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16. Abstract

The objective of this study was the development of geometric design guidelines for suburban, high-speed curb and gutter roadways. Design elements that were addressed included design speed, alignment, cross section, drainage, driveways, and sight distance. Safety studies, operational studies and clear zone studies were conducted. Field data collection sites for the studies were selected from various areas throughout the state of Texas. The safety studies analyzed the safety effects of high-speed curb and gutter roadway sections through accident rates, accident severities, and accident characteristic frequencies. The operational studies included a study concerning shoulder requirements and two-way left-turn lane (TWLTL) requirements. The clear zone study was conducted to determine the most appropriate and cost-beneficial clear zone width requirements for suburban high-speed curb and gutter sections.

The resultant guidelines were based on the input from a panel of experts and the results of several safety, operational, and computer simulation studies. They were prepared in a format such that they can be inserted into the current edition of TxDOT's Design Manual. The geometric design criteria for suburban highways were prepared in both English and metric units. Adoption of these guidelines should result in a consistent, safe and defensible set of geometric design guidelines for suburban high-speed curb and gutter roadways.

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GEOMETRIC DESIGN GUIDELINES FOR SUBURBAN HIGH-SPEED CURB AND GUTTER ROADWAYS

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IMPLEMENTATION STATEMENT

This study developed geometric design guidelines for suburban high-speed curb and gutter roadways in response to a void in current design policies. These guidelines were based on the results of several safety, operational, and computer simulation studies. They have been prepared in a format such that they can be inserted into the current edition of the TxDOT Design Manual. Both English and metric versions of the guidelines are included as appendices to this report. Adoption of these guidelines should result in a consistent, safe, and defensible set of geometric design criteria for suburban high-speed curb and gutter roadways. There are no costs associated with implementing the results of this research.

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 or policies of the Texas Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation and is NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. The engineer in charge was Daniel B. Fambro, P.E. No. 47535.

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The authors also wish to acknowledge the contributions of the research assistants and student technicians at the Texas Transportation Institute who helped collect, reduce, and analyze the data for this report. Specifically, we would like to thank Lee Anne Shull and Melinda Butts for their contributions in preparing the final report.

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SUMMARY

Increasing roadway capacity without acquiring additional right-of-way in suburban areas is a challenge designers are facing more and more often as high-speed transition areas become more urbanized and roadway volumes continue to rise. To complicate the problem, there is little guidance available for designing high-speed suburban roadways, especially those with curb and gutter sections. The objective of this study was the development of geometric design guidelines for suburban high-speed curb and gutter roadways in response to this void in current design policies. Design elements that were addressed included design speed, alignment, cross section, drainage, driveways, and sight distance.

Chapter 1.0 of this report includes an introduction and outlines the problem statement and objectives of this study. Chapter 2.0 provides a more detailed background on the current design guidelines for both urban and rural roadways, focusing on the differences between the two sets of guidelines and identifying a range within which the suburban high-speed roadway guidelines will fall. Also included in Chapter 2.0 are the results from a TxDOT survey of design engineers from all TxDOT districts. This survey was used to catalog designer concerns relating to these roadway sections and to identify potential sites for study.

Chapter 3.0 focuses on the safety studies performed, including analysis of crash rates, crash severity, and crash characteristics. The operational studies conducted are discussed in Chapter 4.0. These include a speed study, lane distribution study, and conflict rate analyses pertaining to shoulder requirements and two-way left-turn lane requirements. The clear roadside study is presented in Chapter 5.0, along with a benefit/cost analysis.

Chapter 6.0 draws conclusions and recommendations from the operational, safety, and clear roadside studies. In addition, Appendices F and G offer guidelines in language suitable for placement in TxDOT's *Geometric Design Policies and Procedures Manual*.

The resultant guidelines were based on the input from a panel of experts and the results of several safety, operational, and computer simulation studies. They were prepared in a format such that they can be inserted into the current edition of the TxDOT Design Manual. Both English and metric versions of the guidelines are included as appendices to this report. Adoption of these guidelines should result in a consistent, safe, and defensible set of geometric design guidelines for suburban high-speed curb and gutter roadways.

1.0 INTRODUCTION

Increasing roadway capacity through expansion in developing suburban areas is a major concern of roadway designers as these areas become more urbanized and the roadway volumes continue to rise. These highways usually have restrictive existing right-of-way with adjoining commercial and residential development. Depending on the extent of adjoining development, right-of-way for a rural type multi-lane facility may or may not be economically feasible due to right-of-way and relocation expense. In order to reduce the right-of-way taking and associated costs, a curbed section with inlets and storm drains is often proposed.

This type of design eliminates the need for parallel drainage ditches and, as such, reduces the right-of-way taking and associated costs to make the project more cost effective. Often, these roadways have high posted speed limits (i.e., 80.5 or 88.5 km/h (50 or 55 mph)), have a high density of driveway access and side road intersections, and serve as mail carrier routes. When confronted with this type of design situation, the required design criteria does not particularly follow urban street criteria nor multi-lane rural highway criteria. Design elements, such as the required clear zone requirements, design speed, shoulder requirements, curb type, etc. need to be clearly defined for projects of this type.

There is currently little guidance available for designing suburban high-speed roadways, especially those with curb and gutter sections. There are a number of potential problems with this type of roadway that must be addressed if the department is to maintain a safe and cost-effective design. First, because this type of roadway typically has a large number of driveways and minor intersections, as well as a high percentage of turning maneuvers, the horizontal and vertical alignment of suburban high-speed roadways is critical. In some situations, horizontal and vertical curves designed with minimum stopping sight distance as a criterion may not be adequate for this type of roadway use due to large numbers of unexpected roadway entrance and exit maneuvers. Drivers must be able to correctly predict and evaluate the actions of entering and exiting traffic and perform in a safe and reasonable manner. Thus, either the design speed and/or minimum allowable stopping sight distances for these roadways may need to be increased if they are to operate in an acceptable manner.

Second, because of the high speeds on these roadways, cross slopes for the outer lanes must be adequate for drainage. In urban areas where speeds are low, it is not generally a problem if water ponds in the outside lane during periods of heavy rainfall. Because of the potential for hydroplaning on high-speed roadways, however, it is not acceptable to have standing water on the travel lanes at any time. Drainage requirements for high-speed curb and gutter sections, therefore, are different than those for low-speed curb and gutter sections. This difference should be addressed in the design guidelines.

Finally, driveway design and intersection sight distance requirements are altered whenever suburban high-speed curb and gutter sections are built. If additional right-of-way is not available, the driveway must be realigned, often in a less than desirable manner with steeper grades and shorter lengths. Due to the increase in the roadway width, the intersection sight distance requirements may have been increased. The available sight distances that were previously adequate may no longer be adequate for the increased roadway width drivers must now cross. Clearly, these and other geometric design issues must be addressed if suburban high-speed curb and gutter sections are to be designed and constructed in a safe and cost effective manner.

Barrier curbs with heights of 0.15 to 0.20 m (6 to 8 in) are commonly used in urban areas to shield sidewalks. The American Association of State Highway and Transportation Officials (AASHTO) policies permit placement of rigid objects such as utility poles within 0.46 m (18 in) of the face of these curbs in urban roadways. Studies have shown that even at relatively low speeds and angles of approach these curbs have minimal capacity to contain and redirect an errant vehicle. Studies have also shown that impact performance of a barrier or breakaway feature can be degraded when a curb is placed between traffic and the feature. Curbs also increase the propensity to overturn for an out-of-control vehicle leaving the travelway, and the propensity increases with decreasing vehicle size. However, curbs can be beneficial for drainage purposes where limited right-of-way precludes roadside drainage. They can also be used for delineation and for traffic control to discourage intentional encroachments on a median or the roadside. Therefore, curbs can offer several benefits to suburban high-speed roadways.

1.1 PROBLEM STATEMENT

Designers are often presented with the need for suburban high-speed curb and gutter sections in many situations, and no design guidelines are available for these sections other than recommendations against their use. Adequate drainage, lack of shoulders, clear zone requirements, operating versus design speeds, and vaulting are some of the considerations which should be addressed in order to document the safety and operational trade-offs between curb and gutter and rural drainage ditch cross sections on high-speed suburban multi-lane highways.

Roadside safety design has been given much more attention for high-speed rural and interstate roadways than for suburban roadways. Designers have developed detailed guidelines on national and state levels to address the design and use of roadside elements for rural and interstate roadways. Such elements include guardrails; median barriers; crash cushions; breakaway support structures for signs; luminaries; utility lines; and geometric features such as curbs, side slopes, and ditches. Well defined clear zones exist for high-speed rural and interstate roadways.

Current AASHTO policies and guidelines on roadside safety design are contained primarily in the Green Book, *A Policy on Geometric Design of Highways and Streets - 1990*, (1) and in the *Roadside Design Guide - 1989* (2). The Green Book refers the reader to the *Roadside Design Guide* for guidance on roadside safety design. As stated in Section 1.3 of the *Roadside Design Guide*, "Much of the guidance in this book is focused on high-volume roadways having operating speeds of 80.5 km/h (50 mph) or more."

Highway designers must address many geometric factors in the development of highway projects, including lane widths, maximum and minimum grades, horizontal and vertical clearances to obstructions, maximum curvature, and cross section design. Current design policies provide guidelines for urban design and rural design, yet fail to provide any transitional standards for suburban designs.

This study examines various facilities of this type throughout the state of Texas to determine optimum design criteria in order to develop specific guidelines to follow when designing high-speed curb and gutter type facilities in suburban areas. These guidelines will address both geometric and roadside design.

1.2 OBJECTIVES

The objectives of this study are to develop geometric design guidelines for suburban high-speed curb and gutter sections. Specific geometric design guidelines addressed include horizontal and vertical alignment, design speed, drainage, driveway design, intersection sight distance, and cross section. Cross section elements discussed include lane widths, shoulder requirements, bicycle and pedestrian accommodations, and two-way left-turn lanes. Also addressed are the clear zone requirements for high-speed curb and gutter sections. Recommendations are made as to the placement of roadside objects. For ease of incorporation into The Texas Department of Transportation's (TxDOT) *Geometric Design Policies and Procedures Manual*, the guidelines are written in clear, concise language which can be readily used by department personnel.

From the objective guidelines presented in this paper, TxDOT, AASHTO, and the Federal Highway Administration (FHWA) will be provided with the opportunity to either formulate or revise design policies relative to geometric design and roadside safety design in suburban areas. These guidelines will enable TxDOT engineers to plan and construct roadways of uniform design throughout the state, to maintain a cost effective and safe highway system, and to increase the capacity and safety of suburban highways.

1.3 REPORT ORGANIZATION

Chapter 1.0 of this report includes an introduction and outlines the problem statement and objectives of this study. Chapter 2.0 provides a more detailed background on the current design guidelines for both urban and rural roadways, focusing on the differences between the two sets of guidelines and identifying a range within which the suburban high-speed roadway guidelines will fall. Also included in Chapter 2.0 are the results from a TxDOT survey of design engineers from all TxDOT districts. This survey was used to catalog designer concerns relating to these roadway sections and to identify potential sites for study.

Chapter 3.0 focuses on the safety studies performed, including analysis of crash rates, crash severity, and crash characteristics. The operational studies conducted are discussed in Chapter 4.0. These include a speed study, lane distribution study, and conflict rate analyses pertaining to shoulder requirements and two-way left-turn lane requirements. The clear roadside study is presented in Chapter 5.0, along with a benefit/cost analysis.

Finally, Chapter 6.0 draws conclusions and recommendations from the operational, safety, and clear roadside studies. In addition, Appendices F and G offer guidelines in language suitable for placement in TxDOT's *Geometric Design Policies and Procedures Manual*.

2.0 BACKGROUND

2.1 INTRODUCTION

As metropolitan areas continue to extend into more rural areas, and as smaller cities continue to grow, the need for increased capacity on suburban highways becomes more evident. Many of these highways are currently two-lane rural designs that utilize earth ditches for drainage. As development has increased, the right-of-way along these routes has become very restrictive. When it becomes necessary to increase capacity on these highways, this restrictive right-of-way necessitates the use of curb and gutter sections for highway drainage. Most suburban highways were originally designed as high-speed facilities and posted speed limits have remained in the 80.5 to 88.5 km/h (50 to 55 mph) range. The absence of any established design criteria for this high-speed, suburban situation has caused many designers to apply less restrictive, low-speed urban criteria to the design. Traffic, however, continues to operate in a high-speed, more rural manner.

Designers must consider many factors when making design decisions, including traffic volumes, roadway cross section, type and intensity of adjacent land use activity, and terrain and climatic conditions. It is important to consider the effect that each of these factors will have on the operational characteristics of the traffic stream. These operational characteristics include free-flow vehicular speeds, vehicle type, directional distribution of vehicles, distribution of vehicles by lane, and accident rates. By examining the effects of different types of cross sections on these operational characteristics, some conclusions may be reached as to when and where to implement specific cross-sectional designs.

2.2 GEOMETRIC DESIGN PROCEDURE

Current TxDOT policy dictates that all geometric design follow the guidelines presented in AASHTO's Green Book (1). The TxDOT Highway Design Division has developed the *Operations and Procedures Manual* (3), based on the Green Book policies, as a more concise and convenient method of designing highway facilities. This manual is commonly referred to as the "Design Manual." In Texas, the guidelines established in the TxDOT Design Manual take precedence over the AASHTO guidelines; however, both publications approach the design procedure similarly with respect to traffic characteristics, speed characteristics, and design elements.

2.2.1 Traffic Characteristics

Traffic volume is a very important basis for determining the improvements necessary on a particular highway facility. Total volume is usually expressed as average daily traffic (ADT), which represents the average traffic volume per day, regardless of any seasonal, weekly, daily, or hourly variation. In order to provide for adequate capacity on a facility, the designer must also consider temporal variations, directional distribution, lane distribution, and traffic composition.

Traffic demands can vary by month of year, day of week, or hour of day. It is important to consider these variations if a highway facility is going to effectively serve peak demands without a breakdown. Studies have shown that monthly variations are much more severe on rural routes, serving mostly recreational traffic. Because suburban, high-speed facilities serve a more commuter-oriented travel pattern, monthly variations need not be addressed. Daily variations in traffic volume are also dependent on the location of the facility and the purpose of travel served. For business and commuter travel, weekday volumes are higher than weekend volumes; therefore, only weekday volumes need be considered for suburban, high-speed facilities. These weekday volumes can be further evaluated for hourly variations.

Urban and suburban commuter routes demonstrate a distinct peak flow in the morning and again in the evening. These peak flows represent the most critical period for operations, demanding the highest level of capacity and are, therefore, used most often in the design of urban and suburban facilities. The percent of ADT occurring in the peak hour is referred to as the "K" factor. This factor, usually 10 to 15 percent in suburban locations, is used in the conversion of ADT to design hour volume (DHV).

Directional distribution is important in the determination of capacity of a highway facility. For multi-lane facilities, however, directional distribution is not explicitly considered in the analysis. While there is indeed a significant variation in the directional distribution of vehicles in a suburban, commuter-type facility, the peak direction is usually reversed from morning to evening, resulting in the need to design both directions for essentially the same peak flow in each direction, or a directional distribution of 50-50.

