of a variety of information about the nature of the downstream traffic conditions and possible diversion routes that can be taken to avoid the problems associated with incidents/congestion. As part of the TransGuide system in San Antonio, several CMSs have already been installed on the freeways.

Arterial CMSs can be used not only to provide trail blazing information for diverted traffic but also to indicate any problems detected on the arterial street system. With background knowledge of the street-freeway network or information provided on the CMS, the informed driver can make alternative route choices and avoid the hazard and congestion caused by incidents.

For this study, CMSs were investigated for their possible use in San Antonio as part of an arterial diversion concept along Interstate 10 on the city's northwest side. Approximately five kilometers north of downtown, IH-10 and Fredericksburg diverge, (both) continue past Loop 410, and reconnect approximately 11 kilometers further, outside of town. Not only is the Interstate near capacity during peak hours, but the interchange of IH-10 and Loop 410 is also highly congested. To attempt to alleviate some of this congestion, especially during incident conditions on IH-10, Fredericksburg Road was investigated as an alternative route for traffic headed out of downtown along IH-10. CMSs that are currently part of the TransGuide traffic management system could direct motorists onto Fredericksburg, and new arterial street CMSs on Fredericksburg could be used to direct motorists to their ultimate destination.

In order for the arterial CMSs to fill their role in the diversion scenarios being investigated along IH-10, they must meet certain criteria. In addition, the placement, message type, environment, size, and visibility of the sign must be considered to ensure sign effectiveness. Where available, the experience of other agencies that use CMSs on arterial streets was sought and recorded. Finally, recommendations are presented that discuss the use of CMSs along Fredericksburg Road.

CMS CRITERIA

The photometric and physical design requirements for CMSs are based on the following four functional requirements. (*10*):

1. Conspicuity: It is the quality of an object or a light source to appear prominent in the surroundings. It is the capability of one entity in the visual field to be more easily noticed than any surrounding information.

2. Legibility: The legibility of a sign is a measure of how readily an observer may recognize the words or symbols. It is usually measured in terms of the threshold distance at which the sign becomes legible.

3. Comprehensibility : It is a measure of how readily an observer can understand the message intended to be conveyed by the sign.

4. Credibility: This refers to the extent to which motorists believe that a traffic control device has a message that is reliable, accurate, up-to-date, and pertinent. A driver information system can be successful only if the drivers believe in the information provided by it. If used for diversion routing, the information must ensure that the recommended alternate route results in a significant improvement in travel.

ARTERIAL APPLICATIONS OF CMSs

Applications of CMSs to arterial streets can be found in Santa Anna and Anaheim, California and in Minneapolis, Minnesota. The Santa Anna system is flexible and will be used for incident management, congestion management, or motorist information, depending on the status of the roadway network. The Anaheim and Minneapolis CMS systems were developed for traffic and congestion management around special events centers. Irvine, California is planning a small CMS installation for congestion management and Dallas, Texas used CMSs on Skillman Avenue as early as 1973 to display information about the nearby North Central Expressway. Very little literature is available on the guidelines used in the design of the arterial CMSs mentioned above. Venglar summarized the arterial CMS applications mentioned above based on telephone interviews with engineers involved in the design and operation of the signs (*11*).

Santa Anna, California

The City of Santa Anna installed a system consisting of sixteen CMSs located at strategic points along the arterial street system. The CMSs are intended to be used for traffic management during recurring and non-recurring congestion in the city's network of freeways and arterial streets.

Predetermined, computer-stored messages for different traffic conditions are displayed based on surveillance information from loop-detectors and cameras placed on the freeways. The CMSs are intended to only convey information to the motorists. The signs are capable of displaying two fifteen-character lines of 191 mm high characters. Visibility was a concern due to the small the character size. The 3962 X 1828 mm CMSs are mounted between 2438 and 3048 mm high. The signs are placed 183 to 244 meters in advance of decision points where drivers may make alternate route choices.

Aesthetics was an overriding concern of the city. Other concerns surrounding the CMS use in Santa Anna included the acceptability of the signs to the local residents and businesses and whether to use the signs only when needed or constantly.

Anaheim, California

Anaheim, California has installed a CMS system to guide unfamiliar motorists to and around the Disneyland amusement park. The CMS's primarily provide information about the status of the local road network and parking facilities in the vicinity of the park.

The CMS messages are computer-stored and selected in real-time based on the condition of the transportation network. Anaheim's arterial CMSs are full matrix LEDs with a width of 6096 mm and a height of 1524 mm. The full matrix LED allows some variability in letter height and style for gaining the attention of drivers and emphasizing important information. Blockage of local business signs and visual clutter along the city streets were the main concerns expressed by the public following CMS installation. Overall, the city is pleased with the usefulness and success of the CMS system.

Minneapolis, Minnesota

The CMS system in Minneapolis was developed for congestion in the vicinity of a convention center. The signs are located on arterials providing access to parking facilities serving the complex. The CMSs are mainly used to convey information on parking availability. In addition, the signs are also used to divert traffic away from the center after events.

The system consists of four electromagnetic reverse polarization flip disk signs that are 1524 mm high by 2438 mm wide and capable of displaying two or three lines of text. The signs reflect light during the day and are backlit at night. Eight characters per line are possible using a predetermined 356 mm minimum letter height. However, character selection is flexible and fewer characters per line can be used with an 559 mm maximum letter possible. Predetermined messages are displayed in response to emergent traffic conditions by personnel in the control center.

Dallas, Texas

The city of Dallas, Texas used three CMSs on Skillman Avenue for providing information about the nearby North Central Expressway as early as 1973. The rotating drum signs had 13 possible message displays with four drums. The 305 mm character drums were remotely controlled via phone lines, and the system included detection and closed circuit TV.

ISSUES IN USE AND OPERATION OF ARTERIAL CMS

As can be seen from past experiences with arterial CMSs discussed in the previous section, the key issues involved in the operation of the CMSs include the placement, message type, environment, size, and visibility. Venglar studied these issues with particular emphasis on arterial CMSs (*11*). The findings are summarized in the following paragraphs.

Placement

The guidelines for placement of arterials CMSs are based on similar guidelines available for freeways(*12*). The main issues in the placement of freeway CMSs are their distance from other guide signs, exit ramps, and interchanges. The same issues are applicable to arterial CMSs.

All static sign and CMS spacing on arterials should be such that drivers have sufficient time to recognize, read, and comprehend one signed message before reaching another. Developing an equation or standard to locate the CMSs on arterials is difficult due to the large number of variables involved, including sign size, letter heights, visibility, conspicuity, arterial speed, driver vigilance, and the complexity of the driving environment. For an arterial speed of 48 km/hr and an eight word message, at least four seconds should be allowed for CMS perception and reaction.

CMSs should be located upstream of potential decision points to give drivers time to read and comprehend the message, decide on an alternative route, and execute the maneuvers necessary to make the diversion. The time required to make the necessary maneuvers is dependent on the traffic and geometric conditions. Trailblazing information should also be provided upstream of major intersections where the motorist is likely to be confused. Whenever traffic is traveling on a temporary bypass and is required to return to the original route, some drivers may inadvertently fail to see or comprehend the return signs. To prevent them from traveling a considerable distance before discovering their error, a "forgiving sign system"may be employed (*13*).

The Manual on Uniform Traffic Control Devices (*14*) recommends that the edge of the sign be at least 610 mm from the left edge of the inside curb in business and residential areas. Minimum heights of 2134 and 5128 mm are recommended for roadside and overhead signs. Because of the size, weight, and necessary support structure for CMS's, they should be placed as far as possible from the travel lanes.

Message Type

The arterial street CMSs used for incident management should employ an attention statement and an action statement. The attention statement describes the incident and identifies the portion of the arterial being addressed. The action statement describes the action to be executed by the affected drivers to avoid the incident. Among the general guidelines for the design of CMS message statements developed by Dudek (*13*), the following are relevant to arterial CMSs in particular.