The distribution of vehicles by lane plays an important role in the analysis of freeways and freeway ramp junctions. Lane distribution is dependent upon many factors, including volumes, speed, vehicle type, and number and frequency of access points. A survey of high-volume facilities indicated that there was very little consistency in lane distribution in most cases (4). Because there are no typical lane distributions, an average per lane capacity is assumed, recognizing the fact that flow may be higher in some lanes than in others. For the design of high-speed, suburban curb sections, lane distribution is not considered a controlling factor.

Adjustment must also be made to the ADT to account for the presence of heavy vehicles, such as trucks, recreational vehicles (RVs) and buses in the traffic stream. The 1985 *Highway Capacity Manual* (4) provides adjustment factors for different percentages of heavy vehicles over a range of vertical grades and lane cross sections.

2.2.2 Speed Characteristics

One of the objectives of designing a highway facility is to provide for the safe and economical movement of people and goods. Therefore, it is important to provide facilities upon which nearly all drivers can drive at a comfortable speed. It would not be economically feasible, however, to provide a facility that satisfies all drivers. AASHTO defines three types of speed that are interrelated and should be considered when designing transportation facilities: design speed, operating speed, and running speed.

Design speed is defined as the "maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern" (1). Design speed is determined as a function of topography, functional classification, and adjacent land use. Many design features, such as horizontal and vertical curvature, superelevation, and sight distance depend on and vary with changes in design speed. Generally, higher functional classification facilities have a lower level of access and, therefore, a higher design speed can be utilized. Drivers do not adjust their speeds according to the classification of a highway, however, so consideration should be given to the desires of nearly all drivers when selecting a design speed.

Operating speed is defined as the "highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed..." (1). Operating speed is controlled by the design speed rather than by design features.

Running speed is one measure of the level of service at which a facility is operating and has been defined as "...the speed of a vehicle over a specified section of highway being the distance traveled divided by the running time." (1). One means of determining running speed is to take the arithmetic mean of the speeds of all vehicles passing a specific point. This speed is important in that it represents actual vehicle speeds on a segment of highway. It is useful in determining the adequacy of a specific design.

2.3 GEOMETRIC DESIGN ELEMENTS

When designing a highway facility, an engineer must consider many design elements, including the cross section and alignment. The cross section is made up of several elements, such as travel lanes, shoulders, medians, bike lanes, and clear zone. These elements must be designed properly to provide proper drainage, minimize conflicts, reduce delay, and satisfy other concerns to ensure safe and efficient traffic operations. Appropriate horizontal and vertical alignment are also important design elements to consider to ensure driver comfort, driver expectancy, adequate sight distance, and proper drainage. Another concern in urban areas, where restricted right-of-way necessitates the use of curb and gutter to facilitate drainage, is which type of curb to use.

As discussed earlier, the objective of this paper is to develop guidelines for designing high-speed, curb-type facilities in suburban areas. In order to develop design guidelines, boundaries must first be defined. A suburban highway functions as a transition between an urban highway and a rural highway. Therefore, to be consistent, the design standards for suburban facilities should fall somewhere between rural and urban design criteria. While there are no criteria currently established for suburban high-speed curb sections, the TxDOT Design Manual and the AASHTO Green Book address basic design criteria common to all types of highways. The following is a discussion of the design elements examined in this study, and the current rural and urban design guidelines. Reviewing these current guidelines will help define boundaries for developing design criteria for suburban highways.

2.3.1 Alignment

Stopping Sight Distance

Due to the large number of driveways and minor intersections on most suburban high-speed roadways, the horizontal and vertical alignment is critical. TxDOT follows the guidelines set forth by AASHTO concerning required stopping sight distances. The recommended stopping sight distances range from 121.9 to 144.8 m (400 to 475 ft) for design speeds of 80.5 km/h (50 mph), and from 160.0 to 198.1 m (525 to 650 ft) for design speeds of 96.6 km/h (60 mph). These values are used to determine design criteria for horizontal curve radii and vertical curve length.

Because of the large numbers of unexpected roadway entrance and exit maneuvers on suburban highways, horizontal and vertical curves designed with minimum stopping sight distance as a criterion may not be adequate. Drivers must be able to correctly evaluate and anticipate the actions of entering and exiting traffic and perform in a safe and reasonable manner. Therefore, the allowable minimum stopping sight distances for these roadways may need to be increased if they are to be negotiated in an acceptable manner.

Horizontal Alignment

Both the TxDOT Design Manual and the AASHTO Green Book maintain that designers should use maximum superelevation rates of 0.04 to 0.06 for urban streets and 0.06 to 0.08 for rural highways. Urban streets require lower superelevation rates because of problems with drainage and grade separation with adjacent cross streets and driveways.

TxDOT uses a different method than does AASHTO for distributing superelevation and side friction on low-speed, urban streets. AASHTO's method is shown in Figure III-6 of the Green Book and is referred to as "Method 2." This method begins with a superelevation of zero and uses only side friction up to the maximum allowable side friction. After side friction has reached its maximum, superelevation is introduced. Therefore, superelevation does not have to be used on flatter curves that require less than the maximum side friction factor. TxDOT recommends that the design superelevation rate be determined, using a figure (Figure 4-7) in the Design Manual, given specific curvature and design speed conditions. Because TxDOT's method allows superelevation to be introduced earlier than it is when using AASHTO's method, it could produce more problems with drainage and driveways on horizontal curves.

Vertical Alignment

TxDOT also follows AASHTO's guidelines concerning vertical grades. Maximum grades for urban arterials range from 5 percent to 7 percent depending upon design speed and terrain. Rural arterials may have maximum grades of 3 percent to 4 percent. These maximum grades, rather than typical design values, should be used only when necessary. For pavements with curbs, desirable minimum grades of .35 percent should be provided to facilitate surface drainage.

2.3.2 Cross Section

Travel Lanes

The cross-sectional element that has the greatest effect on the safety and operations of a highway is the travel lane. The width of a travel lane not only affects the capacity of a highway, but also driver workload and the potential for accidents. AASHTO and TxDOT recommend desirable 3.7 m (12 ft) lanes for high-speed facilities. Studies have shown that this width maximizes lane capacity while minimizing accidents. The minimum lane width for through highways is 3.0 m (10 ft).

TxDOT follows AASHTO's recommendation of a desirable 2.0 percent cross slope for curbed pavements. Under usual conditions, this will provide adequate drainage while minimizing the effect on steering. Areas of high rainfall may require steeper slopes to facilitate drainage. The recommended cross slope for most highways ranges from 1.5 percent to 3 percent.

Shoulders

The purpose of a shoulder on a roadway is to provide a buffer between the main lanes and the pavement edge. Shoulders are desirable on any highway because of their many advantages. Among the most important advantages of shoulders in urban areas are the following (1):

1. Provide refuge for stranded vehicles, buses, mail delivery, and so forth;
2. Provide space for bike use, pedestrians, and parking;
3. Provide area for entering/exiting traffic to be separated from through traffic;
4. Provide space to avoid potential accidents;
5. Move drainage further away from the main lanes in curbed sections;
6. Protect pavement edge; and
7. Increase highway capacity.

The desirable and minimum shoulder width criteria are based upon the function of the shoulder and the characteristics of the highway. Shoulders are used predominantly in rural areas instead of urban areas primarily because of fewer right-of-way restrictions (5). The TxDOT Design Manual maintains that a minimum 1.2 m (4 ft), desirable 3.0 m (10 ft) shoulder be provided on uncurbed new or completely reconstructed urban streets. When refuge lanes are used, they should be a minimum of 3.0 m (10 ft), desirable 3.4 m (11 ft) to 3.7 m (12 ft) wide. The AASHTO Green Book maintains that urban streets with curbs should have shoulder widths of at least 1.8 m (6 ft) to accommodate disabled vehicles.

AASHTO also states that rural arterials with traffic volumes that justify four or more lanes should be provided with outside shoulders of at least 2.4 m (8 ft) in width. This corresponds with the TxDOT Design Manual that requires a minimum 2.4 m (8 ft), desirable 3.0 m (10 ft) shoulder on all multi-lane, rural highways. TxDOT suggests that rural, two-lane arterials have 1.2 to 3.0 m (4 to 10 ft) wide shoulders, based on traffic volumes. AASHTO suggests that in states such as Texas that allow slow moving vehicles to move onto the shoulder to allow other vehicles to pass, a minimum 2.4 m (8 ft), desirable 3.0 m (10 ft) shoulder should be provided.

Medians

TxDOT states that medians are highly desirable on urban streets carrying four or more lanes (typical of most high-speed curb sections) primarily to provide storage space for left-turning vehicles. Mukherjee, et al. (6) conducted a survey of state highway engineers to evaluate how they dealt with left-turning traffic for suburban arterials. The results revealed that the engineers differed in their assessment of non-traversable medians and continuous two-way left-turn lanes (TWLTLs). There appeared to be no clear cut choice between a median and a TWLTL. It was concluded that choosing between a raised median and a TWLTL involves trade-offs among safety, delay, and land development considerations.

Where it is desirable to permit access, a TWLTL offers many advantages including reductions in delays and accident frequencies. A study conducted by Horne and Walton (7) showed that by installing a TWLTL where no median was previously provided, total accidents were reduced by 33 percent with reductions in head-on and rear end accidents of 45 percent and 62 percent, respectively. Other studies have shown that head-on collisions, a primary concern of TWLTL design, have proven to be "an uncommon occurrence and of negligible concern" (5).

Other studies have been conducted in which procedures were developed to estimate the delay and number of accidents for different median designs. Parker (8), for example, developed equations for estimating accidents and left-turn delay for roads with TWLTLs and nontraversable medians. The data he used to develop these equations were collected in Virginia. Squires and Parsonson (9) developed equations for estimating accidents for roads with TWLTLs and raised medians from data collected in Georgia. Harwood (10) developed a procedure that involved using tables and graphs to estimate accidents and delay for roads with TWLTLs and raised medians. Harwood gathered data from California and Michigan for his study.

Mukherjee, et al. (6) conducted a comparative analysis of the mathematical procedures developed by Parker, Squires and Parsonson, and Harwood. The analysis was performed by applying each of the author's procedures to several real-life scenarios. The procedures produced conflicting results, which may be due to the fact that each of the models were built with different data sets. Nevertheless, the analysis showed that these models are not applicable to all cases.

The TxDOT Design Manual contains several criteria regarding TWLTLs in urban environments. The desirable width for TWLTLs ranges from 3.7 to 4.9 m (12 to 16 ft), depending on the maximum legal speed. In addition, TxDOT lists several criteria that would warrant the use of a TWLTL with one travel lane in each direction. These criteria are as follows:

1. ADT volume of 3000 or more;
2. Side road plus driveway density of 12 or more entrances per km (20 or more per mi);
3. Speed limit of 72.4 km/h (45 mph) or less; and
4. Length of three lane section of 2.4 km (1.5 mi) or less.

TxDOT states that, when at least three of the above criteria are met, a three lane (including TWLTL) design is warranted. From this criterion and the above discussion, it can be seen that there are currently no clear guidelines for implementing TWLTLs on high-speed suburban streets.

Clear Zone

The roadside clear zone is the area outside of the travel lane that is relatively flat and free from obstructions. This area should provide enough room for an errant vehicle to recover. The AASHTO *Roadside Design Guide* (2) suggests various clear zone widths based on speed, volume, and embankment slope. TxDOT's own recommendations for clear zone widths are based on these criteria. For low-speed, low-volume rural roadways, the TxDOT Design Manual suggests a minimum clear zone width of 3.0 m (10 ft). High-volume rural arterials with a design speed of 72.4 km/h (45 mph) or greater should be provided with a minimum clear zone width of 9.1 m (30 ft). For urban curbed pavements with a design speed of 72.4 km/h (45 mph) or less, a minimum 0.46 m (1.5 ft), desirable 0.91 m (3.0 ft) clear zone width is designated. For urban curbed pavements with a design speed of 80.5 km/h (50 mph) or greater, the Design Manual assigns those values used for rural arterials, with a minimum clear zone width of 9.1 m (30 ft).

Bike Lanes

AASHTO's *Guide for the Development of Bicycle Facilities* maintains that "every street and highway on which bicycles are permitted to operate is a bicycle street and should be designed and maintained to accommodate shared use by bicycles and motor vehicles" (11). Recently, Texas enacted a law which allows bicyclists to travel on all streets and highways. Therefore, all streets and highways in Texas should at least accommodate the experienced bicyclists. The following discussion and criteria on bicycle lanes is from FHWA's report entitled *Selecting Roadway Design Treatments to Accommodate Bicycles* (12). This report is a first attempt to provide comprehensive guidelines for accommodating bicycles on highways; however, it is not intended to serve as a comprehensive guide to the design of bicycle facilities. For detailed specifications, designers are referred to AASHTO's *Guide for the Development of Bicycle Facilities*.

Bicycle riders fall into one of two categories, experienced and non-experienced. Experienced riders are those who can operate under most traffic conditions and are typically the users of collector and arterial streets. These riders will best be served by designing all roadways to accommodate shared use by bicycles and motor vehicles. Non-experienced riders can include children, teenagers, or adults who are not comfortable with riding on streets without special provisions for bicycles. These cyclists can best be served by providing designated bike facilities on streets or separate bike paths.

There are five basic types of facilities that are used to accommodate bicyclists. They are as follows:

1. Shared lane - shared motor vehicle/bicycle use of a "standard-width" travel lane;
2. Wide outside lane - an outside travel lane with a width of at least 4.3 m (14 ft);
3. Bike lane - a portion of the roadway designated by striping, signing, and/or pavement marking for preferential or exclusive use of bicycles;
4. Shoulder - a paved portion of the roadway to the right of the edge stripe designed to serve bicyclists; and
5. Separate bike path - a facility physically separated from the roadway and intended for bicycle use.

An experienced rider can be accommodated with any of the above; however, a bike lane, shoulder, or separate bike path should be provided for non-experienced bicyclists. This must be taken into consideration during the bicycle facility planning process. Roadways that are undesirable for the non-experienced bicyclist (i.e., urban streets with high volumes of traffic and/or high operating speeds) should not have bicycle facilities that would encourage use by non-experienced bicyclists.

The criteria for the accommodation of bicycle lanes are based on AADT, average motor vehicle operating speed, available sight distance (adequate or inadequate), and the amount of truck, bus, and RV traffic. AADT refers to the average annual daily traffic which is defined as the average 24-hour traffic volume at a given location over a full 365-day year. For high-speed (i.e., over 80.5 km/h (50 mph)) urban sections, it is recommended that a minimum 1.8 m (6 ft) shoulder be provided for experienced riders, and a desirable 1.8 m (6 ft) designated bike lane be provided for non-experienced bicyclists. For high-speed rural sections, it is recommended that a minimum 1.2 to 1.8 m (4 to 6 ft) shoulder (based on truck, bus, and RV traffic) be provided for the experienced rider, and a desirable 1.8 m (6 ft) designated bike lane be provided for the non-experienced rider.

2.3.3 Curbs

The method through which drainage is achieved is one design element which changes as the roadway features change. On rural roadways, parallel drainage ditches are utilized for drainage; however, in urban areas where design speeds are lower and right-of-way is more restrictive, a curb and gutter section is often implemented. Curbs are widely used on urban collector roads and highways, however, because high-speed rural roadway (80.5 km/h (50 mph) or greater) curbs are considered undesirable (1).

Curbs, according to AASHTO, serve a combination of the following purposes: drainage control, pavement edge delineation, right-of-way reduction, aesthetics, delineation of pedestrian walkways, reduction of maintenance operations, and assistance in orderly roadside development (1). There are two types of curbs, barrier and mountable. Barrier curbs are higher than mountable curbs, and have a steep face. This curb type ranges from 0.15 to 0.23 m (6 to 9 in) in height. Mountable curbs are designed for vehicles to cross over, having a height of 0.10 to 0.15 m (4 to 6 in).

The Green Book states in three separate instances that barrier curbs in combination with high-speed arterials and/or freeways are "highly undesirable" (1). The two reasons listed as the justification for this guideline are that (1) vehicle operators may have increased difficulty in maintaining control of their vehicle when a barrier curb is traversed or impacted at high speeds, and (2) barrier curbs are not adequate to prevent a vehicle from exiting the roadway. In locations where a suburban roadway is in need of expansion and right-of-way is limited, using a barrier curb is the only feasible method through which drainage control can be accomplished. On these roadways, it is likely that the clear zone requirement is not met due to roadside development, and that a speed limit in excess of 80.5 km/h (50 mph) is in existence. Therefore, there exists a concern according to AASHTO's statements in the Green Book that accident rates and accident severity might be increased.

Studies in the past have addressed these potential problems of redirection and vaulting, and confirmed AASHTO's concerns over barrier curbs on high-speed roadways. In a study by Olson, Weaver, Ross and Post, it was found that "curbs offer no safety benefit on high-speed highways from the standpoint of vehicle behavior following impact" (13). This study conducted 18 full-scale vehicle impact tests on four typical curb designs (C, E, H, and X) as well as 30 simulation tests using HVOSM. Evaluation criteria included vehicle path, vehicle attitude, and vehicle acceleration. None of the curbs that were tested redirected the vehicles satisfactorily. They concluded that the omission of curbs along high-speed roadways will enhance safety, and they recommended that the use of curbs be discontinued on high-speed roadways. If a barrier curb is needed, this study concluded that a full height barrier curb should be selected for use.

A similar study, also conducted by Ross and Post (14), involved the traversing of certain curb configurations (0.15 and 0.20 m (6 and 8 in) heights) and sloped medians with regard to vaulting over a barrier behind the curb or in the sloped median. This study, using 14 Highway-Vehicle-Object Simulation Model (HVOSM) simulations, concluded that traffic barriers should not be placed near curbs. In many cases, with the existing configurations, vehicles have the potential to vault over the barrier, or snag on the barrier. A flat approach to the barrier is highly recommended by Ross and Post; however, the problem can also be mitigated by sloping the median or roadside.

2.4 SAFETY OF DESIGN AND OPERATIONAL FEATURES

It has been shown through past research that several road design and operational features have an effect on the accident experience of a roadway (15). These features include median type and width, shoulder type and width, access control, lane width, traffic volumes, and roadside features such as clear zone. These design features will have an effect on the accident experience on high-speed suburban highways if they are modified. The operational features, ADT and access level and control, will account for a portion of the different accident experience throughout all high-speed curb and gutter sections. A discussion of research regarding each of these elements as they relate to roadway safety is presented below.

2.4.1 Medians

In cases where operational and safety problems exist on a high-volume two-lane roadway, several options exist which can upgrade the roadway. A study by Harwood (16) investigated the safety aspects of several of these options including the following: three-lane divided highway with a two-way left-turn lane in the median, a four-lane undivided highway, a four-lane divided highway with one-way left turn lanes in the median, and a five-lane highway with a continuously alternating left-turn lane in the median. Researchers calculated accident rates, based on data from California and Michigan, in accidents per million vehicle miles for the above options, for both commercial and residential locations. In commercial areas, accident rates higher than the original two-lane design were experienced by the two four-lane designs (undivided and divided with one-way left-turn lanes in the median) as well as the five-lane with continuous one-way left-turn lane in the median. The only commercial alternative to experience a reduction in accident rate as compared to the original two-lane design was the three-lane divided highway with a two-way left-turn lane in the median design. In residential areas, the original two-lane option had the highest accident rate. The two four-lane options experienced the next highest accident rates. The three-lane design followed with the next highest rate, and the five-lane alternative had the lowest accident rate.

2.4.2 Pavement and Shoulder Width

A study by Zeager and Deacon sponsored by the Federal Highway Administration (17) analyzed over 8000 kilometers (5000 miles) of two-lane highway accident data in seven states. This study revealed three accident types related to shoulder and lane width: run-off-the-road accidents, head-on collisions, and sideswipes (both opposing and same direction traffic). Through the use of an accident prediction model, researchers calculated the expected effects of lane widening and shoulder widening on the group of three accident types. These effects are appropriate for estimating reductions on two-lane roads with ADTs of 100 to 10000 vehicles per day, lane widths of 2.4 to 3.7 m (8 to 12 ft), and shoulder widths of 0 to 3.7 m (0 to 12 ft) either paved or unpaved. For a lane widening project, the percent reduction in related accident rates ranged from 12 percent to 40 percent as the lane increased in width from 0.30 to 1.2 m (1 to 4 ft). For a shoulder widening project, the percent reduction in related accident rates for paved shoulders ranged from 16 percent to 49 percent as the shoulder increased from 0.61 m (2 ft) per side to 2.44 m (8 ft) per side; and for unpaved shoulders, the percent reduction in related accident rates ranged from 13 percent to 43 percent as the shoulder increased from 0.61 m (2 ft) per side 2.44 m (8 ft) per side.

Other studies have revealed similar results, in general, showing that wider shoulders and lanes tend to decrease accident rates. Griffin and Mak (18) performed a study on rural, farm-to-market roads in Texas, and indicated that single vehicle accident rates decreased for wider roadway widths for various ADT groupings. A before/after type study by Rogness, et al. (19) analyzed 30 sections of two-lane roads to which paved shoulders had been added. This study found reductions in single vehicle accidents of 55 percent for ADTs from 1000 to 3000, 21.4 percent for ADTs from 3000 to 5000, and 0 percent for ADTs from 5000 to 7000. This indicates greater accident reductions due to shoulder widening at lower ADT levels.

2.4.3 Access Control

Several studies have been undertaken to determine the relationship between access and highway safety, with respect to both level of access and access control. Most predominant was a study by Stover, et al. (20) which utilized data from over 30 states. This study resulted in the submission of a report to Congress concluding that full access control was the most important design factor for accident reduction. Full access control decreased accident rates by approximately 50 percent as compared to rural highways with no access control, and 33 percent compared to urban highways with no access control.

A similar study was undertaken by the Bureau of Public Roads (21). This study primarily oriented to determine the safety of the interstate system, included data from 40 states. Results indicated a very strong relationship between access control and accident rate. In addition to full access control lowering accident rates, the study revealed that accident rates increase as the number of access points increase.

A study by Cribbins, et al. (22) conducted in North Carolina suggested that access was one of the most contributory variables to accidents. All accident rates within the study increased with frequency of access points and signalized openings per kilometer (mile). A study in Indiana at Purdue University conducted by McGuirk (23) experienced similar results in driveway accidents. The accident rates calculated for sites in this study increased with number of lanes, commercial driveways, intersections per kilometer (mile), driveways per kilometer (mile), commercial driveways per kilometer (mile), and urban area population.

2.4.4 Traffic Volumes

Several studies have portrayed that accident rates tend to increase with average daily traffic (ADT). A study by McGuirk (23) demonstrated that accident rates increase with both ADT and frequency of access. In the study mentioned by Cribbins, et al. (22), it was found that traffic volume and measures of access were the two most significant contributors to accidents. In essence, it is widely accepted that accident rates increase with increasing ADT.

2.4.5 Roadside Features

The roadside clear zone is another design element which has a profound effect on the safety of the roadway. The three characteristics often used to describe the roadside are the recovery distance (clear zone), side slope, and obstacles (15). It was found by Graham and Harwood (24) that the clear zone policy had an effect on the single vehicle accident rate. Within various levels of ADT, single vehicle accident rates were found to be highest on roadways with a non clear zone policy. They were found to be lower for roadways with a 4:1 clear zone side slope policy, and lower yet for roadways with a 6:1 clear zone side slope policy.

Zegeer and Deacon's study (17) also included accident reduction rates for related accidents due to increasing the roadside clear zone. It was found that with increases of clear zone from 1.5 to 6.1 m (5 to 20 ft), the percent reduction in related accident rates ranged from 13 percent to 44 percent, respectively. Within this study, it was also found that the ratio of single vehicle accidents to total accidents was highest for side slopes of 2:1 or steeper. The level of single vehicle accidents drops slightly when increased to a side slope of 3:1, and drops linearly for even flatter slopes.

In a study by Perchonok, et al. (25) in 1978, researchers analyzed several single vehicle crashes to determine the percent of injuries and percent of deaths occurring from particular roadside obstacles, leading to the classification of the most dangerous obstacles. The obstacles which had the highest percentages of injuries and deaths are bridge or overpass entrances, trees, field approaches (ditches created by driveways), culverts, embankments, and wooden utility poles.

2.5 CLEAR ZONE CONCEPT

Until the 1960's little emphasis was placed on roadside safety design. The prevailing philosophy was that reasonable and prudent drivers did not inadvertently leave the travelway, and the penalty for doing so by others was acceptable. Studies by Cooper and others (26,27) showed that even professionally trained drivers can be expected to stray from the travelway, and that measures to minimize risks of roadside encroachments were needed and warranted. This need was underscored by the alarming number of run-off-the-road single vehicle accidents which resulted in serious injuries and/or fatalities.

Recommended measures to minimize risks to errant motorists included providing an unencumbered "recovery area" along the roadside of width sufficient to permit a driver to safely bring his vehicle under control or to stop. Results of the GM studies formed the basis for initial dimensions of recommended recovery areas beyond which potential roadside obstacles did not require removal or protection (28,29). These areas were later referred to as "clear zones," (2,30) "clear recovery zones," (31) or "roadside recovery distance" (32).

The GM studies, which included analysis of over 200 accidents at the Proving Ground, provided probability data on lateral extent of vehicular movement for run-off-the-road accidents. Using these data, AASHTO suggested that, where feasible, a clear, unencumbered recovery area should be provided adjacent to the travelway (28,29). For high-speed highways, it was recommended that the width of the recovery area should extend 9.1 m (30 ft) or more laterally from the travelway. The GM studies indicated that the lateral extent of vehicular movement would not exceed 9.1 m (30 ft) in approximately 80 percent of run-off-the-road accidents on high-speed highways.

National guidelines continued to recommend a 9.1 m (30 ft) clear zone up to 1977, although it was recognized that this width was somewhat arbitrary and based on accident studies at the GM Proving Grounds, where relatively flat roadsides were provided. The 1977 AASHTO *Guide for Selecting, Locating, and Designing Traffic Barriers* (30) contained clear zone recommendations that were dependent on design speed, the slope of the cut or fill section, and whether the hinge at the juncture of the shoulder with the side slope was rounded. These guidelines indicated that the width of the clear zone should increase with increasing design speed and increasing steepness of fill slopes. For example, the recommended clearance for a 96.6 km/h (60 mph) high-speed roadway with a fill section having a 4:1 unrounded side slope was approximately 13.1 m (43 ft). For this same example and a 64.4 km/h (40 mph) design speed, the recommended clearance was approximately 5.5 m (18 ft). These clear zone criteria were developed using computer simulation to determine the lateral extent of vehicular movement for encroachments on fill and cut roadside sections, rounded and unrounded, at speeds of 64.4 km/h (40 mph), 80.5 km/h (50 mph), and 96.6 km/h (60 mph). Assumed driver response for the simulated encroachments included an emergency steer-back-to-the-travelway maneuver and emergency full braking.

The 1989 AASHTO *Roadside Design Guide* (2) contained certain revisions to the clear zone criteria of the 1977 *Barrier Guide*. The guidelines provided a range of values for recovery area distances depending on traffic volume, design speed, side slope, and other roadside conditions that exist, or will exist, along the roadway. In addition to the variables considered previously, clear zone widths were also defined in terms of traffic volume, and greater ranges of design speed were adopted. The effects of slope rounding were not considered in the update. Clear zone criteria presented in the 1989 *Guide* were derived from data in the 1977 *Guide*, in combination with state practices and the collective judgement of the task force that prepared the *Guide*.

The following is a comparison of recommendations in the 1977 *Guide* and the 1989 *Guide*, assuming the previous example, i.e., a 4:1 side slope (fill) and a 96.6 km/h (60 mph) design speed:

1977 *Barrier Guide*		1989 *Roadside Design Guide*	
Design ADT	Clear Zone Distance, m (ft)	Design ADT	Clear Zone Distance, m (ft)
All	13.1 (43)	< 750	6.1-7.3 (20-24)
		750-1500	7.9-9.8 (26-32)
		1500-6000	9.8-12.2 (32-40)
		> 6000	11.0-13.4 (36-44)

It can be seen that the 1989 guidelines recommend essentially the same clearance as the 1977 guidelines for high-volume roadways, but recommend considerably lower clearances for lower-volume roadways.

Although these guidelines provide a more realistic approach than the application of a single distance, there are still concerns regarding the appropriateness of these values because they are based on studies conducted many years ago that used relatively limited data to extrapolate numbers to cover a variety of roadside conditions. Of particular significance to this study is the fact that these guidelines were developed for rural highways and freeways, and do not specifically address the issue of appropriate clear zones for suburban high-speed curb and gutter sections. Furthermore, transportation agencies frequently face difficulties in providing desirable clear zones because of right-of-way constraints or construction costs. Updated guidelines are needed to aid highway engineers in determining safe and cost-effective clear zones, while recognizing the constraints associated with building or improving the highway system.

2.6 CURRENT GEOMETRIC DESIGN PRACTICES

In order to identify current high-speed curb and gutter design practices and problem areas for TxDOT design engineers, a questionnaire survey was sent to each TxDOT district in January 1993. Questions were asked about the high-speed, suburban roadways with curb and gutter sections within the jurisdiction of each district. A total of 17 districts responded identifying 193 high-speed curb and gutter sections in Texas. Following are a list of the questions asked and some of the most common responses. Also given is the percentage of districts that agreed with each response. An example of the questionnaire sent to the state districts is presented in Appendix A.

The first question was "*Describe any operational or safety problems associated with this type of roadway.*" The most common problems among the districts seemed to be as follows:

1. Lack of refuge for stranded private vehicles and public service vehicles (e.g., mail truck, garbage truck, etc.) on roadways without shoulders (47.1%);
2. Ponding water in outside lanes (41.2%); and
3. Right turns in and out of driveways causing safety and operational problems for roadways without shoulders (41.2%).

Other problems mentioned were as follows:

1. Bicycle accommodations without sidewalks or shoulders (23.5%);
2. Difficulty in adding driveways (11.8%);
3. Lack of recovery area for high-speed vehicles that pass over barrier curb (11.8%);
4. Trash/dirt accumulation in gutters (5.9%);
5. Difficulty in seal coating (5.9%); and
6. Problems with proper protection of bridge railing due to adjacent entrance (5.9%).

Roughly one-fourth (23.5%) of all respondents indicated no problems had been observed with this type of roadway.