1. An attention statement must always be accompanied by an action statement;

2. Generally the word "traffic" after a destination name is not necessary;

3. Names used for cities should be identical to those on existing static signing;

4. Names used for major generators must be specific and address the exact place where an activity takes place;

5. Names describing certain special activities should be displayed rather than the location where the activity is being held;

6. Use of a highway marker is preferred to a written highway number;

7. Highway route numbers should be displayed when referring to highways used for intercity trips;

8. Local highway names can be used for intracity trips if the intent is to communicate with local commuters;

9. Locally popular descriptors for frontage roads should be used;

10. Giving a diversion route a name which implies characteristics which the facility or route does not possess may weaken confidence in signing. It is better to use either an appropriate name or one which carries no particular connotations;

11. Before diverting, drivers desire to know that the incident bypass route will eventually return to the primary route and the point at which they will be returned;

12. Drivers need to know as a minimum what they should do and one good reason for doing it;

13. Because of the limitations on the amount of visual information drivers can read and recall, it is preferred to exclude lane blockage information and include diversion information when combination messages are displayed; and

14. Trailblazing signs used for diversion routing should be clearly identified with their alternate route.

Environment

Environmental issues in CMS placement and operation refer to the driving environment and local land use. If the arterial is already cluttered with road signs and advertisements, CMSs may not be very effective in serving their intended purpose. In environments where driver work load is high, drivers may ignore the CMS in favor of information of higher priority.

Sensitivity to local residents and businesses is essential for CMS to be effective. In residential areas, aesthetics (due to large size of CMSs) and the fact that the signs either emit or reflect light at night are major issues. In commercial areas, businessmen may be concerned that the CMS will distract attention from or block their advertising signs.

Size

Size includes various CMSs features such as the physical size of the sign, character size, number of lines, number of characters per line, number of units of information, etc. The factors that affect reading time such as, driver work load, message load, message length, message familiarity, and display format determine the sizing of CMSs. These factors are detailed in the following list, developed by Dudek (*15*):

1. The message must be legible at a distance that allows sufficient exposure time for drivers to attend to the complex driving situations and glance at the sign a sufficient number of times to read and comprehend the message;

2. There is evidence that no more than three units of information should be displayed on one sequence when all three units must be recalled by drivers. Four units may be displayed when one of the units is minor;

3. A unit of information may be displayed on more than one line on the sign. However, a sign line should not contain more than two units of information;

4. There is evidence that an 8-word message, excluding prepositions, is approaching the processing limits of drivers at high speeds;

5. Research indicates that a minimum exposure time of one second per short word (four to eight characters) or two seconds per unit of information, whichever is largest, should be used for unfamiliar drivers. On a sign with 12 to 16 characters per line, this minimum exposure will be two seconds per line; and

6. For a message containing three or more phrases or elements that are sequenced or cycled on a sign, three or more "stars" or asterisks should be displayed on a frame at the end of the cycle to positively separate successive repetitions of the message.

Visibility

The visibility of signs and other traffic control devices depends on the visual capabilities of motorists and the photometric qualities of the devices. The two aspects of sign visibility are conspicuity and legibility. Generally, the characteristics that make a sign conspicuous also make it legible. This, however, is not true if a sign is too luminous, making it conspicuous but illegible because of glare. Visibility concerns for light emitting and light reflecting signs differ.

Factors affecting the legibility of light-emitting CMSs include character height; font style; pixel size and spacing; spacing of characters, lines and, words; sign border size; and contrast ratio (*15*). Though freeway experience with CMSs has indicated that at least an 457 mm letter height is necessary, no such standard exists for arterial applications. Legibility information is available for various technologies, and it is presented in Table V-1. Table values indicate sign legibility under several conditions, including daylight, night, washout, and blacklight. The table distances produce a nominal legibility for light-emitting CMSs of 432 to 480 mm per mm of letter height.

Light reflecting CMSs generally have shorter legibility distances than light-emitting CMSs. Disk matrix signs were found to have daylight, night, washout, and blacklight legibility distances of 213, 108, 128, and 67 meters, respectively. Four hundred fifty seven mm letter height reflective disk signs were found to exhibit daylight legibility of 221 meters for the 50th percentile and 152 meters for the 85th percentile driver (*15*).

TABLE V-1. Legibility Distances (meters) for Light- Emitting CMSs (*15*)

Condition	Bulb Matrix		LED	Fiber Optic
	50th %ile	85th %ile		
Daylight	259	213	226	300
Night	N/A	N/A	212	207
Washout	N/A	N/A	148	260
Blacklight	N/A	N/A	153	201

RECOMMENDATIONS

Past experience with the use of arterial CMSs was discussed. Based on the past experience and based on similar guidelines for freeway changeable message sign usage, guidelines for CMS use on arterials has been presented.

Based on the guidelines discussed in this chapter the following recommendations are made for the use of CMSs on the freeway and arterial for enabling diversion of traffic to Fredericksburg.

As discussed in Chapters III and IV, it was found that during the peak hours, Fredericksburg is not suitable for diversion of IH-10 outbound traffic beyond IH-410 interchange due to lack of excess capacity on Frederickburg. During off-peak hours, however, there is excess capacity throughout Fredericksburg for additional diverted traffic.

Although, during off-peak hours Fredericksburg may be used for diversion, the frontage road may be a better alternative due to a fewer number of signalized intersections. Hence, installation of CMSs signs on Fredericksburg purely for the purposes of diversion may not be warranted.

If the excess capacity on Fredericksburg south of the IH-410 interchange is used during the A.M. and P.M. peak hours for diverting IH-10 outbound traffic exiting at IH-410, only one trailblazer sign may be necessary. This sign should be placed at the east side of the first outbound interchange of Fredericksburg with IH-10. This sign may be used to display a simple message such as "TEMP BYPASS TO IH-410" with a left arrow. As there are no major cross streets in this section of Fredericksburg south of IH-410, diverted motorists are not likely to be confused. Normal signing for IH-410 on Fredericksburg may be sufficient for diverted traffic to identify the freeway. A 6096 mm wide by 1524 mm wide sign may be sufficient.

If the frontage road is used for diverting traffic during off-peak hours, CMSs are recommended for the frontage road at least at one location on the outbound frontage road. At IH-410, the IH-10 outbound frontage road is continued as the IH-410 westbound frontage road. In order to continue on the frontage road in the outbound direction, a U-turn is necessary at Cherry Ridge. A CMS may be used on the frontage road at Cherry Ridge interchange to guide diverted traffic through the interchange back to the IH-10 outbound frontage road.

CHAPTER VI.
CONCLUSIONS AND RECOMMENDATIONS

The implementation of ITS systems in Texas, including the TransGuide center of San Antonio, increased the interest in corridor traffic management strategies. One of the strategies being considered in Texas is the use of excess capacity available on arterial streets and frontage roads to divert traffic from freeways in order to relieve recurring and non-recurring congestion. This study investigates the feasibility of using integrated corridor traffic management strategies for relieving recurring and non-recurring congestion in the IH-10/Fredericksburg Road corridor. Although the study corridor was used as a test site for the application of the methodologies developed in this study, the methodologies are applicable to any arterial or frontage road.

A methodology for the estimation of excess capacity available at signalized intersections and interchanges for diverted traffic was developed. This methodology was applied to determine the excess capacity in the outbound direction of Fredericksburg Road and the IH-10 frontage road during the peak and off-peak periods.

Researchers found that during the A.M. and P.M. peak periods, the section of Fredericksburg Road south of IH-410 has some excess capacity for outbound traffic diverted from IH-10. The section north of IH-410, however, was found to have no excess capacity due to the large cross street and turning movement volumes. During the off-peak period, however, both sections of Fredericksburg have excess capacity for diverted traffic.

The outbound IH-10 frontage road contains excess capacity during the off-peak period. During the A.M. peak period, the Callaghan and Huebner interchanges do not have any excess capacity. Vance Jackson, Crossroads, and Hildebrand have a small amount of excess capacity for diverted traffic. Hildebrand, Vance Jackson, and West Avenue were also found to be critical during the P.M. peak period. Geometric improvements at Hildebrand, Fresno, and West Avenue increased the capacity for the outbound through movement.

For the interchanges, it was found that the three-phase timing plan for diamond interchanges results in higher capacity for the frontage road through movement than TTI-Lead phasing. This is because in TTI-Lead phasing a large amount of green time is allocated to the interior movements in order to avoid interior storage of vehicles. It was also observed that with three-phase timing plans, long cycle lengths are not possible due to storage problems.

A simulation study, using INTEGRATION, examined the amount of diversion to the frontage road and Fredericksburg when timing plans to create excess capacity for the outbound traffic are implemented. Although the model needs further calibration, several observations were made. The simulation revealed that despite the P.M. hour congestion experienced by the outbound IH-10 traffic, the travel time on the frontage road as well as Fredericksburg is higher. Therefore, no significant traffic diversion was observed.