The second question on the survey was "*When designing facilities that do not specifically conform to either urban street or multi-lane rural highway design criteria, what guidelines do you follow?*" Most districts were closely divided on one of the following:

1. Multi-lane rural highway design criteria (23.5%);
2. Urban street design criteria (35.3%); or
3. Combination of urban street design and multi-lane rural highway design criteria (35.3%).

Respondents were next asked to "*State any suggestions or recommendations you feel are appropriate and feasible for roadside 'clear zone' on suburban high-speed curb and gutter sections.*" The answers to this question varied from district to district. The most common recommendation was to provide shoulders whenever possible. Other suggestions given by individual districts were as follows:

1. Provide as much clear zone as possible within the available right-of-way;
2. Increase clear zone beyond curb for high-speed sections;
3. Allow for the clear zone to be at least the required shoulder width with a desirable width of 9.1 m (30 ft);
4. Provide 2.4 m (8 ft) shoulder with mountable curbs and a minimum 2.4 m (8 ft) border width;
5. Provide a 1.8 m (6 ft) minimum, 2.4 to 3.0 m (8 to 10 ft) desirable shoulder, and a 1.8 m (6 ft) minimum, 3.7 m (12 ft) desirable berm behind the curb along with a 4.9 m (16 ft) minimum clear zone;

6. Provide a continuous full width shoulder, and a laydown 0.76 cm (3 in) curb with a 4.6 m (15 ft) clear zone or a 0.15 m (6 in) curb with a 3.0 m (10 ft) clear zone;
7. If shoulders are not feasible, provide frequent turn out lanes for disabled vehicles or bus stops; or
8. Reduce minimum clear zone requirements because of difficulty in attaining these requirements within limited right-of-way.

Finally, the respondents were given a list of factors and were asked to rank the factors as to importance in the selection of design criteria for high-speed suburban roadways with curb and gutter (1 - most important, 2 - second most important, etc.). The following are a list of the factors that were ranked and an overall summary of the results from all the districts that responded.

Rank	Factors
1	Traffic Demand Volume
2	Available Right-of-Way
3	Accident Experience
4	Design/Posted Speed
5	Intersection Sight Distance
6	Drainage Requirements
7	Vehicle Turning Movements
8	Adjacent Land Development
9	Driveway Locations and Frequency
10	Utility Accommodation
11	School Bus Route
12	Bicycles
13	Mail Boxes

Traffic demand volume, available right-of-way, accident experience, and design/posted speed all ranked closely. These roadway factors seemed to be the most important in the districts' selection of design criteria for high-speed curb and gutter sections.

In conclusion, the survey responses identified several operational and design concerns regarding suburban high-speed multi-lane highways. Nearly all of the responses indicated storm water ponding as a potential problem, meaning higher potential for losing control of a vehicle in an outer lane during wet weather. Driveway density was also a concern for engineers due to the high speeds combined with frequent access points and lack of shoulders. Vehicles requiring frequent stops on the roadway (garbage trucks, mail trucks, school buses, and others) are not provided with a safe refuge on high-speed curb and gutter sections. The lack of shoulders also affects pedestrians and bicyclists, forcing them into the traveled way. Another factor indicated on many responses was that clear zone requirements were not properly satisfied in many cases; aside from the hazard of collision with a fixed object, there is the difficulty associated with removing stranded vehicles from the travel lanes without a buffer zone.

2.7 TRAFFIC CONFLICTS TECHNIQUE

While traffic accidents are the most direct measure of highway safety, unreliable accident records and the time required to establish adequate accident sample sizes has led to the development of a surrogate measure of effectiveness. In 1968, Perkins and Harris (33) developed the traffic conflicts technique (TCT) as a means of predicting accident potential at intersection sites. This technique was used to objectively measure the accident potential of a highway location without the need to wait for a suitable accident history to evolve. The development of this technique involved various tasks, including the establishment of traffic conflict definitions, conflict categories, field studies, and statistical evaluation of the results.

In the broadest sense, a *traffic conflict* is a traffic event involving the interaction of two vehicles, where one or both drivers may have to take an evasive action to avoid a collision (34). It is important to note that an operational definition of traffic conflict should be related statistically to safety and provide a reliable and practical means of measuring conflicts. This definition may then be applied to several different conflict categories, dependent upon the study location. Additional conflict definitions have been developed for secondary conflicts and severe conflicts. A secondary conflict may be said to occur when, in an attempt to avoid a conflict, a vehicle actually creates a conflict with a third vehicle. A descriptive definition of a severe conflict was developed, based on a time-to-collision measurement (35). Time-to-collision is defined as the time interval from when a driver anticipating a conflict reacts (brakes or swerves) until a collision would have occurred had there been no reaction. A study of near-miss determination as related to time-to-collision resulted in a mean value of 1.46 seconds as an indicator of a near-miss (36). A severe conflict is defined as a conflict with a time-to-collision value of less than 1.5 seconds.

In their 1968 study, Perkins and Harris established five conflict categories, including left-turn, weaving, cross-traffic, red (brake) light, and rear-end conflicts. In 1980, Glauz and Migletz (34) further established conflict categories that included the following: right-turn, same direction; left-turn, same direction; slow vehicle; opposing left-turn; right-turn from right; cross traffic from right; cross traffic from left; and left-turn from left. A high-speed curb location, for example, might be studied for many categories, including the following: right-turn, same direction; right-turn from right; slow vehicle; opposing left-turn; cross traffic; and left-turn from left.

In a 1972 study and evaluation of the traffic conflicts technique, the Federal Highway Administration performed a field study and arrived at, among others, the following conclusions (37). The data supported the correlation between conflicts and accidents. The TCT can quickly and reliably pinpoint safety deficiencies. Also, the TCT can be applied, with minor modification, to locations other than intersections. Finally, the effectiveness of spot improvements can be quickly evaluated using the TCT. A 1977 critique of the traffic conflicts technique (38) identified three practical applications of the technique. These included the following: the identification and ranking of locations for safety improvements, the diagnosis of specific safety deficiencies in order to determine specific countermeasures, and the evaluation of implemented countermeasures using a before and after study design.

The traffic conflicts technique is a useful tool for detecting safety or operational problems at newly modified locations for which there is no suitable accident history. In order to be useful, total conflict numbers should be divided by traffic volume to produce a conflict rate for a specific high-speed section. Various cross sections may then be compared using standard statistical tests. The TCT is useful for various operational applications including, before/after improvement studies and the evaluation of construction zones, freeway weaving areas, and intersections and interchanges.

3.0 SAFETY STUDIES

The objective of the safety studies was to quantify the safety effects (negative and positive) of barrier curbs on high-speed suburban roadways through the collection and analysis of accident data pertaining to these sections. To accomplish this objective, an accident study was designed to determine the differences in the type, rate, and severity of accidents occurring on high-speed multilane highways with and without barrier curbs.

3.1 STUDY DESIGN

The basic study was designed to examine and analyze accident experience on high-speed suburban multilane highways with a curb and gutter cross section using sites in the state of Texas. The study utilized a before and after structure for these sites. Ten sites were studied in Texas, and at least three years of accident data were collected for each site. The sites were recently modified from a rural parallel drainage ditch design to a curb and gutter cross section design. Accident experience before roadway modification was compared to accident experience after roadway modification.

The roads being compared were paired samples of two populations of highways. Accident data prior to the modification were the control to which accident data after the modification were compared. It was assumed that the populations were normal with identical variances.

There were three measures of effectiveness which were examined to determine if the type, rate, and severity of accidents were different from one population to another. These were accident rate, accident characteristic frequency, and accident severity. The information from these measures of effectiveness provided a comprehensive analysis of the accident experience occurring on these roadway sections.

The change in accident rates is a strong indicator of the effects of a safety related improvement on a highway. Accident rates are often defined as the number of accidents per kilometer per year (accidents per mile per year) on a section of roadway. Therefore, in this study, accident data was converted to accident rates by dividing by the length of the site and the number of years the data spanned.

One important aspect of an accident study is the underlying cause in the increase in accident rates. This increase or cause can often be determined through examination of the percentage of accidents occurring with a certain characteristic. Accident characteristics examined in this study include wet road surface, non-clear weather, and impaired visibility, each of which was identified by roadway designers as a potential cause of increased accident rates on high-speed suburban curb and gutter sections.

In many cases, the accident rate of a certain type of accident may remain constant after a roadway modification; however, the severity of those accidents may increase or decrease. An increase in the severity of a group of accidents indicates that the roadway improvement has caused the road to become less safe.

3.1.1 Site Selection

Due to the fact that Texas utilizes high-speed curb and gutter roadways, maintains a quality accident and roadway database, and was convenient for site inspection, it was selected as the state to furnish the sites for this study. In January 1993, each TxDOT district was questioned through a survey circulated by the Texas Transportation Institute to determine potential sites for this analysis as well as to acquire information concerning current design practices and problems which the design engineers encounter (see Chapter 2.0 for a detailed description of this survey). The responses indicated that there were 193 high-speed curb and gutter sections scattered in 17 Texas districts. These roadways all had a posted speed limit of 80.5 km/h (50 mph) or greater. Completion dates ranged from the 1950's through the 1990's. Several sites utilized two-way left-turn lanes as well as raised medians. Most sites did not have any shoulders; however, some had 2.4 to 3.0 m (8 to 10 ft) shoulders.

In order to compare accident experience at each site before and after curbs and gutters were installed, sites were selected which had completion dates allowing at least one year of data before construction began and one year after the construction ended. This mandated selection of sites with completion dates of 1990 or earlier. In addition, the accident database maintained accident reports after and including 1985. Therefore, with elimination of construction time, the earliest completion date for modification of a site was 1987. Out of the 193 potential sites returned in the survey, 26 had completion dates between 1987 and 1990.

Accident records for these sites were obtained through LANSER (Local Area Network Safety Evaluation and Reporting) for the years 1985 through 1992. An initial screening of the 26 sites revealed that some of the sites were not included in the LANSER database or had no accident data recorded for several years. This initial screening left 10 sites to be analyzed. The sites selected for study encompassed a range of Texas topography. East Texas sites were located in Gregg, Henderson, Rusk, and Smith counties. One west Texas site was located in Tom Green county. Central Texas sites included two sites in Bexar county. Two sites were also located in South Texas in San Patricio and Nueces counties.

The sites varied in length, number of lanes, and driveway density. All of the sites, however, had a minimum of two through travel lanes in each direction, and a minimum posted speed limit of 80.5 km/h (50 mph). None of the selected sites included a paved shoulder. Table 1 lists the geometric attributes of the 11 sites.

Table 1. Geometric Attributes of Study Sites for Safety Studies

County	Highway Number	Cross Section	Speed Limit km/h (mph)	ADT
Tom Green	RM 584	FLUSH MEDIAN	88.5 (55)	5,900
Smith	SH 155	TWLTL[1]	88.5 (55)	11,200
Henderson	SH 31	TWLTL	80.5-88.5 (50-55)	13,000
Rusk	US 79	TWLTL	80.5-88.5 (50-55)	5,700
Smith	SH 31	TWLTL	88.5 (55)	11,900
Gregg	Loop 281	TWLTL	88.5 (55)	18,300
Bexar	IH 410 Frontage Road (1 way)	Median	80.5 (50)	16,003
Bexar	IH 410 Frontage Road (1 way)	Median	80.5 (50)	14,920
Nueces	SH 357	LTL[2]	80.5 (50)	12,900
San Patricio	SH 35	TWLTL	88.5 (55)	10,900

[1]TWLTL - Two-Way Left-Turn Lane
[2]LTL - Left-Turn Lane at Intersection

3.1.2 Data Collection

LANSER is a microcomputer software package that provides access to traffic records data for the State of Texas (39). LANSER was developed by the Texas Transportation Institute (TTI) in cooperation with TxDOT. The accident data entered into the LANSER database is the same information collected by the Department of Public Safety.

The records in the database date from 1985 though the beginning of 1992. They are accessed through a process called subsetting. In essence, one searches the entire database for accidents meeting a certain specification. In this case, certain control sections and milepoints on those control sections were selected as criteria. Once the qualifying records are obtained, LANSER creates a subset of these accident records that have met the required definition. The records within the subset can then be printed or stored in a file. Certain variables can be selected from these records to be placed into a file to be imported into spreadsheet programs or statistical analysis programs. LANSER also can perform frequency distributions on one or two user specified variables contained in the accident report.

Through LANSER, all of the accident records for the necessary years were accessed using the control section and beginning and ending milepoints for each site to provide two years of before data and two years of after data. In the event that additional years of accident information were available, those years were included in the analysis. Certain variables were extracted from these records and placed into a file to be imported into the statistical analysis program SAS. The variables that were chosen were accident number, year, light condition, weather, roadway related, other factor, control/section, milepoint for control section, population group, date, first harmful event, surface condition, intersection related, manner of collision, road class, time, severity of collision, road condition, object struck, number of lanes, shoulder type, ADT for current year, designated speed limit, surface width, shoulder type, and number of vehicles involved. All of these variables for all of the accidents at each site were combined into a single database which was manipulated for the data analysis.

In addition, each site was visited in order to better understand the design and operation of that roadway. This visual inspection provided information on the driveway density and adjacent development at the site and confirmed the information provided in the survey regarding the geometric attributes of the sites.

3.2 ACCIDENT RATES

Accident rates were calculated by dividing the number of accidents occurring at each site by the length of the site and the number of years the data spanned, and are presented in Appendix B in accidents per mile per year. Table 2 summarizes the mean accident rates before and after site modification as well as the difference between the two accident rates.

As shown in Table 2, sites 1 through 6 experienced a decrease in mean accident rates, whereas sites 7 through 10 experienced an increase in the mean accident rates after site modification to a curb and gutter section. The overall mean accident rate combined for all sites decreased by 0.44 accidents per mile per year.

To determine if the mean accident rates for certain types of accidents were significantly different for sites with and without curbs, paired one-sided t-tests were performed on the data using the SAS MEANS procedure. The research hypothesis was that the accident rates for roadways with barrier curbs were not equal to the accident rates for roadways without the barrier curb, depending on the mean accident rate for that accident type. Differences were deemed significant at the 95 percent confidence level.

Table 3 is a summary of accident rates for the sites before and after roadway modification for the 13 different accident types. Also listed is the raw and percent difference in the accident rates, the percentage of sites experiencing a difference with the same sign as the mean difference, and the p-value for the t-tests conducted.

Table 2. Differences In Mean Accident Rates

Site Number	Accident Rate Before Acc/Km/Yr (Acc/Mi/Yr)	Accident Rate After Acc/Km/Yr (Acc/Mi/Yr)	Difference Acc/Km/Yr (Acc/Mi/Yr)
1	15.91 (4.85)	14.83 (4.52)	-1.08 (-0.33)
2	31.53 (9.61)	28.51 (8.69)	-3.02 (-0.92)
3	54.40 (16.58)	21.16 (6.45)	-33.24 (-10.13)
4	28.84 (8.79)	25.43 (7.75)	-3.41 (-1.04)
5	25.16 (7.67)	24.87 (7.58)	-0.29 (-0.10)
6	34.68 (10.57)	29.89 (9.11)	-4.79 (-1.45)
7	27.07 (8.25)	33.63 (10.25)	6.56 (2.00)
8	16.83 (5.13)	18.86 (5.75)	2.03 (0.63)
9	16.44 (5.01)	29.59 (9.02)	13.15 (4.01)
10	25.95 (7.91)	44.91 (13.69)	18.96 (5.77)
Average	24.60 (7.50)	23.16 (7.06)	-1.44 (-0.44)

As shown in Table 3, the types of accidents which experienced an increase were nighttime accidents, impaired weather accidents, impaired visibility accidents, run-off-road accidents, accidents on wet roadways surfaces, and accidents due to striking the curb. The accident rates which experienced a decrease were exiting vehicle accidents, bicycle and pedestrian accidents, sideswipes, and rear-end accidents.