Although the simulation logic did not result in any diversion, IH-410 traffic was diverted to Fredericksburg extraneously, to study if the timing plans provided on Fredericksburg could actually accommodate any excess diverted traffic. It was found that, as estimated, the section of Fredericksburg south of IH-410 can accommodate more traffic with the proposed timing plans for the arterial. As projected, the Crossroads intersection and the IH-410 interchange were found to be critical in determining the amount of traffic that can be diverted.

Weaving at the IH-410 entrance and exit ramps resulted in increased travel time for the outbound traffic. Diverting IH-410 bound traffic IH-10 to Fredericksburg produced reduced weaving activity in the vicinity of IH-410 entrance and exit ramps. This resulted in better speeds and travel time for the outbound freeway traffic up to the IH-410 interchange. New weaving problems upstream of Heubner exit were observed in the absence of the weaving problem at IH-410.

The simulation and PASSER II/PASSER III analysis showed that it may not be beneficial to divert traffic to the frontage road and Fredericksburg to alleviate recurring peak hour congestion on IH-10 due to the lack of additional capacity and long travel times. However, during the off-peak period, diversion may be feasible because of the availability of excess capacity both on the frontage road as well as Fredericksburg Road. Diversion to the frontage road may be more beneficial due to the fewer number of signalized intersections on the frontage road.

Researchers performed an investigation of the past experience with the use of changeable message signs (CMSs) on arterials. Guidelines for the use of CMSs on arterials are also presented. Diversion to Fredericksburg is not beneficial during the peak periods, and diversion to frontage roads may be more beneficial during off-peak periods, so the use of CMSs on Fredericksburg Road may not be warranted. However, if the excess capacity in the south section of Fredericksburg is used to divert IH-410 bound traffic, one CMS on the east side of the Fredericksburg interchange with IH-10 is recommended. On the frontage road, one CMS at IH-410/Cherry Ridge interchange is recommended to provide trail blazing information to IH-10 outbound traffic diverted to the frontage road.

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APPENDIX A
EXCESS CAPACITY ANALYSIS FOR FREDERICKSBURG ROAD

TABLE A-1. Excess Capacity on Fredericksburg, South of IH-410 for AM Peak Period

Phase ---->	Arterial									Cross Street									Intersection	Target	Excess
	5		6		1		2		y_c	3		4		7		8		y_c	Y_c	Y_c	Capacity
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.				
Woodlawn	0	1	0	1	0	1	400	1800	0.22	0	1	227	1800	100	1800	175	1800	0.15	0.37	0.88	910
IH-10	102	1705	0	1	0	1	154	1800	0.07	0	1	558	2923	107	561	260	1792	0.34	0.41	0.88	1651
IH-10	0	1	125	1750	364	1705	0	1	0.21	4	42	245	1851	0	1	230	3648	0.20	0.41	0.88	1737
Buckeye	0	1	594	3416	0	1	255	3407	0.17	0	1	82	1597	0	1	79	1581	0.05	0.23	0.9	2299
Fulton/Zaram	12	1710	699	3409	17	1710	207	3429	0.21	0	1	334	3411	0	1	42	3429	0.10	0.31	0.88	1955
Lynwood	0	1	591	3422	0	1	495	3388	0.17	0	1	288	3239	0	1	136	3393	0.09	0.26	0.9	2163
West Avenue	0	1	600	3423	0	1	561	3261	0.18	0	1	34	1800	0	1	227	1800	0.13	0.30	0.9	1952
Hildebrand	0	1	674	3422	0	1	309	3409	0.20	0	1	294	1608	0	1	269	4722	0.18	0.38	0.9	1773
Babcock/Fresno	111	1710	816	4688	0	1	589	4910	0.24	0	1	460	1800	330	1710	279	4910	0.26	0.49	0.85	1745
Vance Jackson	0	1	323	3429	0	1	613	4610	0.13	0	1	0	1	0	1	240	1800	0.13	0.27	0.9	2921
De Chantle	0	1	569	3410	0	1	563	3420	0.17	0	1	157	1800	0	1	30	1800	0.09	0.25	0.9	2209
Williamsburg	0	1	612	3412	0	1	556	3429	0.18	0	1	69	1800	0	1	0	1	0.04	0.22	0.9	2340
Balcones Hts.	0	1	639	4824	75	1710	596	3429	0.22	28	1710	62	3300	0	1	70	1547	0.05	0.26	0.85	2013
Crossroads	0	1	671	4910	165	1710	680	3429	0.29	140	3155	0	1	0	1	0	1	0.04	0.34	0.88	1854
Hillcrest	0	1	690	4910	0	1	763	4910	0.16	0	1	118	1800	127	1710	0	1	0.07	0.23	0.9	3291
IH-410	0	1	718	4211	619	2679	1075	5268	0.44	0	1	140	2524	331	2614	0	1	0.13	0.56	0.88	1677
IH-410	316	2226	938	5268	0	1	1067	4687	0.32	271	3234	0	1	0	1	128	1873	0.08	0.40	0.88	2232

TABLE A-2. Excess Capacity on Fredericksburg, South of IH-410 for Off-Peak Period

Phase ---->	Arterial									Cross Street									Intersection	Target	Excess
	5		6		1		2		y_c	3		4		7		8		y_c			
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Y_c	Y_c	Capacity
Woodlawn	56	300	0	1	0	1	64	1500	0.07	0	1	92	1800	108	1800	144	1800	0.14	0.21	0.88	1212
IH-10	124	2033	0	1	0	1	108	1364	0.07	0	1	284	1800	120	1800	136	1792	0.16	0.23	0.88	2221
IH-10	0	1	128	1750	212	1705	0	1	0.12	0	42	192	1851	0	1	260	3648	0.17	0.30	0.88	2146
Buckeye	0	1	303	3416	0	1	251	3407	0.09	0	1	45	1597	0	1	48	1581	0.03	0.12	0.9	2661
Fulton/Zaram	10	1710	469	3409	10	1710	256	3429	0.14	0	1	223	3411	0	1	30	3429	0.07	0.21	0.88	2302
Lynwood	0	1	358	3422	0	1	436	3388	0.13	0	1	122	3239	0	1	56	3393	0.04	0.17	0.9	2486
West Avenue	0	1	543	3423	0	1	510	3261	0.16	12	1800	42	1800	12	1800	194	1800	0.11	0.27	0.9	2044
Hildebrand	0	1	433	3422	0	1	297	3409	0.13	0	1	103	1608	0	1	191	4722	0.06	0.19	0.9	2418
Babcock/Fresno	72	1710	668	4688	0	1	571	4910	0.18	0	1	188	1800	190	1710	182	4910	0.15	0.33	0.85	2540
Vance Jackson	0	1	447	3429	0	1	714	4610	0.15	0	1	0	1	0	1	332	1800	0.18	0.34	0.9	2585
De Chantle	0	1	482	3410	0	1	581	3420	0.17	0	1	92	1800	0	1	30	1800	0.05	0.22	0.9	2322
Williamsburg	0	1	643	3412	0	1	663	3429	0.19	0	1	67	1800	0	1	0	1	0.04	0.23	0.9	2295
Balcones Hts.	0	1	679	4824	71	1710	631	3429	0.23	31	1710	77	3300	0	1	77	1547	0.05	0.28	0.85	1971
Crossroads	0	1	696	4910	161	1710	728	3429	0.31	150	3155	0	1	0	1	0	1	0.05	0.35	0.88	1804
Hillcrest	0	1	759	4910	0	1	808	4910	0.16	0	1	146	1800	146	1710	0	1	0.09	0.25	0.9	3192
IH-410	0	1	924	4211	672	2679	1296	5268	0.50	0	1	164	2524	340	2614	0	1	0.13	0.63	0.88	1333
IH-410	484	2226	1288	5268	0	1	1152	4687	0.46	308	3234	0	1	0	1	152	1873	0.10	0.56	0.88	1513