Only two types of accident rates experienced a statistically significant increase at the 95 percent confidence level: run-off-road accidents, and run-off-road into fixed object accidents. Even though in many cases 70 percent of the sites indicated the same effect of the curb and gutter on a specific accident rate, there was too much variance in the data for mean accident rates to be significantly different. The p-value for the overall combined mean accident rate for all sites did not indicate a statistically significant decrease.

In order to determine if the large variance in the mean accident rates was caused by a difference in the nature of the sites, several models were applied to test for any differences in the accident rates. The following three operational and geometric variables were examined: ADT, difference in the before and after configurations of sites, and driveway density.

Table 3. Mean Accident Rates and Descriptive Statistics by Accident Type

Accident Type	Accident Rate Acc/Km/Yr (Acc/Mi/Yr)		Difference Acc/Km/Yr (Acc/Mi/Yr)		Percent of Sites	p-value
	Before (No Curb)	After (Curb)	Raw	Percent		
All	24.61 (7.50)	23.16 (7.06)	-1.44 (-0.44)	-5.8	60	.455
Nighttime	7.38 (2.25)	8.27 (2.52)	0.92 (0.28)	12.27	70	.137
Bad Weather	2.85 (0.87)	3.45 (1.02)	0.49 (0.15)	17.16	80	.426
Raining	2.56 (0.78)	2.89 (0.88)	0.30 (0.09)	11.79	60	.477
Bad Visibility	9.32 (2.84)	10.27 (3.13)	0.95 (0.29)	10.21	90	.455
Run-off-road	0.92 (0.28)	1.25 (0.38)	0.33 (0.10)	35.03	70	.039
Fixed Object	0.49 (0.15)	0.79 (0.24)	0.30 (0.09)	57.54	60	.025
Exiting Vehicle	2.82 (0.86)	2.43 (0.74)	-0.39 (-0.12)	-14.35	40	.463
Bike/Pedestrians	0.33 (0.10)	0.26 (0.08)	-0.39 (-0.12)	-24.04	50	.198
Sideswipes	3.15 (0.96)	2.43 (0.74)	-0.75 (-0.23)	-23.48	70	.442
Rear-Ends	2.79 (0.85)	2.23 (0.68)	-0.52 (-0.16)	-19.19	70	.443
Wet Surface	3.54 (1.08)	5.25 (1.60)	1.71 (0.52)	47.89	70	.255
Strike Curb	0.20 (0.06)	0.36 (0.11)	0.16 (0.05)	75.54	40	.460

3.2.1 Average Daily Traffic

The ADT values were tested to determine if they exhibited a linear relationship with the change in the accident rates for all accident types under investigation. Two regression models were run through the SAS REG (regression) procedure using the continuous variable ADT to model the difference in accident rate. The r^2 values resulting from the model for each accident type tested are listed in Table 4. F-tests were conducted in the REG procedure to determine what effect ADT had on the variance among the accident rates. The resulting p-values are listed in Table 4.

The r^2 values for the data are extremely low, even for studies dealing with accidents. The high p-values associated with the model also indicate that for those sites, ADT was not a good predictor of how accident rates change when curbs and gutters are placed on high-speed suburban roadways.

Table 4. Coefficient of Determination (r^2) Values from Regression Model

Accident Type	r^2 value	p-value
All	0.0004	0.9589
Nighttime	0.0005	0.9514
Bad Weather	0.0001	0.9732
Raining	0.0014	0.9196
Bad Visibility	0.0004	0.9589
Run-Off-Road	0.0824	0.4214
Fixed Object	0.0357	0.6010
Exiting Vehicle	0.0272	0.6491
Bike/Pedestrians	0.0595	0.4972
Sideswipes	0.0033	0.8755
Rear-Ends	0.0478	0.5400
Wet Surface	0.0098	0.7853
Strike Curb	0.0117	0.7665

3.2.2 Site Configuration and Driveway Density

The study sites can be grouped into two distinct types of modifications: those in which the number of lanes and the shoulder width remained the same and those in which lanes were added and the shoulder width was reduced. Driveway density for the sites falls into three categories: high, low, and frontage road. In order to determine whether either of these variables affected the change in the accident rates, general linear models were developed using the GLM (general linear model) procedure in SAS.

A general linear model was run for three accident types: all accidents, run-off-road accidents, and run-off-road into fixed object accidents. The difference between the accident rates from before to after the site was modified was the dependent variable, and the site configuration and driveway density were the independent variables used in the models. The model included the two class variables, site configuration and driveway density, and their interaction.

The three general linear models for the three accident types provided means for the difference between each level of driveway density and each different site configuration change. These means as well as the p-values resulting from the f-tests are listed in Tables 5 and 6.

From Table 5, it appears that the mean accident rate difference for all accidents increased for sites with low driveway density and decreased for sites with high driveway density. The mean accident rates for run-off-road accidents also increased with low driveway density; however, it increased less with high driveway density. For run-off-road into fixed object accidents, the mean accident rates for sites with low driveway density increased slightly, and increased less for sites with high driveway density. For each of the three accident types tested, the mean accident rates for sites with frontage roads increased. According to the model, for all accidents, it can be said that driveway density significantly affects the difference in accident rates only to about a 90 percent confidence level.

Driveway density, nonetheless, appears from this data to be a large factor in the determination of the effect of placing curbs and gutters on a roadway, especially when coupled with the effect of ADT. Site 1 had a very low driveway density, and experienced a slight reduction in accident rate upon modification. Sites 2 through 6 all had very high driveway densities and all experienced a decrease in their mean accident rate upon modification to a curb and gutter section. Sites 7 through 10 all had very low driveway densities, with sites 7 and 8 being frontage roads, and all experienced an increase in accident rate upon modification. Site 1 had a very low volume and sites 7 through 10 had fairly high volumes, which may explain the reduction in accident rate for site 1 and the increase in accident rate for sites 7 through 10 even though they both had low driveway densities.

Table 5. General Linear Model Results: Driveway Density

Accident Type and p-value	Driveway Density Level	Mean Difference In Accident Rate Acc/Km/Yr (Acc/Mi/Yr)
All Accidents	Low	10.37 (3.16)
p = 0.133	High	-8.92 (-2.72)
	Frontage	4.33 (1.32)
Run-Off-Road Accidents	Low	1.21 (0.37)
	High	0.62 (0.19)
p = 0.848	Frontage	0.56 (0.17)
Run-Off-Road Into Fixed Object Accidents	Low	0.85 (0.26)
	High	0.56 (0.17)
p = 0.918	Frontage	0.85 (0.26)

Table 6. General Linear Model Results: Configuration Change

Accident Type and p-value	Configuration Change	Mean Difference In Accident Rate Acc/Km/Yr (Acc/Mi/Yr)
All Accidents p = 0.116	No Change	5.54 (1.69)
	Lanes Add/Shld Rdc	-6.53 (-1.99)
Run-Off-Road Accidents p = 0.238	No change	1.18 (0.36)
	Lanes Add/Shld Rdc	0.39 (0.12)
Run-Off-Road Into Fixed Object Accidents	No Change	0.82 (0.25)
p = 0.234	Lanes Add/Shld Rdc	0.59 (0.18)

The p-values for the site configuration variable were much smaller than those for the driveway density; however, they still were not significant at the 95 percent confidence level. For each of the three accident rates tested, the mean accident rates for those sites which had lanes added and shoulders reduced had either a smaller increase or a decrease compared to those sites which had no changes. This finding indicates that the addition of lanes and reduction of shoulder width when a site is modified to a curb and gutter section may indeed result in relatively lower accident rates and improve the safety of the section.

3.3 ACCIDENT SEVERITIES

Even though the accident rate may remain constant when a site is modified to a high-speed curb and gutter section, there may be an effect on the severity of a certain accident type. For example, with respect to run-off-road accidents, with a curb and gutter in place, it is more difficult for an errant vehicle to regain control once leaving the roadway. Therefore, the accident may be more severe.

Relative frequencies were calculated to determine the percentage of accidents at each severity level before and after site modification. These values are listed in Appendix B. Figures 1 and 2 depict the percentages of accidents with no injuries and with fatalities for all accidents and for run-off-road accidents for the collected data. The figures show that the percentage of all accidents and run-off-road accidents with no injuries decreased, and that the percentage of all accidents and run-off-road accidents with fatalities increased.

Three loglinear models were run using the SAS CATMOD (categorical model) procedure to determine if severity and the type of road interact. The research hypothesis was that interaction between road type and severity exists. The ANOCAT tables for severity in Appendix B, for each accident type, all accidents, run-off-road accidents, and run-off-road into fixed object accidents, indicate that the severities of the accidents did not significantly change.

3.4 ACCIDENT CHARACTERISTIC FREQUENCIES

For this study, it was desirable to determine the underlying cause of any increase or decrease in accident rates. The mean accident rates for the sites indicated that run-off-road accidents experienced a statistically significant increase when a curb and gutter was added to the section. There may be several causes for this increase in accident rate, including storm water ponding, poor curb visibility during nighttime or impaired weather, or intersection related issues.

In order to determine whether these factors were causing an increase in accidents, frequency tables were prepared depicting the relative frequency distributions of the variables described in Section 3.1. These tables are included in Appendix B. Loglinear models were created and tested for each of the variables listed to determine whether there existed any interaction between that variable and the road type. Three models were run for each variable, one for all accidents, one for run-off-road accidents, and one for run-off-road into fixed object accidents.

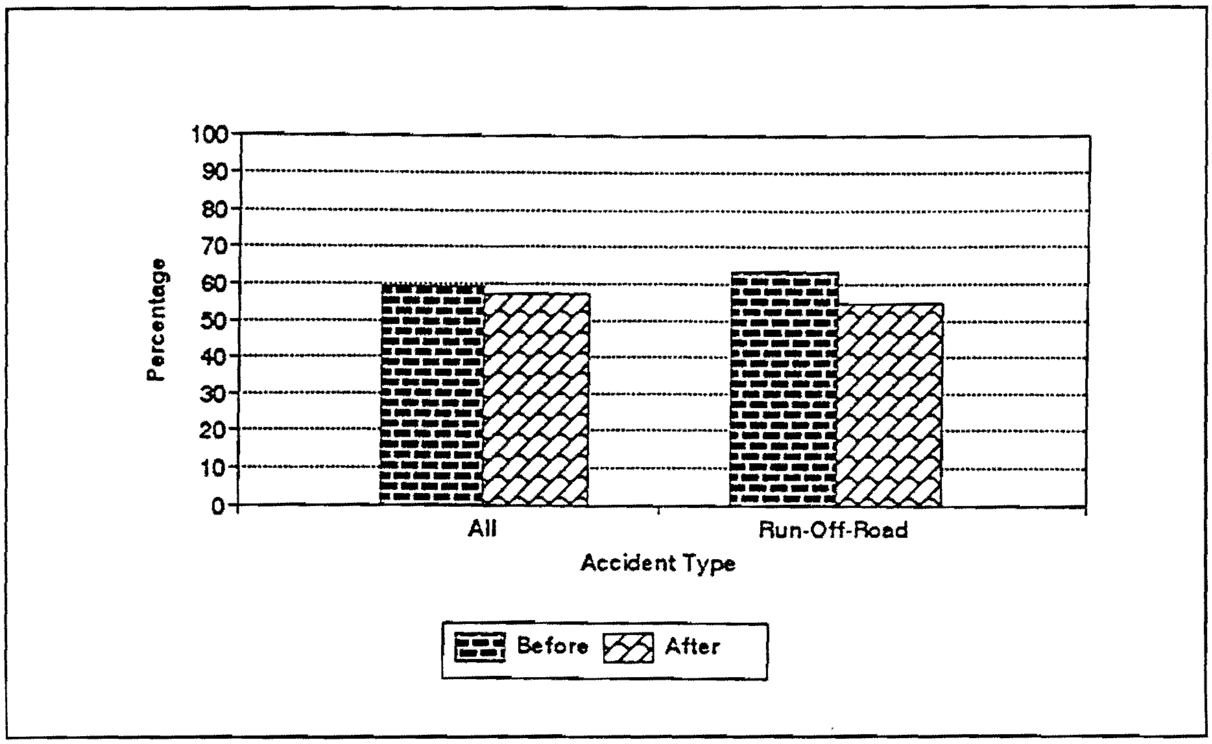

Figure 1. Percentage of Accidents with No Injuries

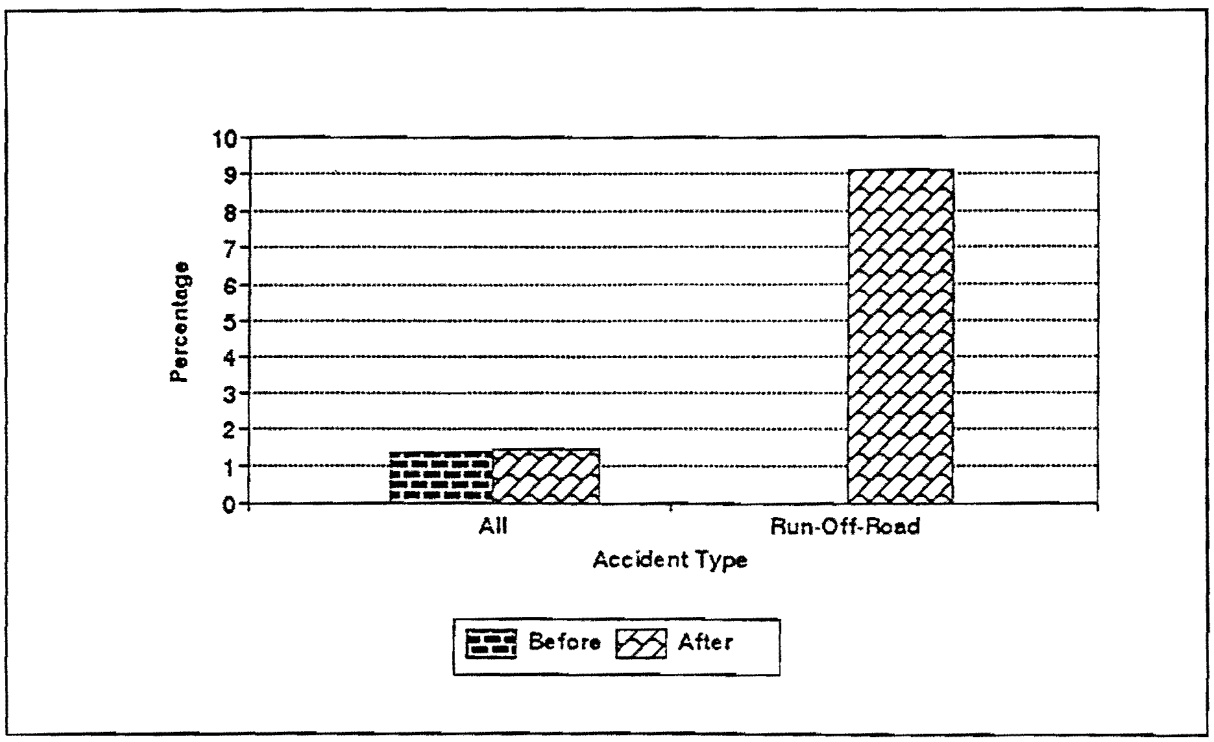

Figure 2. Percentage of Accidents with Fatalities

3.4.1 Visibility

In order to determine whether there was a problem with drivers unable to see the curb during poor weather or lighting conditions, the percentage of poor visibility accidents before the roadway modification was compared to the percentage after the modification. The visibility variable was divided into two classes: impaired visibility and non-impaired visibility. Impaired visibility indicates that the weather was not clear, that the accident did not occur during daylight, or both.

Figure 3 depicts the percentage of accidents with impaired visibility for all accidents and for run-off-road accidents both before and after modification. This figure shows that the percentage of all accidents at sites with impaired visibility increased after modification to a curb and gutter section.

For all accidents, loglinear models indicated that the presence of curbs affected the percentage of impaired visibility accidents, but that curbs did not affect the percentage of impaired visibility accidents for run-off-road accidents or for run-off-road into fixed object accidents.

3.4.2 Lighting

To determine whether there was a problem with people being unable to detect the curb specifically due to poor lighting conditions, the percentage of accidents occurring when it was not daylight was compared before and after roadway modification. For lighting, there were five categories utilized in the accident report. These five levels were condensed into two for this model: daylight and non-daylight. Figure 4 shows the percentage of accidents occurring during non-daylight, or nighttime, for all accidents and for run-off-road accidents at the sites before and after modification. As shown in Figure 4, the percentage of all accidents occurring at nighttime increased.

For all accidents, loglinear models indicated that curbs did not affect the percentage of nighttime accidents. The models also indicated that curbs did not affect the percentage of nighttime run-off-road accidents or run-off-road into fixed object accidents.

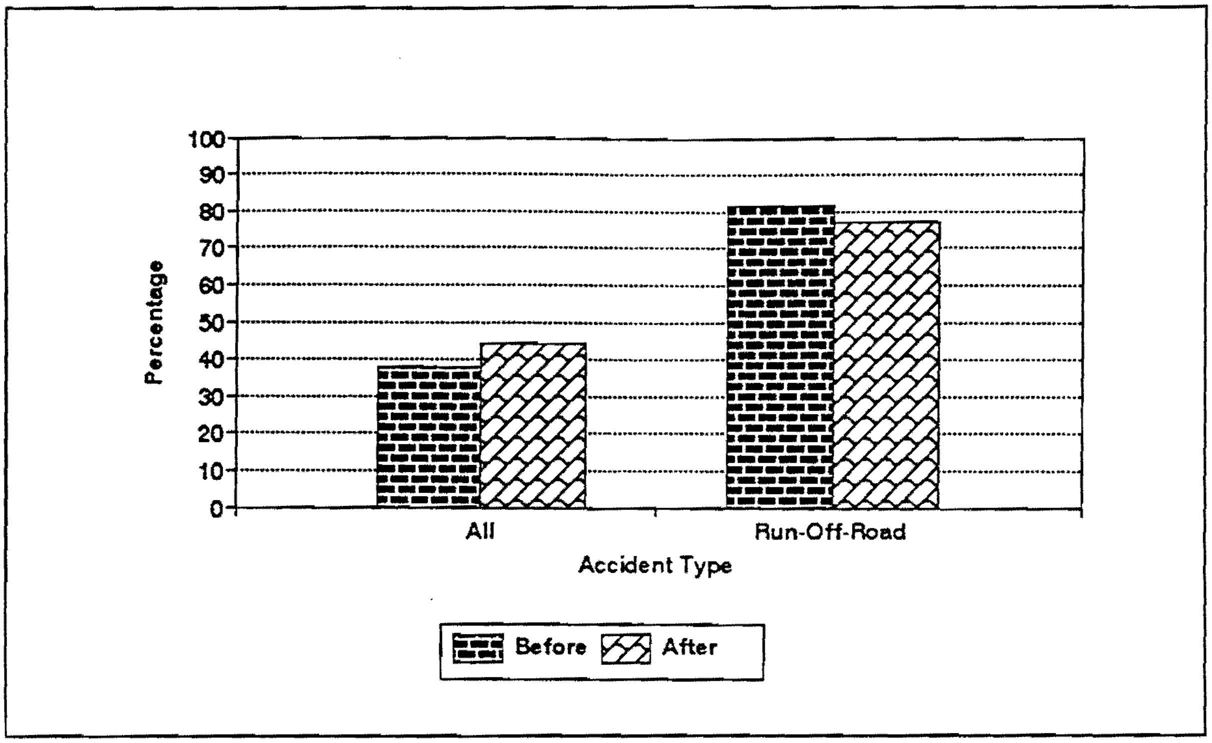

Figure 3. Percentage of Accidents with Impaired Visibility

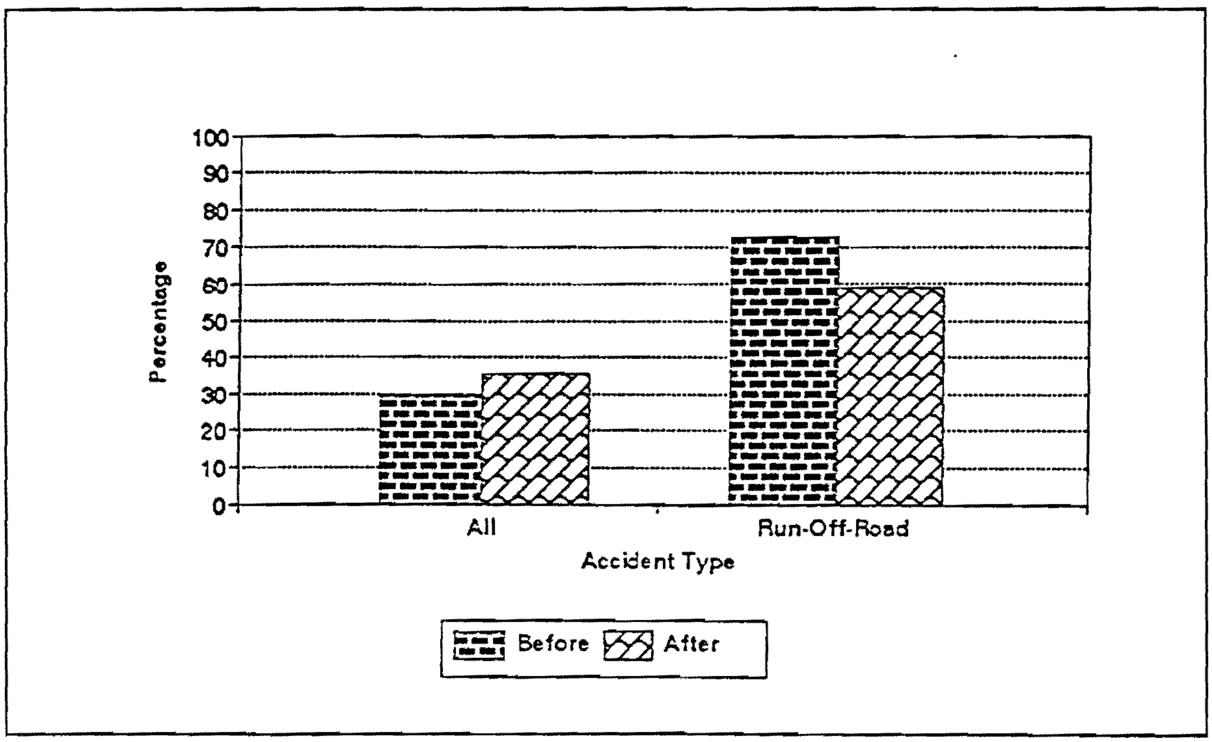

Figure 4. Percentage of Accidents During Nighttime

3.4.3 Surface

To investigate the effects of storm water ponding causing safety problems, it was desirable to determine if a higher percentage of accidents was occurring on a slick or wet roadway surface after the roadway was modified to curb and gutter cross section. There were three categories for the surface variable: dry, wet and other. Other indicated either ice or snow present on the road. The percentages of wet roadway surface accidents and wet roadway surface run-off-the-road accidents occurring before and after modification are shown in Figure 5. This figure shows that the percentage of all accidents occurring on a wet roadway surface was higher for curbed sites than for non-curbed sites. For run-off-road accidents, the percentage of accidents occurring on a wet roadway surface was lower after modification. The rate of all accidents occurring on a wet roadway surface was higher for curbed sites than for non-curbed sites.

Loglinear models indicated that the percentage of accidents occurring on wet roadway surfaces was not affected by the presence of curbs for all accidents, run-off-road accidents, or run-off-road into fixed object accidents.

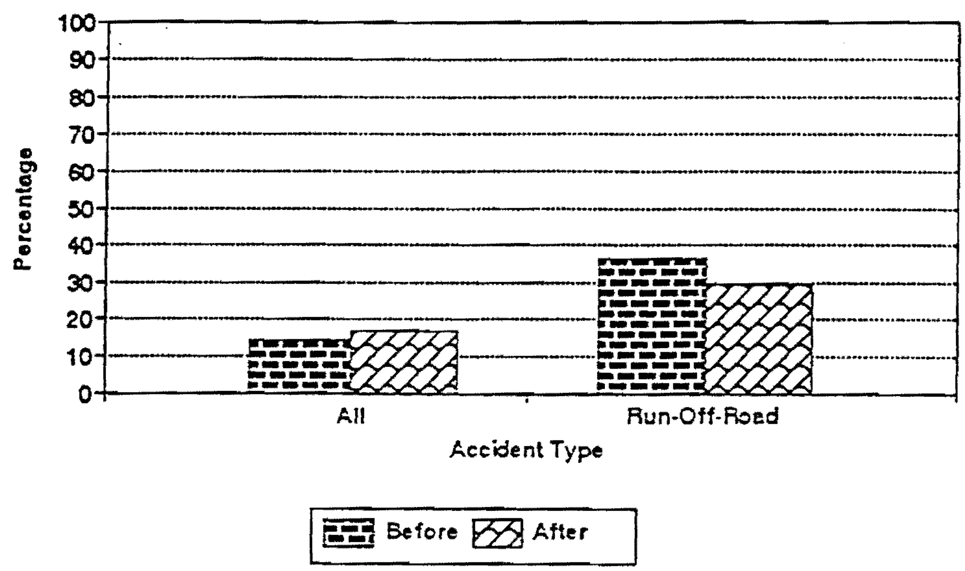

Figure 5. Percentage of Accidents on Wet Road Surfaces

3.4.4 Intersection Related

The percentage of accidents occurring at an intersection was also tested to determine whether there was a different percentage of accidents occurring at intersections due to the addition of curbs and gutters. The intersection variable extracted from the accident reports was defined as intersection, intersection related, driveway access, or non intersection. Figure 6 portrays the percentage of accidents which were intersection related. The figure shows that there was a slight increase in the percentage of intersection related accidents for all accidents and for run-off-road accidents after roadway modification.

According to loglinear models, for all three accident types tested (all accidents, run-off-the-road accidents, and run-off-the-road into fixed object accidents), curbs did not affect the percentage of intersection related accidents.

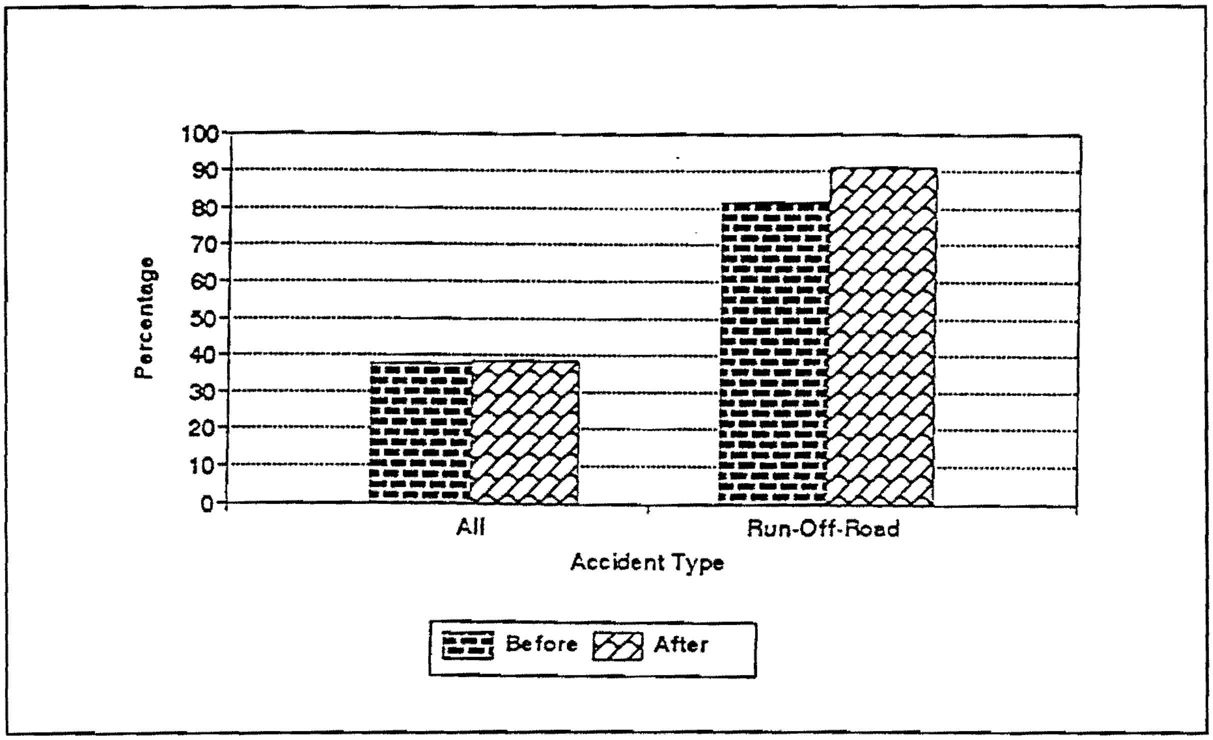

Figure 6. Percentage of Accidents Intersection Related

4.0 OPERATIONAL STUDIES

The objective of the operational studies was to evaluate the effect of different cross sections on traffic operations. In order to realize this objective, one portion of this study was designed to determine whether the presence of a paved shoulder would have any effect on conflict frequency, lane distribution, or speed. The second portion was designed to determine two-way left-turn lane requirements.

4.1 STUDY DESIGN

The study procedure used in this research was a comparison of two independent populations, one being high-speed curb sections without shoulders and the second being sections with shoulders. The dependent variables in this study are conflict rates, lane distributions, and free-flow speeds. Control variables include posted speed limit, level of side road access, length of study site, number of through travel lanes, and the presence of traffic control devices. In the comparison of these independent samples, it was assumed that they were from normal populations with identical variances. These two samples could then be compared and inferences drawn about their respective means.