TABLE A-3. Excess Capacity on Fredericksburg, South of IH-410 for PM Peak Period

Phase ---->	Arterial									Cross Street									Intersection	Target	Excess
	5		6		1		2		y_c	3		4		7		8		y_c			
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Y_c	Y_c	Capacity
Woodlawn	100	200	0	1	0	1	400	1600	0.28	0	1	100	1800	75	1800	125	1800	0.11	0.39	0.88	884
IH-10	164	2033	0	1	0	1	110	1364	0.08	0	1	333	3190	32	307	260	1792	0.25	0.33	0.88	1868
IH-10	0	1	241	1750	175	1705	0	1	0.14	5	42	190	1851	0	1	414	3648	0.22	0.35	0.88	1941
Buckeye	0	1	358	3416	0	1	480	3407	0.14	0	1	76	1597	0	1	66	1581	0.05	0.19	0.9	2424
Fulton/Zaram	12	1710	556	3409	17	1710	430	3429	0.17	0	1	375	3411	0	1	36	3429	0.11	0.28	0.88	2057
Lynwood	0	1	588	3422	0	1	826	3388	0.24	0	1	189	3239	0	1	127	3393	0.06	0.30	0.9	2026
West Avenue	0	1	543	3423	0	1	923	3261	0.28	0	1	32	1800	0	1	354	1800	0.20	0.48	0.9	1371
Hildebrand	0	1	583	3422	0	1	612	3409	0.18	0	1	150	1608	0	1	482	4722	0.10	0.28	0.9	2108
Babcock/Fresno	182	1710	1034	4688	0	1	850	4910	0.33	0	1	254	1800	300	1710	391	4910	0.26	0.58	0.85	1316
Vance Jackson	0	1	915	3429	0	1	1316	4610	0.29	0	1	0	1	0	1	536	1800	0.30	0.58	0.9	1460
De Chantle	0	1	839	3410	0	1	973	3420	0.28	0	1	103	1800	0	1	81	1800	0.06	0.34	0.9	1909
Williamsburg	0	1	950	3412	0	1	1073	3429	0.31	0	1	87	1800	0	1	0	1	0.05	0.36	0.9	1847
Balcones Hts.	0	1	1155	4824	120	1710	1073	3429	0.38	40	1710	100	3300	0	1	100	1547	0.06	0.45	0.85	1379
Crossroads	0	1	1184	4910	273	1710	1239	3429	0.52	195	3155	0	1	0	1	348	1750	0.20	0.72	0.88	549
Hillcrest	0	1	1291	4910	0	1	1375	4910	0.28	0	1	190	1800	190	1710	0	1	0.11	0.39	0.9	2498
IH-410	0	1	1261	4211	802	2679	1574	5268	0.60	0	1	279	2524	289	2614	0	1	0.11	0.71	0.88	902
IH-410	600	2226	1666	5268	0	1	1263	4687	0.59	397	3234	0	1	0	1	230	1873	0.12	0.71	0.88	803

TABLE A-4. Excess Capacity on Fredericksburg, North of IH-410 for AM Peak Period

Phase ---->	Arterial									Cross Street											
	5		6		1		2			3		4		7		8			Intersection	Target	Excess
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	y_c	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	y_c	Y_c	Y_c	Capacity
Woodlake	45	1710	1105	4891	13	1710	1409	4886	0.30	10	1710	59	3083	10	1710	34	3193	0.02	0.32	0.85	2585
Lakeridge	6	1710	980	4910	9	1710	1391	4830	0.29	0	1710	19	3190	0	1710	36	1800	0.02	0.31	0.88	2737
Magic	0	1	805	5200	12	1805	1169	5200	0.23	131	1710	0	1	0	1	24	1800	0.08	0.31	0.9	3078
Callaghan	36	1805	1417	5400	294	1805	1823	5400	0.50	55	1805	538	3571	340	1805	283	3600	0.27	0.77	0.85	446
Mockingbird	10	1805	966	5200	22	1805	1519	5200	0.30	0	1	18	1481	0	1	14	1516	0.01	0.32	0.9	3034
Chamber	0	1	921	5200	47	1805	1365	5200	0.29	34	1710	0	1	0	1	95	1800	0.05	0.34	0.9	2905
Louis Pasteur	323	1800	987	5200	3	1805	1096	5200	0.37	30	1710	124	1800	37	1710	30	1750	0.09	0.46	0.85	2050
Medical	458	1805	1600	5400	93	1805	1296	5400	0.55	165	1805	546	3403	139	1805	451	3522	0.25	0.80	0.85	260
Data Point	79	1710	995	5200	68	1710	798	5200	0.24	176	1710	58	1750	58	1750	61	1800	0.14	0.37	0.85	2477
Wurzbach	135	1805	2115	6000	259	1805	745	5400	0.43	162	1805	713	4000	301	1805	921	4000	0.40	0.82	0.85	139
Bluemel	18	1805	1445	5155	237	1805	746	5138	0.29	37	1805	116	1805	21	1805	1	1805	0.08	0.38	0.85	2440
Cinnamon	61	1710	2626	6000	28	1805	751	5400	0.47	19	1710	400	2000	200	2000	18	1800	0.21	0.68	0.85	894
Gus Eckhert	27	1805	1622	5156	1	1805	715	5200	0.33	10	1805	75	1805	50	1805	10	1805	0.05	0.38	0.88	2617
USAA Blvd	500	1800	2445	6000	600	1800	997	5400	0.69	56	1710	940	4000	62	1805	33	3400	0.27	0.95	0.85	0
USAA G2	0	1	2275	5400	344	1805	669	5400	0.42	10	1805	0	1	0	1	17	1805	0.01	0.43	0.9	2534
Huebner	26	1805	1936	6000	486	1900	270	5200	0.34	443	1900	1023	5200	46	1805	996	3600	0.43	0.77	0.85	432

TABLE A-5. Excess Capacity on Fredericksburg, North of IH-410 for Off-Peak Period

	Arterial									Cross Street									Intersection	Target	Excess
Phase ---->	5		6		1		2		y_c	3		4		7		8		y_c			
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.		Y_c	Y_c	Capacity
Woodlake	64	1710	1045	4891	21	1710	1257	4886	0.27	10	1710	84	3083	10	1710	85	3193	0.03	0.30	0.85	2677
Lakeridge	14	1710	871	4910	35	1710	1000	4830	0.23	0	1710	33	3190	0	1710	90	1800	0.05	0.28	0.88	2910
Magic	0	1	746	4904	23	1710	820	4847	0.18	0	1805	3	5000	0	1	102	1800	0.06	0.24	0.9	3202
Callaghan	34	1710	787	4791	207	1710	782	4833	0.28	193	1750	152	1750	152	1750	339	2500	0.22	0.51	0.85	1666
Mockingbird	18	1710	967	4910	11	1710	1051	4896	0.22	0	1	20	1481	0	1	31	1516	0.02	0.24	0.9	3224
Chambers	0	1	960	4910	21	1710	958	4891	0.21	0	1	3	5000	0	1	74	1800	0.04	0.25	0.9	3183
Louis Pasteur	235	1710	801	4872	9	1710	839	4910	0.30	30	1710	150	1800	57	1710	30	1750	0.10	0.40	0.85	2196
Medical	249	1710	544	4823	30	1710	543	4804	0.26	118	1710	145	1800	59	1710	135	3262	0.15	0.41	0.85	2124
Data Point	52	1710	579	4879	30	1710	518	4749	0.15	111	3400	32	1750	32	1750	35	1800	0.05	0.20	0.85	3087
Wurzbach	99	1710	576	4910	145	1710	388	4910	0.18	167	1710	508	3429	95	1710	620	3429	0.25	0.42	0.85	2106
Cinamon Creek	63	1710	614	4761	3	1710	377	4828	0.17	17	1710	105	1750	61	1710	10	1800	0.07	0.24	0.85	2966
Mira Mesa	12	1710	485	4864	45	1750	336	4910	0.11	19	1710	49	3600	12	1710	59	1750	0.04	0.15	0.85	3449
Huebner	49	1805	373	4766	151	1710	249	4910	0.14	1	1710	373	1800	54	1710	273	1800	0.21	0.35	0.88	2618