The operational characteristics which were addressed in this research included conflict rates, lane distributions, and free-flow speeds. The ultimate measure of effectiveness in any safety related highway improvement would be a reduction in the accident rates. Due to the time required to establish a suitable accident sample size, conflict rates were used in this study as a surrogate measure of effectiveness. Conflict rates, per 1000 vehicles, were determined at each location for the morning peak and the evening peak. By comparing the conflict rates associated with a section containing a paved shoulder to those in a section without a paved shoulder, it could be determined whether the presence of a paved shoulder results in a lower conflict rate and a corresponding increase in the safety effectiveness of the section.

If drivers are expecting to encounter heavy turning movements for ingress and egress from a section of roadway, they might tend to avoid the right lane in order to also avoid those turning movements. Drivers might also tend to avoid driving in a high-speed lane adjacent to a curb in order to distance themselves from any obstacles behind the curb. Traffic volumes were collected on a lane by lane basis to determine the proportion of vehicles in the right lane. By comparing the lane distributions in both types of cross section, it could be determined whether the presence of a paved shoulder has any effect on how drivers make their lane choice.

The free-flow speeds studied in this research were measured in a spot speed study at each location. These speeds are a measure of the level of service at which a particular highway section is operating. By comparing the 85th percentile speeds to the posted speeds in the two types of sections, it may be determined if the presence of a paved shoulder will have an effect on the level of service of a section as measured by free-flow running speed.

sections, it may be determined if the presence of a paved shoulder will have an effect on the level of service of a section as measured by free-flow running speed.

4.1.1 Site Selection

In order to collect data for analysis, a field study was designed and conducted. This field study included the selection of sites, definition and categorization of conflicts, and the establishment of a data collection procedure.

Field data collection sites were selected based on pavement cross section, geographic location, posted speed limits, and intensity of adjacent land use activities. From the questionnaire sent to various TxDOT districts (see Chapter 2.0), 16 sites were selected for study, encompassing a range of Texas topography. Selected sites included pairs of roadways in east, west, and central Texas, the panhandle region, and the Gulf Coast region. East Texas sites were located in Gregg, Henderson, Rusk, and Smith counties. The two west Texas sites were located in Tom Green and Mills counties. In central Texas, Bastrop, Comal, and Williamson counties were selected. Two sites in Lamb county were observed in the panhandle region. Calhoun, Nueces, San Patricio, and Victoria counties were chosen in the Gulf Coast region. The locations of these counties are shown in Figure 7.

All roadway sites selected had a minimum posted speed limit of 80.5 km/h (50 mph) and included a minimum of two through travel lanes in each direction. One location per pair included a paved shoulder of a minimum 2.4 m (8 ft) width. The second location did not include a paved shoulder. In order to account for the control variables, the intensity of adjacent development was kept reasonably similar between pairs. In order to maintain this similarity, the length of the study sites had to be variable. The lengths varied from 161.5 to 313.9 m (530 to 1030 ft). All sites were through travel sections with no traffic control devices in the direction of travel being observed. Traffic volumes, as provided by TxDOT, were similar between pairs of roadways. Before visiting the data collection sites, the resident engineer most familiar with the location was contacted in order to obtain traffic counts and become familiar with any special situations or complications which might arise. Table 7 lists the various study sites and their geometric attributes.

4.1.2 Data Collection

In order to determine a conflict rate per 1000 vehicles, it was necessary to establish traffic volumes at each site. Traffic was observed at each location for three hours beginning at 7:00 a.m. and again for three hours beginning at 3:00 p.m. The intent was to record traffic operations during the morning and evening peak periods. During the observation period, total one-way traffic volumes were counted and recorded in fifteen minute periods. By conducting fifteen minute traffic counts, it was possible to determine at what point the peak hour was observed. These volumes were recorded lane by lane in order to determine a lane distribution. A separate count was maintained of heavy vehicles in order to determine the percentage of these vehicles in the traffic stream. A heavy vehicle was defined as any vehicle with more than two axles.

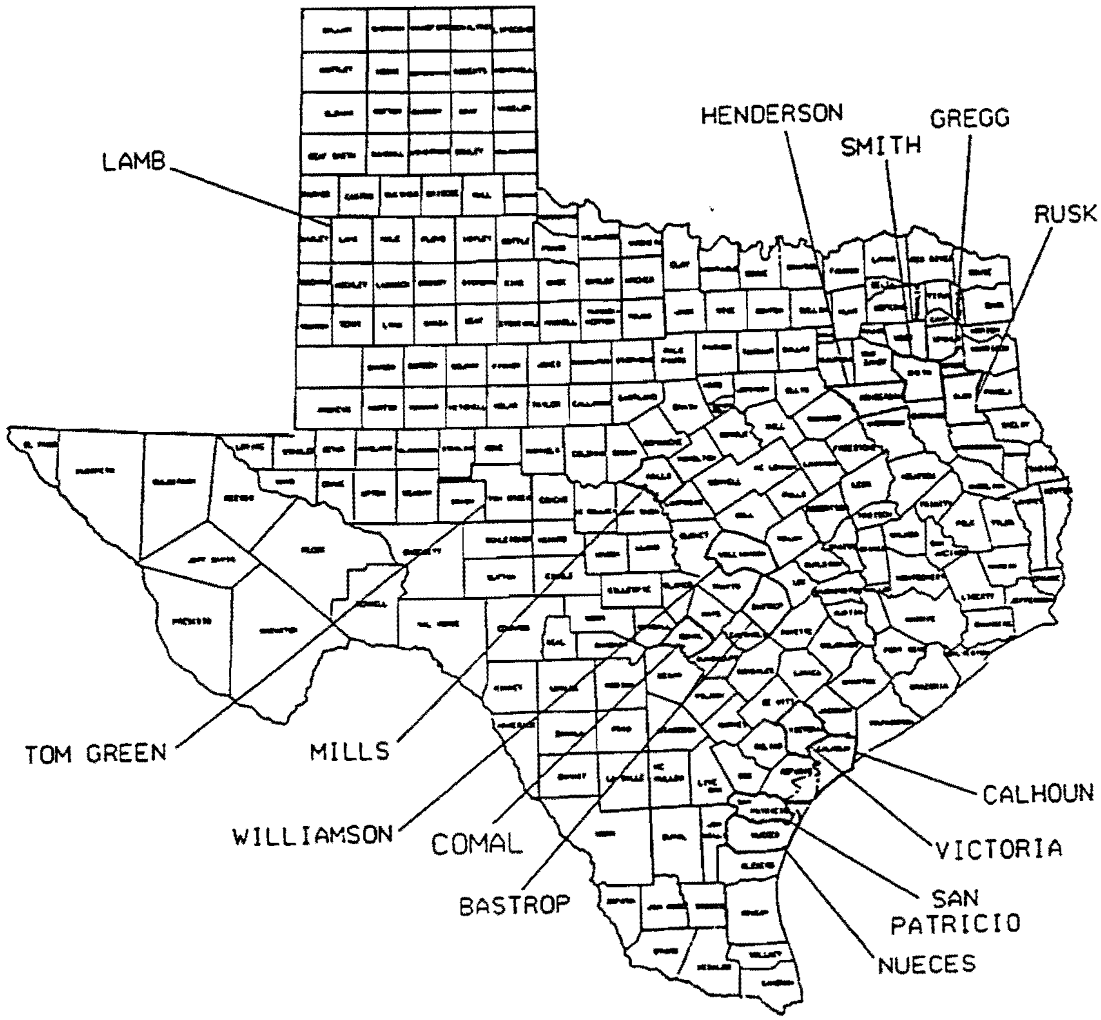

Figure 7. Location of Study Sites for Conflict Analysis

Table 7. Geometric Attributes of Study Sites for Conflict Analysis

Site Type	County	Highway Number	Cross Section	Shoulder Width m (ft)	ADT	Speed Limit km/h (mph)
Shoulder No TWLTL[1]	Bastrop	SH 21	4-lane, Undivided	3.0 (10)	21,000	80.5 (50)
	Calhoun	SH 35	4-lane, Divided	3.0 (10)	12,500	80.5 (50)
	Lamb	Loop 430	4-lane, Undivided	2.4 (8)	1,750	80.5 (50)
	Mills	US 84	4-lane, Undivided	3.0 (10)	3,500	88.5 (55)
	Williamson	US 79	4-lane, Undivided	3.0 (10)	5,500	80.5 (50)
Shoulder With TWLTL	Smith	SH 64	4-lane, TWLTL	3.0 (10)	8,900	88.5 (55)
	Victoria	SH 185	4-lane, TWLTL	2.4 (8)	10,200	88.5 (55)
No Shoulder With TWLTL	Comal	SH 46	4-lane, TWLTL	None	8,200	88.5 (55)
	Gregg	Loop 281	5-lane, TWLTL	None	18,300	80.5 (50)
	Henderson	SH 31	4-lane, TWLTL	None	13,000	88.5 (55)
	Lamb	US 84	4-lane, TWLTL	None	4,700	80.5 (50)
	Nueces	FM 2444	4-lane, TWLTL	None	9,200	80.5 (50)
	Rusk	US 79	4-lane, TWLTL	None	5,700	80.5 (50)
	San Patricio	SH 35	4-lane, TWLTL	None	10,900	88.5 (55)
	Smith	SH 155	4-lane, TWLTL	None	11,200	88.5 (55)
	Tom Green	RM 584	4-lane, TWLTL	None	5,900	88.5 (55)

[1] TWLTL - Two-Way, Left-Turn Lane

During the counting period, conflicts were also being observed. If any conflict, as defined in this study, was observed, it was recorded on the data collection form at the appropriate fifteen minute time period. Once a braking maneuver was detected, the vehicle was tracked until it left the study area to assure that the braking was due to a conflict. If the braking vehicle continued on through the study area after the instigating vehicle had exited, a conflict was recorded. In those cases where the braking vehicle followed the instigating vehicle in exiting the roadway, no conflict was recorded, due to the fact that it was impossible to determine whether the braking was an evasive maneuver or a slowing maneuver preceding the turn.

Following the three hour morning traffic count and preceding the three hour afternoon count, speed data was collected in the same direction as the conflict data. Radar guns, provided by the Texas Transportation Institute, were utilized. Speed data was collected for one hour or for sixty vehicles, whichever came first. In order to record a reasonable representation of free-flow speeds, only those vehicles in which the driver was choosing his or her own speed were recorded. When a platoon of vehicles passed through the study site, the speed of only the lead vehicle was recorded, since the following vehicles might not have been choosing their own speeds.

The radar gun was turned on only when recording a speed. By turning the unit off until a vehicle approached, no continuous radar signal could be detected. Speeds were recorded for both passenger cars and heavy vehicles.

4.2 SPEED STUDY

In the Texas Department of Transportation publication, "Procedure for Establishing Speed Zones" (40), reference is made to the 85th percentile speed. This speed is used by many states and cities to establish regulatory speed zones, based on the belief that a large majority of drivers are safe and prudent and that they desire to reach their destination in the shortest possible time. The speed at or below which 85 percent of drivers operate their vehicles is considered the maximum safe speed for a given location.

By observing and recording free-flow speeds, the 85th percentile speed may be determined. This speed may then be compared to the posted speed in each section and an average difference determined. This difference may then be compared between cross sections to determine if there is any disparity in the differences. The same statistical tests used to compare conflict rates may be used to compare speed observations. The equal variance assumption must be confirmed or denied as it was in the conflict rate comparison before the 85th percentile speeds from both types of sections are compared to determine if the paved shoulder has a significant effect on vehicle speeds.

Free-flow speeds were recorded at each site to determine whether the presence of a paved shoulder would have any effect. For each site, mean speed and 85th percentile speeds were determined and have been summarized in Table 8 and Table 9. The collected data sheets have been included in Appendix C. The difference between posted speed and the 85th percentile speed was determined for each location and an average difference was calculated. This difference can be compared between sites with shoulders and sites without shoulders to determine if there was a

Table 8. Speed Data for Sites with Paved Shoulder

Location	Speed			
	Mean Speed km/h (mph)	85th Percentile km/h (mph)	Posted Speed km/h (mph)	85th %-ile - Posted Speed km/h (mph)
Bastrop A.M.	83.98 (52.2)	88.5 (55)	80.5 (50)	+8.0 (+5)
Bastrop P.M.	81.85 (50.9)	90.1 (56)	80.5 (50)	+9.6 (+6)
Calhoun A.M.	75.67 (47.0)	82.1 (51)	80.5 (50)	+1.6 (+1)
Calhoun P.M.	76.19 (47.3)	82.1 (51)	80.5 (50)	+1.6 (+1)
Lamb (430) A.M.	72.81 (45.2)	82.1 (51)	80.5 (50)	+1.6 (+1)
Lamb (430) P.M.	74.63 (46.4)	85.3 (53)	80.5 (50)	+4.8 (+)3
Mills A.M.	80.32 (49.9)	88.5 (55)	88.5 (55)	+0.0 (+0)
Mills P.M.	83.59 (51.9)	90.1 (56)	88.5 (55)	+1.6 (+1)
Smith (64) A.M.	86.94 (54.0)	96.6 (60)	88.5 (55)	+8.0 (+5)
Smith (64) P.M.	89.43 (55.6)	96.6 (60)	88.5 (55)	+8.0 (+5)
Victoria A.M.	77.59 (48.2)	85.3 (53)	88.5 (55)	-3.2 (-2)
Victoria P.M.	79.15 (49.2)	88.5 (55)	88.5 (55)	+0.0 (+0)
Williamson A.M.	75.32 (46.8)	82.1 (51)	80.5 (50)	+1.6 (+1)
Williamson P.M.	73.16 (45.5)	80.5 (50)	80.5 (50)	+0.0 (+0)

Observation Periods (n_1)	14
Mean Diff. in Speeds (y_1), km/h (mph)	3.11 (1.9)
Standard Deviation (s_1), km/h (mph)	3.91 (2.4)
Variance (s_1^2), km/h (mph)	9.53 (5.9)

Table 9. Speed Data for Sites with No Paved Shoulder

Location	Speed			
	Mean Speed km/h (mph)	85th Percentile km/h (mph)	Posted Speed km/h (mph)	85th %-ile - Posted Speed km/h (mph)
Comal A.M.	81.38 (50.6)	86.9 (54)	88.5 (55)	-1.6 (-1)
Comal P.M.	84.62 (52.6)	93.3 (58)	88.5 (55)	+4.8 (+3)
Gregg A.M.	77.04 (47.9)	83.7 (52)	80.5 (50)	+3.2 (+2)
Gregg P.M.	77.55 (48.2)	86.9 (54)	80.5 (50)	+6.4 (+4)
Henderson A.M.	82.88 (51.5)	91.7 (57)	88.5 (55)	+3.2 (+2)
Henderson P.M.	81.77 (50.8)	88.5 (55)	88.5 (55)	+0.0 (+0)
Lamb (84) A.M.	76.96 (47.8)	83.7 (52)	80.5 (50)	+3.2 (+2)
Lamb (84) P.M.	77.68 (48.3)	85.3 (53)	80.5 (50)	+4.8 (+3)
Nueces A.M.	75.53 (46.9)	83.7 (52)	80.5 (50)	+3.2 (+2)
Nueces P.M.	75.60 (44.0)	83.7 (52)	80.5 (50)	+3.2 (+2)
Rusk A.M.	74.91 (46.6)	82.1 (51)	80.5 (50)	+1.6 (+1)
Rusk P.M.	74.75 (46.5)	82.1 (51)	80.5 (50)	+1.6 (+1)
San Patricio A.M.	74.64 (46.4)	80.5 (50)	88.5 (55)	-8.0 (-5)
San Patricio P.M.	75.03 (46.6)	83.7 (52)	88.5 (55)	-4.8 (-3)
Smith (155) A.M.	86.65 (53.8)	93.3 (58)	88.5 (55)	+4.8 (+3)
Smith (155) P.M.	87.56 (54.4)	95.0 (59)	88.5 (55)	+6.4 (+4)
Tom Green A.M.	81.67 (50.8)	90.1 (56)	88.5 (55)	+1.6 (+1)
Tom Green P.M.	84.73 (52.7)	93.3 (58)	88.5 (55)	+4.8 (+3)

Observation Periods (n_2)	18
Mean Diff. in Speeds (y_2), km/h (mph) Standard	2.14 (1.3)
Deviation (s_2), km/h (mph)	3.79 (2.4)
Variance (s_2^2), km/h (mph)	9.90 (5.5)

greater difference in posted speed and 85th percentile speed from one type of cross section to the next. Differences were deemed significant at the 95 percent confidence level.