TABLE A-6. Excess Capacity on Fredericksburg, North of IH-410 for PM Peak Period

Phase ---->	Arterial									Cross Street									Intersection	Target	Excess
	5		6		1		2			3		4		7		8					
	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	y_c	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	Vol.	Sat.	y_c	Y_c	Y_c	Capacity
Woodlake	81	1710	1554	5100	30	1710	1222	5100	0.35	10	1710	119	3083	10	1710	224	3193	0.08	0.43	0.85	2152
Lakeridge	26	1710	1425	5100	72	1710	1296	5100	0.30	0	1710	43	3190	0	1710	240	1800	0.13	0.43	0.88	2297
Magic	0	1	1434	5100	31	1710	1136	5100	0.28	165	1805	0	1	0	1	203	1805	0.11	0.39	0.9	2582
Callaghan	96	1805	2504	5400	452	1805	1236	5400	0.52	112	1805	640	3521	232	1805	420	3600	0.25	0.76	0.85	475
Mockingbird	5	1710	1735	5100	10	1710	1223	5100	0.34	0	1	7	1481	0	1	112	1516	0.07	0.42	0.9	2463
Chamber	0	1	1691	5100	47	1710	1332	5100	0.33	42	1805	0	1	0	1	42	1805	0.02	0.35	0.9	2780
Louis Pasteur	108	1800	1474	5100	2	1805	1064	5100	0.35	30	1710	250	1800	191	1710	30	1750	0.16	0.51	0.85	1757
Medical	304	1805	2076	6000	132	1710	1412	6000	0.51	176	1900	972	4000	508	1805	524	4000	0.41	0.93	0.85	0
Data Point	31	1710	955	5100	42	1710	1426	5100	0.30	299	3400	162	1750	162	1750	165	1800	0.18	0.49	0.85	1844
Wurzbach	268	1900	1504	6000	228	1900	1756	6000	0.41	288	1900	980	4000	504	1900	968	4000	0.51	0.92	0.85	0
Bluemel	4	1805	1744	5120	260	1805	940	5120	0.34	36	900	56	1805	20	1805	36	900	0.07	0.41	0.85	2233
Cinnamon Creek	196	1805	1396	5105	1	1805	1508	5400	0.38	568	1800	112	1800	208	1800	376	1800	0.38	0.76	0.85	487
Gus Eckhert	148	1805	1189	5124	0	1	1700	5124	0.33	1	1805	72	900	52	900	1	1805	0.08	0.41	0.88	2396
USAA Blvd	168	1805	1312	5159	44	1805	1704	5400	0.35	334	1800	120	3149	88	1800	334	1800	0.23	0.58	0.85	1448
USAA G2	0	1	1180	5700	6	1805	900	5100	0.21	48	3500	0	1	0	1	416	2000	0.21	0.42	0.9	2473
Huebner	100	1805	800	5400	384	1805	1480	5400	0.49	264	1805	932	5156	100	1805	1120	3800	0.35	0.84	0.85	70

APPENDIX B
EXCESS CAPACITY ANALYSIS FOR IH-410 INTERCHANGES

TABLE B-1. Existing Conditions of Interchanges on IH-10 for AM Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	100	375	300	1124	24	90	125	1605	90	1710	270	1800	0.4169	88.5	29.6	19.9
Fredericksburg	102	1710	154	1800	0	0	131	1472	100	623	509	2858	0.3351	34.9	37.2	65.9
Fulton	40	1710	200	1648	16	132	136	2667	35	532	200	3041	0.2381	56.5	51.1	30.4
Hildebrand	110	1710	139	1800	232	1530	383	2570	155	574	810	2997	0.5707	25.7	66.3	46.0
Fresno	71	1710	99	1800	200	1530	471	2293	94	567	498	3005	0.5019	31.8	65.7	40.5
West Avenue	225	2373	105	1108	338	1530	828	3368	87	473	559	3103	0.6507	42.0	61.0	35.0
Vance Jackson	264	1710	392	2426	164	1015	712	3322	317	1394	486	2136	0.6033	30.6	64.1	43.3
Crossroads	392	2534	315	2036	94	608	383	2739	408	1608	485	1912	0.5483	30.3	57.8	49.9
Callaghan	1007	3231	220	1756	156	1493	1022	4842	284	1668	705	3512	0.6930	62.0	42.2	33.8
Wurzbach	932	3059	437	1434	164	538	1371	6373	311	1720	626	3462	0.7008	59.2	43.8	35.0
Huebner	642	3111	180	872	201	974	1757	5268	268	1668	618	3512	0.7006	40.6	65.9	31.5

	Total Interchange Delay (Veh-Hrs/Hr)	Phase Sequence	Internal Offset
Woodlawn	12.60	Lag-Lead	9
Fredericksburg	18.04	Lead-Lead	34
Fulton	9.20	Lag-Lead	1
Hildebrand	41.06	Lead-Lead	20
Fresno	23.00	Lead-Lead	28
West Avenue	36.44	Lag-Lead	0
Vance Jackson	51.97	Lead-Lead	18
Crossroads	61.74	Lead-Lead	12
Callaghan	180.95	Lead-Lead	46
Wurzbach	87.89	Lead-Lead	35
Huebner	109.48	Lead-Lead	32

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	25	1710	200	1800	0.2508	53.1	79.2	5.7
Fredericksburg	364	1668	125	1494	19	227	245	2396	4	60	229	3449	0.3871	59.0	61.2	17.8
Fulton	65	1668	56	862	50	770	170	3256	36	711	140	2766	0.1678	43.3	60.9	33.8
Hildebrand	517	3274	156	1320	49	415	448	2941	83	601	410	2969	0.4483	37.9	66.6	33.5
Fresno	341	3264	80	1594	9	179	251	3600	339	2175	203	1316	0.3301	40.8	36.3	60.9
West Avenue	173	2391	98	1355	93	1286	470	3404	328	1104	725	2441	0.5075	16.3	54.6	67.1
Vance Jackson	161	1710	327	2018	221	1364	642	2993	200	730	776	2833	0.6505	31.6	53.1	53.3
Crossroads	196	1710	604	1800	408	1530	697	2721	94	434	681	3144	0.8083	54.4	48.6	35.0
Callaghan	317	1668	945	3141	97	322	672	5268	680	1736	1349	3445	0.8205	37.3	52.2	48.5
Wurzbach	183	1668	846	2864	166	562	754	5268	788	3231	1515	3512	0.7268	51.5	44.2	42.3
Huebner	241	1668	781	3378	27	117	645	5268	963	2054	1436	3112	0.8225	32.9	38.4	66.7

TABLE B-2. Excess Capacity with Existing Geometry for Interchanges on IH-10 for AM Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	100	137	1050	1438	24	33	125	1605	90	1710	270	1800	0.8802	114.8	13.9	9.3	29.03	Lag-Lead	19
Fredericksburg	102	1710	1104	1800	0	0	131	1472	100	623	509	2858	0.8628	98.1	14.4	25.5	38.24	Lead-Lead	72
Fulton	40	1710	1200	1772	16	24	136	2667	35	532	200	3041	0.7940	109.7	17.8	10.5	24.25	Lag-Lead	19
Hildebrand	110	1710	489	2324	232	1102	383	2570	155	574	810	2997	0.6296	33.3	61.8	42.9	55.01	Lead-Lead	21
Fresno	71	1710	1349	3074	200	456	471	2293	94	567	498	3005	0.8100	69.4	42.4	26.2	53.30	Lead-Lead	44
West Avenue	225	1710	930	2535	338	921	828	3368	87	473	559	3103	0.7968	58.2	50.7	29.1	58.91	Lag-Lead	11
Vance Jackson	264	1710	817	2923	164	587	712	3322	317	1394	486	2136	0.7212	45.6	55.2	37.2	65.61	Lead-Lead	24
Crossroads	392	1710	740	3140	94	399	383	2739	408	1608	485	1912	0.6292	41.4	51.9	44.7	72.82	Lead-Lead	27
Callaghan	1007	3231	220	1756	156	1493	1022	4842	284	1668	705	3512	0.6930	62.0	42.2	33.8	180.95	Lead-Lead	46
Wurzbach	932	2000	1287	2762	164	352	1371	6373	311	1720	626	3462	0.8619	74.0	35.5	28.5	118.58	Lead-Lead	33
Huebner	642	3111	180	872	201	974	1757	5268	268	1668	618	3512	0.7006	40.6	65.9	31.5	109.48	Lead-Lead	32

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	25	1710	200	1800	0.2508	53.1	79.2	5.7
Fredericksburg	364	1668	125	1494	19	227	245	2396	4	60	229	3449	0.3871	59.0	61.2	17.8
Fulton	65	1668	56	862	50	770	170	3256	36	711	140	2766	0.1678	43.3	60.9	33.8
Hildebrand	517	3274	156	1320	49	415	448	2941	83	601	410	2969	0.4483	37.9	66.6	33.5
Fresno	341	3264	80	1594	9	179	251	3600	339	2175	203	1316	0.3301	40.8	36.3	60.9
West Avenue	173	2391	98	1355	93	1286	470	3404	328	1104	725	2441	0.5075	16.3	54.6	67.1
Vance Jackson	161	1710	327	2018	221	1364	642	2993	200	730	776	2833	0.6505	31.6	53.1	53.3
Crossroads	196	1710	604	1800	408	1530	697	2721	94	434	681	3144	0.8083	54.4	48.6	35.0
Callaghan	317	1668	945	3141	97	322	672	5268	680	1736	1349	3445	0.8205	37.3	52.2	48.5
Wurzbach	183	1668	846	2864	166	562	754	5268	788	3231	1515	3512	0.7268	51.5	44.2	42.3
Huebner	241	1668	781	3378	27	117	645	5268	963	2054	1436	3112	0.8225	32.9	38.4	66.7

TABLE B-3. Excess Capacity with Geometric Improvements for Interchanges on IH-10 for AM Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	100	137	1050	1438	24	33	125	1605	90	1710	270	1800	0.8802	114.8	13.9	9.3	29.03	Lag-Lead	19
Fredericksburg	102	1710	1054	1800	0	0	131	1472	109	1668	509	1756	0.8754	92.6	27.5	17.9	34.87	Lag-Lag	138
Fulton	40	1710	1225	1774	16	23	136	2667	35	1710	200	1800	0.8068	116.4	18.3	3.3	21.77	Lag-Lead	12
Hildebrand	110	1710	1129	2910	232	598	383	2570	155	1710	810	1800	0.8380	61.6	62.1	14.3	64.75	Lag-Lag	121
Fresno	71	1710	1674	3164	200	378	471	2293	94	1710	498	1800	0.8058	85.8	43.5	8.7	53.80	Lag-Lag	136
West Avenue	225	1710	1355	2795	338	697	828	3368	87	1710	559	1800	0.7955	78.4	51.6	8.0	58.27	Lag-Lead	23
Vance Jackson	264	1710	1067	3058	164	470	712	3322	317	1710	486	1800	0.7486	55.1	53.5	29.4	68.33	Lead-Lag	100
Crossroads	392	1710	315	2677	94	799	383	2739	408	1710	485	1800	0.6077	41.5	53.3	43.2	53.68	Lead-Lead	44
Callaghan	1007	3231	220	1756	156	1493	1022	4842	284	1668	705	3512	0.6930	41.5	53.3	43.2	180.95	Lead-Lead	46
Wurzbach	932	2000	1287	2762	164	352	1371	6373	311	1720	626	3462	0.8619	74.0	35.5	28.5	118.58	Lead-Lead	33
Huebner	642	3111	180	872	201	974	1757	5268	268	1668	618	3512	0.7006	40.6	65.9	31.5	109.48	Lead-Lead	32

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	25	1710	200	1800	0.2508	53.1	79.2	5.7
Fredericksburg	364	1668	125	1494	19	227	245	2396	4	60	229	3449	0.3871	59.0	61.2	17.8
Fulton	65	1668	56	862	50	770	170	3256	36	1668	140	1756	0.1447	50.4	71.1	16.5
Hildebrand	517	1690	156	2641	49	830	448	2941	83	1710	410	1800	0.5337	67.1	60.4	10.5
Fresno	341	1685	80	3187	9	358	251	3600	339	1710	203	1800	0.4703	56.7	25.8	55.5
West Avenue	173	1681	98	1712	93	1625	470	3404	328	1710	725	1800	0.5057	26.5	62.2	49.3
Vance Jackson	161	1710	327	2018	221	1364	642	2993	200	1710	776	1800	0.5932	37.7	70.4	29.9
Crossroads	196	1710	604	1800	408	1530	697	2721	94	1710	681	1800	0.7139	65.0	62.0	11.0
Callaghan	317	1668	945	3141	97	322	672	5268	680	1736	1349	3445	0.8205	37.3	52.2	48.5
Wurzbach	183	1668	846	2864	166	562	754	5268	788	3231	1515	3512	0.7268	51.5	44.2	42.3
Huebner	241	1668	781	3378	27	117	645	5268	963	2054	1436	3112	0.8225	32.9	38.4	66.7

TABLE B-4. Existing Conditions of Interchanges on IH-10 for Off-Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	sat.flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	56	602	64	688	24	258	144	1607	96	1710	104	1800	0.2388	53.7	52.0	32.3	7.86	Lag-Lead	19
Fredericksburg	124	1874	108	1632	0	0	136	1756	120	1028	284	2433	0.2604	26.7	64.3	47.0	13.05	Lead-Lead	9
Fulton	20	1710	76	1077	44	624	64	2526	35	1014	105	2535	0.1304	62.3	45.5	30.2	4.83	Lead-Lag	133
Hildebrand	112	1756	112	1756	128	1530	344	2384	92	591	494	2979	0.3836	20.4	79.4	38.2	25.03	Lead-Lead	12
Fresno	104	2323	52	1161	100	1530	360	2435	39	355	354	3227	0.3231	24.8	79.4	33.8	14.53	TTI-Lead	18
West Avenue	164	2146	103	1347	330	1530	595	3087	138	677	533	2889	0.6123	43.1	54.2	40.7	30.45	Lag-Lead	148
Vance Jackson	216	1710	401	2369	180	1064	521	3243	260	1270	498	2266	0.5346	39.7	50.3	48.0	46.86	Lead-Lead	34
Crossroads	392	3568	140	1274	36	328	188	2497	132	916	380	2638	0.3293	37.9	49.9	50.2	27.76	Lead-Lead	24
Callaghan	685	3231	216	1756	160	493	539	4279	224	1668	493	3512	0.5848	62.0	36.9	39.1	47.71	Lead-Lead	51
Wurzbach	860	2942	379	1296	223	763	1159	6030	388	1668	997	3512	0.7173	52.3	44.3	41.4	76.67	Lead-Lead	29
Huebner	624	3138	156	784	204	1026	700	5268	316	1668	720	3512	0.5213	51.9	36.5	49.6	44.04	Lag-Lead	138

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	sat. flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	12	1710	104	1246	40	479	188	2825	56	1710	144	1800	0.1828	56.1	60.3	21.6
Fredericksburg	212	1668	128	1585	12	149	192	3060	1	14	259	3498	0.2613	67.1	33.4	37.5
Fulton	52	1668	24	639	36	959	88	3372	24	989	60	2474	0.0879	54.9	47.5	35.6
Hildebrand	336	3032	128	1155	96	866	244	2894	156	781	300	2780	0.3949	33.6	43.8	60.6
Fresno	204	3233	44	1267	16	461	184	3600	228	1727	236	1787	0.2462	24.0	31.8	82.2
West Avenue	183	2822	56	864	87	1342	487	3453	247	1153	512	2389	0.4201	18.5	59.4	60.1
Vance Jackson	165	1710	348	1702	334	1634	593	2964	213	891	524	2664	0.6436	38.6	54.2	45.2
Crossroads	180	1710	348	1775	308	1571	332	2975	88	542	492	3031	0.4700	51.1	44.6	42.3
Callaghan	123	1668	449	2248	218	1092	594	5268	307	1668	917	3512	0.4965	38.6	63.8	35.6
Wurzbach	223	1936	285	2474	76	660	1162	5268	535	3231	1484	3512	0.5377	29.5	65.4	43.1
Huebner	140	1668	252	2715	64	690	896	5268	210	1668	1114	3512	0.4100	25.3	78.2	34.5

TABLE B-5. Excess Capacity with Existing Geometry for Interchanges on IH-10 for Off-Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	sat.flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	56	79	1064	1498	24	34	144	1607	96	1710	104	1800	0.8560	114.5	14.6	8.9	23.71	Lag-Lead	23
Fredericksburg	124	1710	1058	1800	0	0	136	1756	120	1028	284	2433	0.7820	94.1	25.4	18.5	31.41	Lag-Lag	132
Fulton	20	1710	1301	1732	44	59	64	2526	35	1014	105	2535	0.8110	124.1	8.4	5.5	16.61	Lag-Lead	17
Hildebrand	112	1710	1262	3223	128	327	344	2384	92	591	494	2979	0.6915	62.1	51.3	24.6	54.23	Lead-Lead	45
Fresno	104	1710	1852	3389	100	183	360	2435	39	355	354	3227	0.8042	87.3	33.3	17.4	44.77	Lead-Lead	36
West Avenue	164	1710	1053	2643	330	828	595	3087	138	677	533	2889	0.7951	63.0	42.9	32.1	55.84	Lead-Lead	34
Vance Jackson	216	1710	1426	3142	180	397	521	3243	260	1270	498	2266	0.8192	71.9	33.8	32.3	71.64	Lag-Lead	149
Crossroads	392	1710	2060	3529	36	62	188	2497	132	916	380	2638	0.8031	92.5	22.7	22.8	55.18	Lead-Lead	46
Callaghan	685	1668	2166	3512	160	1493	539	4279	224	1668	493	3512	0.8770	97.1	19.8	21.1	79.19	Lag-Lead	16
Wurzbach	860	2221	879	2270	223	576	1159	6030	388	1668	997	3512	0.8120	61.6	39.5	36.9	96.14	Lead-Lead	38
Huebner	624	1668	1681	3081	204	374	700	5268	316	1668	720	3512	0.8679	86.2	22.0	29.8	76.46	Lag-Lead	16

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	sat. flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	12	1710	104	1246	40	479	188	2825	56	1710	144	1800	0.1828	56.1	60.3	21.6
Fredericksburg	212	1668	128	1585	12	149	192	3060	1	14	259	3498	0.2613	67.1	33.4	37.5
Fulton	52	1668	24	639	36	959	88	3372	24	989	60	2474	0.0879	54.9	47.5	35.6
Hildebrand	336	3032	128	1155	96	866	244	2894	156	781	300	2780	0.3949	33.6	43.8	60.6
Fresno	204	3233	44	1267	16	461	184	3600	228	1727	236	1787	0.2462	31.2	41.2	65.6
West Avenue	183	2822	56	864	87	1342	487	3453	247	1153	512	2389	0.4201	18.5	59.4	60.1
Vance Jackson	165	1710	348	1702	334	1634	593	2964	213	891	524	2664	0.6436	38.6	54.2	45.2
Crossroads	180	1710	348	1775	308	1571	332	2975	88	542	492	3031	0.4700	51.1	44.6	42.3
Callaghan	123	1668	449	2248	218	1092	594	5268	307	1668	917	3512	0.4965	38.6	63.8	35.6
Wurzbach	223	1936	285	2474	76	660	1162	5268	535	3231	1484	3512	0.5377	29.5	65.4	43.1
Huebner	140	1668	252	2715	64	690	896	5268	210	1668	1114	3512	0.4100	25.3	78.2	34.5

TABLE B-6. Excess Capacity with Geometric Improvements for Interchanges on IH-10 for Off-Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	sat.flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	56	79	1064	1498	24	34	144	1607	96	1710	104	1800	0.8560	114.5	14.6	8.9
Fredericksbu	124	1710	1133	1800	0	0	136	1756	120	1668	284	1756	0.7912	101.2	25.5	11.3
Fulton	20	1710	1301	1732	44	59	64	2526	35	1710	105	1800	0.8095	126.1	8.6	3.3
Hilderbrand	112	1710	1587	3294	128	266	344	2384	92	1710	494	1800	0.7562	77.5	52.0	8.5
Fresno	104	1710	2077	3411	100	164	360	2435	39	1710	354	1800	0.8064	100.1	34.3	3.6
West Avenue	164	1710	1453	2852	330	648	595	3087	138	1710	533	1800	0.8056	81.7	43.5	12.8
Vance Jacks	216	1710	1601	3187	180	358	521	3243	260	1710	498	1800	0.8155	79.8	34.0	24.2
Crossroads	392	1710	2285	3536	36	56	188	2497	132	1710	380	1800	0.8573	102.9	22.9	12.2
Callaghan	685	1668	2166	3512	160	1493	539	4279	224	1668	493	3512	0.8770	97.1	19.8	21.1
Wurzbach	860	2221	879	2270	223	576	1159	6030	388	1668	997	3512	0.8120	61.6	39.5	36.9
Huebner	624	1668	1681	3081	204	374	700	5268	316	1668	720	3512	0.8679	86.2	22.0	29.8

	Total Interchange Delay (Veh-Hrs/Hr)	Phase Sequence	Internal Offset
Woodlawn	23.71	Lag-Lead	23
Fredericksbu	31.34	Lag-Lag	143
Fulton	15.26	Lag-Lag	6
Hilderbrand	53.76	Lag-Lag	126
Fresno	41.79	Lead-Lead	63
West Avenue	56.53	Lag-Lead	8
Vance Jacks	67.30	Lag-Lead	1
Crossroads	53.68	Lag-Lead	5
Callaghan	79.19	Lag-Lead	16
Wurzbach	96.14	Lead-Lead	38
Huebner	76.46	Lag-Lead	16

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	sat. flow	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	12	1710	104	1246	40	479	188	2825	56	1710	144	1800	0.1828	56.1	60.3	21.6
Fredericksbu	212	1668	128	1585	12	149	192	3060	1	14	259	3498	0.2613	67.1	33.4	37.5
Fulton	52	1668	24	639	36	959	88	3372	24	1668	60	1756	0.0780	61.4	53.3	23.3
Hilderbrand	336	1680	128	1925	96	144	244	2894	156	1710	300	1800	0.8422	63.4	45.8	28.8
Fresno	204	1669	44	2534	16	922	184	3600	228	1710	236	1800	0.3067	49.7	34.0	54.3
West Avenue	183	1701	56	1800	87	1530	487	3453	247	1710	512	1800	0.3931	32.8	62.6	42.6
Vance Jacks	165	1710	348	1702	334	1634	593	2964	213	1710	524	1800	0.5291	45.8	64.4	27.8
Crossroads	180	1710	348	1775	308	1571	332	2975	88	1710	492	1800	0.4694	57.7	61.9	18.4
Callaghan	123	1668	449	2248	218	1092	594	5268	307	1668	917	3512	0.4965	38.6	63.8	35.6
Wurzbach	223	1936	285	2474	76	660	1162	5268	535	3231	1484	3512	0.5377	29.5	65.4	43.1
Huebner	140	1668	252	2715	64	690	896	5268	210	1668	1114	3512	0.4100	25.3	78.2	34.5

TABLE B-7. Existing Conditions of Interchanges on IH-10 for PM Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	100	278	450	1251	24	67	144	1607	90	1710	270	1800	0.5097	97.6	25.7	14.7	14.17	Lag-Lead	9
Fredericksburg	124	1858	110	1649	0	0	260	1792	32	307	333	3190	0.3161	28.8	63.7	45.5	16.71	Lead-Lag	135
Fulton	40	1710	150	1603	16	171	136	2667	35	532	200	3041	0.2104	48.0	56.4	33.6	8.52	Lead-Lag	133
Hildebrand	202	1908	169	1597	218	1530	596	2624	114	552	720	3020	0.5761	23.6	80.1	34.3	47.28	Lead-Lead	16
Fresno	238	1860	264	2063	153	1196	632	2627	153	738	586	2825	0.5759	26.6	67.9	43.5	34.38	Lead-Lead	17
West Avenue	136	1710	223	1800	557	1530	1011	3202	70	406	548	3174	0.8522	51.7	61.9	24.4	67.70	Lag-Lead	8
Vance Jackson	380	1710	421	1866	337	1494	634	3225	260	1103	576	2442	0.6579	39.3	57.6	41.1	68.30	Lead-Lead	35
Crossroads	392	2534	315	2036	94	608	383	2739	408	1608	485	1912	0.5483	30.3	57.8	49.9	61.74	Lead-Lead	12
Callaghan	790	3565	338	1525	224	1493	667	4394	465	1668	1034	3512	0.6522	46.8	32.1	59.1	98.15	Lead-Lead	17
Wurzbach	778	2877	455	1683	132	488	1167	6282	415	1668	1254	3512	0.7051	48.6	44.6	44.8	87.46	Lead-Lead	47
Huebner	633	2866	111	503	483	1493	962	5268	650	1821	1198	3356	0.8631	51.8	29.2	57.0	91.97	Lead-Lead	12

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	24	1710	220	1800	0.2505	53.1	79.2	5.7
Fredericksburg	175	1668	241	1646	14	96	190	2600	10	91	374	3417	0.3294	45.4	58.6	34.0
Fulton	65	1668	56	862	50	770	170	3256	36	711	140	2766	0.1678	43.3	60.9	33.8
Hildebrand	540	3463	160	1026	94	603	288	2715	200	891	598	2664	0.4865	37.5	46.9	53.6
Fresno	502	3279	102	1238	40	486	237	3521	365	1479	505	2047	0.4672	44.4	22.1	71.5
West Avenue	238	2891	72	875	93	1130	380	3411	401	1237	746	2301	0.5179	18.7	44.9	74.4
Vance Jackson	237	1710	395	1754	358	1589	599	2982	150	529	864	3045	0.7097	39.0	49.8	49.2
Crossroads	196	1710	604	1800	408	1530	697	2721	94	434	681	3144	0.8083	54.4	48.6	35.0
Callaghan	479	1668	1092	3177	100	291	1020	5268	364	1668	1093	3512	0.7556	52.1	52.7	33.2
Wurzbach	421	1668	578	3115	64	345	1248	4648	482	3231	1463	3512	0.6701	50.5	57.7	29.8
Huebner	291	2234	324	2488	49	376	1557	5268	493	1430	1156	3767	0.7706	24.5	55.8	57.7

TABLE B-8. Excess Capacity with Existing Geometry for Interchanges on IH-10 for PM Peak Period

Right Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	100	137	1050	1438	24	33	144	1607	90	1710	270	1800	0.8802	114.8	14.8	8.4
Fredericksburg	124	1710	1110	1800	0	0	260	1792	32	307	333	3190	0.8660	98.3	23.2	16.5
Fulton	40	1710	1200	1772	16	24	136	2667	35	532	200	3041	0.7940	109.7	17.8	10.5
Hildebrand	202	1710	394	2194	218	1214	596	2624	114	552	720	3020	0.6132	28.5	76.7	32.8
Fresno	238	1710	1039	3078	153	453	632	2627	153	738	586	2825	0.7856	53.7	51.4	32.9
West Avenue	136	1710	223	1800	557	1530	1011	3202	70	406	548	3174	0.8522	51.7	61.9	24.4
Vance Jackson	380	1710	536	2083	337	1309	634	3225	260	1103	576	2442	0.6898	43.1	55.4	39.5
Crossroads	392	1710	940	3228	94	323	383	2739	408	1608	485	1912	0.6848	48.0	48.4	41.6
Callaghan	790	1796	1488	3382	224	1493	667	4394	465	1668	1034	3512	0.8706	69.7	24.1	44.2
Wurzbach	778	2076	1055	2682	132	352	1167	6282	415	1668	1254	3512	0.8279	59.5	39.2	39.3
Huebner	633	1921	511	1551	483	1466	962	5268	650	1821	1198	3356	0.8691	52.4	29.0	56.6

	Total Interchange Delay (Veh-Hrs/Hr)	Phase Sequence	Internal Offset
Woodlawn	31.08	Lag-Lead	9
Fredericksburg	38.27	Lag-Lag	143
Fulton	24.25	Lag-Lead	19
Hildebrand	56.89	Lead-Lead	22
Fresno	57.38	Lead-Lead	29
West Avenue	67.70	Lag-Lead	8
Vance Jackson	71.34	Lead-Lead	35
Crossroads	78.57	Lead-Lead	25
Callaghan	132.62	Lead-Lead	48
Wurzbach	109.43	Lead-Lead	51
Huebner	107.38	Lead-Lead	7

Left Intersection

	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	24	1710	220	1800	0.2505	53.1	79.2	5.7
Fredericksburg	175	1668	241	1646	14	96	190	2600	10	91	374	3417	0.3294	45.4	58.6	34.0
Fulton	65	1668	56	862	50	770	170	3256	36	711	140	2766	0.1678	43.3	60.9	33.8
Hildebrand	540	3463	160	1026	94	603	288	2715	200	891	598	2664	0.4865	37.5	46.9	53.6
Fresno	502	3279	102	1238	40	486	237	3521	365	1479	505	2047	0.4672	44.4	22.1	71.5
West Avenue	238	2891	72	875	93	1130	380	3411	401	1237	746	2301	0.5179	18.7	44.9	74.4
Vance Jackson	237	1710	395	1754	358	1589	599	2982	150	529	864	3045	0.7097	39.0	49.8	49.2
Crossroads	196	1710	604	1800	408	1530	697	2721	94	434	681	3144	0.8083	54.4	48.6	35.0
Callaghan	479	1668	1092	3177	100	291	1020	5268	364	1668	1093	3512	0.7556	52.1	52.7	33.2
Wurzbach	421	1668	578	3115	64	345	1248	4648	482	3231	1463	3512	0.6701	50.5	57.7	29.8
Huebner	291	2234	324	2488	49	376	1557	5268	493	1430	1156	3767	0.7706	24.5	55.8	57.7

TABLE B-9. Excess Capacity with Geometric Improvements for Interchanges on IH-10 for PM Peak Period

	Right Intersection																		
	Frontage Road						Arterial		Interior				Y	g	g	g interior	Total Interchange	Phase	Internal
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left	Delay (Veh-Hrs/Hr)	Sequence	Offset
Woodlawn	100	137	1050	1438	24	33	144	1607	90	1710	270	1800	0.8802	114.8	14.8	8.4	31.05	Lag-Lead	4
Fredericksburg	124	1710	1235	1800	0	0	260	1792	32	1668	333	1756	0.8757	108.5	26.2	3.3	34.75	Lag-Lag	144
Fulton	40	1710	1225	1774	16	23	136	2667	35	1710	200	1800	0.8068	18.3	116.4	3.3	21.77	Lag-Lead	12
Hildebrand	202	1710	869	2792	218	700	596	2624	114	1710	720	1800	0.7114	50.1	77.4	10.5	62.80	Lead-Lead	48
Fresno	238	1710	1464	3213	153	336	632	2627	153	1710	586	1800	0.7857	72.5	51.4	14.1	62.98	Lead-Lead	50
West Avenue	136	1710	673	1836	557	1519	1011	3202	70	1710	548	1800	0.7234	60.0	71.5	6.5	57.13	Lag-Lead	146
Vance Jackson	380	1710	421	1866	337	1494	634	3225	260	1710	576	1800	0.5743	43.9	64.4	29.7	56.96	Lead-Lead	70
Crossroads	392	1710	315	2677	94	799	383	2739	408	1710	485	1800	0.6077	41.5	53.3	43.2	53.92	Lead-Lead	34
Callaghan	790	1796	1488	3382	224	1493	667	4394	465	1668	1034	3512	0.8706	69.7	24.1	44.2	132.62	Lead-Lead	48
Wurzbach	778	2076	1055	2682	132	352	1167	6282	415	1668	1254	3512	0.8279	59.5	39.2	39.3	109.43	Lead-Lead	51
Huebner	633	1921	511	1551	483	1466	962	5268	650	1821	1198	3356	0.8691	52.4	29.0	56.6	107.38	Lead-Lead	7

	Left Intersection															
	Frontage Road						Arterial		Interior				Y	g	g	g interior
	left	sat. flow	thru	sat.flow	right	thru	thru	sat. flow	left	sat.flow	thru	sat.flow		frontage	arterial	left
Woodlawn	40	1668	156	1216	60	468	320	2965	24	1710	220	1800	0.2505	53.1	79.2	5.7
Fredericksburg	175	1668	241	1646	14	96	190	2600	10	1668	374	1756	0.3594	56.4	79.7	1.9
Fulton	65	1668	56	862	50	770	170	3256	36	1668	140	1756	0.1447	50.4	71.1	16.5
Hildebrand	540	1691	160	2142	94	1258	288	2715	200	1710	598	1800	0.6516	67.7	44.2	26.1
Fresno	502	1693	102	2477	40	971	237	3521	365	1710	505	1800	0.5773	69.8	18.0	50.2
West Avenue	238	1615	72	1800	93	1530	380	3411	401	1710	746	1800	0.5618	35.1	46.8	56.1
Vance Jackson	237	1710	395	1754	358	1589	599	2982	150	1710	864	1800	0.7053	44.0	72.2	21.8
Crossroads	196	1710	604	1800	408	1530	697	2721	94	1710	681	1800	0.7139	65.0	62.0	11.00
Callaghan	479	1668	1092	3177	100	291	1020	5268	364	1668	1093	3512	0.7556	52.1	52.7	33.2
Wurzbach	421	1668	578	3115	64	345	1248	4648	482	3231	1463	3512	0.6701	50.5	57.7	29.8
Huebner	291	2234	324	2488	49	376	1557	5268	493	1430	1156	3767	0.7706	24.5	55.8	57.7