A pooled t-test was applied to test the research hypothesis that there were discrepancies in the differences between 85th percentile speed and posted speed from one cross section to the other. The results yielded a p-value that was not significant at the 95 percent confidence level. For each site, mean speed and 85th percentile speeds were determined and have been summarized in Table 8 and Table 9. The collected data sheets have been included in Appendix C. The difference between posted speed and 85th percentile speed was determined for each location and an average difference calculated. This difference can be compared between sites with shoulders and sites without shoulders to determine if there was a greater it was concluded that there was no significant difference between the 85th percentile speed and posted speed from sites with shoulders and sites without shoulders.

4.3 LANE DISTRIBUTIONS

Traffic volumes were collected at each site on a lane by lane basis in order to establish lane distribution characteristics. This information was used to determine an average lane distribution for those sites with and without a paved shoulder, to determine whether drivers were avoiding the use of the right lane for any reason. Reasons for avoiding the right lane could include the expectation of slow moving or turning vehicles or the avoidance of behind the curb obstacles. For peak hour volumes below 400, there appeared to be a definite increase in the percentage of vehicles in the right lane for those locations which included a paved shoulder. The data was more closely grouped at peak hour volumes above 400. For those sections with shoulders, the right lane proportion was consistently greater than 50 percent at lower volume and then declined towards 50 percent as volume increased. For those sites without shoulders, the right lane proportion was generally below 50 percent throughout the range of volume. The total volumes and right lane proportions have been compiled in Table 10.

In order to determine if there was a different proportion of vehicles in the right lane due to the presence of a shoulder, a comparison of two binomial proportions was conducted. The research hypothesis was that the proportion of volume in the right lane is not equal for both populations. Applying the test to the data collected for peak hour volumes ≤ 400 yielded a p-value of approximately zero. Applying the same test to the data collected for peak hour volumes > 400 yielded similar results. Therefore, there was nearly a 100 percent confidence that the proportion of vehicles in the right lane was not equal for sites with shoulders and sites without shoulders.

The results indicated that for the range of data collected, a higher proportion of vehicles was utilizing the right lane in those sections with shoulders. This tendency could be due to drivers expecting to encounter slow moving vehicles and turning vehicles on the shoulder instead of the through travel lane. Drivers could also be perceiving the shoulder as a type of buffer zone from obstacles behind the curb and, thus, are more comfortable in the right lane than if there were no shoulder.

Table 10. Right Lane Volume Distribution

	Peak Volume ≤ 400		Peak Volume > 400	
	Sites with Shoulders	Sites without Shoulders	Sites with Shoulders	Sites without Shoulders
Total Volume (n)	2,087	2,076	3,463	7,394
Volume in Right Lane (y)	1,359	1,021	1,995	3,367
Proportion (π)	0.651	0.492	0.576	0.455

4.4 SHOULDER REQUIREMENTS

To develop criteria warranting the use of a shoulder on high-speed curb sections, a study involving conflict analysis was conducted. Conflict rates are a surrogate measure of accidents utilized due to the length of time required to establish a sufficient sample of accident data. In order to perform a conflict analysis on the collected data, both standard statistical tests and a regression analysis were performed.

4.4.1 Conflict Definitions

In order to establish a means by which conflicts could be observed and categorized in the field, it was necessary to develop an operational definition of a traffic conflict. As mentioned earlier, a traffic conflict is generally a traffic event involving the interaction of two vehicles, where one or both drivers may have to take an evasive action to avoid a collision. It is necessary to further define a traffic conflict so as to direct the definition towards operations in a high-speed, suburban curb section. For the purpose of this research, a traffic conflict has been defined as:

> A traffic event involving two or more road users, in which one user performs some atypical or unusual action, such as a change in direction or speed, that places another user in jeopardy of a collision unless an evasive maneuver is undertaken.

By establishing this conflict definition, it becomes necessary to also define an evasive maneuver and to determine what constitutes an atypical or unusual action. An evasive maneuver is defined as the visible braking of a vehicle to avoid a collision with another vehicle. Although swerving, lane changing, and slowing down might be construed as evasive maneuvers, each of them introduces a measure of subjectivity into the observation. Repeatability would therefore be difficult to obtain from one observer to the next. The observation of visible braking is thus required to determine whether or not a conflict has occurred.

This visible braking must result from some atypical or unusual action of another vehicle. These actions form the basis for the conflict categories used during this research. The following conflict categories were observed and recorded:

1. Right Turn, Exiting - A vehicle slows in a through travel lane to make a right turn, causing a following vehicle to perform an evasive maneuver.

2. Right Turn, Entering - A vehicle approaching from the right enters a through travel lane with a right turn, causing a following vehicle to perform an evasive maneuver.

3. Slow Vehicle - A conflict which occurs due only to the fact that the following vehicle is traveling at a higher rate of speed than the leading vehicle, and therefore runs up on the lead vehicle.

4. Secondary Conflict - In an attempt to avoid a collision with a leading vehicle, a following vehicle makes an evasive maneuver, causing a second following vehicle to also make an evasive maneuver.

5. Left Turn, from Left - A vehicle approaching from the left enters a through travel lane with a left turn, causing a following vehicle to perform an evasive maneuver.

6. Left Turn, Opposing - A vehicle approaching from the opposite direction performs a left turn, causing a vehicle to perform an evasive maneuver.

7. Left Turn, from Right - A vehicle approaching from the right crosses the through travel lanes with a left turn, causing a vehicle to perform an evasive maneuver.

8. Left Turn, Same Direction - A vehicle slows in a through travel lane to perform a left turn, causing a following vehicle to perform an evasive maneuver.

9. Other Conflicts - Any other observed conflict not covered by the previous definitions.

4.4.2 Mean Conflict Rates

The study sites included seven locations with a paved shoulder and nine locations without a paved shoulder. During each three hour observation period total conflicts were observed. These total conflicts were divided by the total volumes to determine a conflict rate per 1000 vehicles. From this information, a mean conflict rate was established for those locations without a paved shoulder and for those with a paved shoulder. This data has been tabulated in Tables 11 and 12, respectively.

Table 11. Conflict Rates for Sites with No Paved Shoulder

Location	Peak Hour Volume	Conflicts	Conflict Rate per 1000 vehicles
Comal A.M.	356	2	5.62
Comal P.M.	676	5	7.40
Gregg A.M.	788	40	50.76
Gregg P.M.	949	59	62.17
Henderson A.M.	897	30	33.44
Henderson P.M.	480	12	25.00
Lamb (84) A.M.	184	1	5.43
Lamb (84) P.M.	211	2	9.48
Nueces A.M.	338	8	23.67
Nueces P.M.	810	56	69.14
Rusk A.M.	315	13	41.27
Rusk P.M.	587	31	52.81
San Patricio A.M.	388	3	7.73
San Patricio P.M.	447	8	17.90
Smith (155) A.M.	402	9	22.39
Smith (155) P.M.	873	30	34.36
Tom Green A.M.	284	4	14.08
Tom Green P.M.	485	8	16.49

Observation Periods (n_1)	18
Mean Conflict Rate (y_1)	27.73
Standard Deviation (s_1)	20.17
Variance (s_1^2)	406.73

Table 12. Conflict Rates for Sites with Paved Shoulder

Location	Peak Hour Volume	Conflicts	Conflict Rate per 1000 vehicles
Bastrop A.M.	716	9	12.57
Bastrop P.M.	717	13	18.13
Calhoun A.M.	258	4	15.50
Calhoun P.M.	957	50	52.25
Lamb (430) A.M.	86	0	0.00
Lamb (430) P.M.	70	1	14.29
Mills A.M.	187	6	32.09
Mills P.M.	247	10	40.49
Smith (64) A.M.	648	11	16.98
Smith (64) P.M.	375	6	16.00
Victoria A.M.	425	7	16.47
Victoria P.M.	366	6	16.39
Williamson A.M.	205	3	14.63
Williamson P.M.	293	4	13.65

Observation Periods (n_2)	14
Mean Conflict Rate (y_2)	19.96
Standard Deviation (s_2)	13.12
Variance (s_2^2)	172.11

The conflicts were further broken down by conflict category and a conflict rate determined for each particular category. For both types of cross section, the majority of observed conflicts were associated with right turn movements, slow vehicles, and secondary conflicts. Left turn conflicts accounted for a small portion of the total conflict rate in both cases. For those sites without a paved shoulder, only 2.59 of the 27.73 conflicts per 1000 vehicles can be attributed to left turn operations. In those sites with a paved shoulder, left turn operations account for only 3.42 of the 19.96 conflicts per 1000. A majority of the secondary conflicts which occurred were associated with right turn exiting vehicles.

Based on the observations in the field, vehicles tended to utilize the paved shoulder as a deceleration lane when exiting the roadway. Drivers tended to utilize the shoulder as an acceleration lane only when there were vehicles approaching. There was a reduction in observed conflicts when a paved shoulder was present, which may be confirmed through the performance of standard statistical tests.

A pooled t-test was applied to the data to determine whether or not there is a higher conflict rate in those sites without a paved shoulder. The research hypothesis was that the mean conflict rates are higher in those sites without a paved shoulder. The t-test produced a p-value of 0.05. The p-value translates to a 95 percent confidence that the conflict rate is higher in those locations without a paved shoulder.

4.4.3 Regression Analysis

The collected data indicates an apparent correlation between traffic volume and conflict rate. One method of predicting conflict rates for a given traffic volume is the method of least squares. This method chooses the prediction line $y = \beta_0 + \beta_1 x$ that minimizes the sum of the squared errors of prediction. By performing a linear regression analysis on the observed data, using the traffic volume as the independent variable and the observed conflict rates as the dependent variable, it is possible to apply the prediction equation to traffic volumes to predict conflict rates.

In this study, the regression analysis feature of the Quattro Pro spreadsheet software program was used. The regression analysis worksheets have been included in Appendix C and the results summarized in Table 13.

The results of the analysis indicate a positive linear relationship between traffic volume and conflict rate for both data sets. This information can be entered into the prediction equation described earlier and a line drawn through the scatter plot of observed data. Several outliers fall within the data set. For those sites with shoulders, three locations vary considerably from the expected value. The two sites in Mills County demonstrated conflict rates much higher than might have been expected for their peak hour volume, which could be attributed to the divergence of US Highway 84 and US Highway 183 approximately one-half mile downstream from the study site. Vehicles could have been making lane choice decisions in the study site in anticipation of this downstream divergence. The data collected in Calhoun County during the p.m. peak period demonstrated by far the highest conflict rate for all of the sites with shoulders. There was a threefold

Table 13. Results of Regression Analysis for Conflict Study

	Study Sites with Shoulders	Study Sites without Shoulders
Observation Periods (n)	14	18
y-intercept (β_0)	11.6777	-1.7949
Slope of Prediction Line (β_1)	0.0209	0.0561
Standard Error (S_ϵ)	12.3683	15.0781
Correlation Coefficient (r)	0.4237	0.6884

increase in traffic volume from the a.m. peak to the p.m. peak, and a large number of these vehicles exited the roadway to stop at a combination gas station and food store. The number of vehicles entering and exiting the roadway at this location caused a large number of right turn and secondary conflicts.

For those sites without shoulders, three sites produced results which deviated from the expected values. The p.m. peak at the Nueces County location produced the highest rate of conflicts for any of these sites. A large apartment complex was located on the right side of the roadway, and most of the observed conflicts were right turn exiting and secondary conflicts. Tenants returning to the apartment complex at the end of the work day appeared to be the cause of this high conflict rate. The Comal County p.m. peak produced a lower than expected conflict rate. There was an extremely low level of access at this location, which is evidenced in the fact that no conflicts were attributed to entering or exiting vehicles. Conversely, the Rusk County location produced higher than expected conflict rates. A large retail center, including a Wal-Mart, was located on the left side of the roadway and generated a high volume of entering and exiting traffic.

The correlation coefficients (r-values) for the data sets provide some insight into the relationship between traffic volume and conflict rates. R-values between 0 and 1 indicate a positive linear relationship. While both data sets show a positive linear relationship between traffic volume and conflict rate, the relationship appears to be much stronger for those locations without a paved shoulder. In fact, the slope of the prediction equation for those locations having a paved shoulder is flat enough to warrant a test to determine whether volumes are a useful predictor of conflict rates.

The research hypothesis was that traffic volume is a useful predictor of conflict rates. The results from the test yielded a p-value that was not significant at the 95 percent confidence level. Therefore, it was concluded that for those sites with a paved shoulder, conflict rates are relatively insensitive to increases in traffic volume.

Conducting the same test on the data for those locations without shoulders produced a p-value of 0.002. The p-value translates to a 99.8 percent confidence that traffic volume is a useful predictor of conflict rate. Therefore, it was concluded that conflict rates increase linearly with increases in traffic volume.

Once it has been established that traffic volume is a useful predictor of conflict rates for those locations without shoulders, an additional test may be run to determine whether the y-intercept of the line is equal to zero. In other words, does zero traffic volume translate to a conflict rate of zero? The research hypothesis was that the y-intercept of the prediction line is not equal to zero. The results from the test yielded a p-value that was not significant at the 95 percent confidence level. Therefore, it was concluded that zero conflicts may be expected as traffic volume approaches zero.

Assuming that the y-intercept of the prediction line is zero, it was necessary to recompute the slope, β_1. The results of this regression analysis are included in Appendix B. This computation resulted in a new prediction line for those locations without shoulders represented by the equation $y = 0 + 0.0533\,(x)$, where y = conflict rate and x = peak hour volume.

For the range of data collected for this project, the prediction equation for locations with no shoulders suggests that conflict rates approach zero at peak hour volumes near zero, then increase linearly with increases in volume. In those locations with shoulders, conflict rates were much less sensitive to increases in traffic volume, so it appears that the inclusion of a paved shoulder limits the rate of increase in conflict rate.

4.4.4 Shoulder Guidelines

The point at which both types of cross section exhibited approximately equal conflict rates was at a peak hour volume of approximately 350 (See Figure 8). Above this volume, conflict rates continued to increase linearly if no shoulder was provided. Below this volume, the presence of a paved shoulder demonstrated little effectiveness in reducing conflict rates. A peak hour volume range of plus or minus 50 would provide some flexibility for whether or not to include shoulders, depending on other circumstances, such as intensity of access and severity of right-of-way restriction.

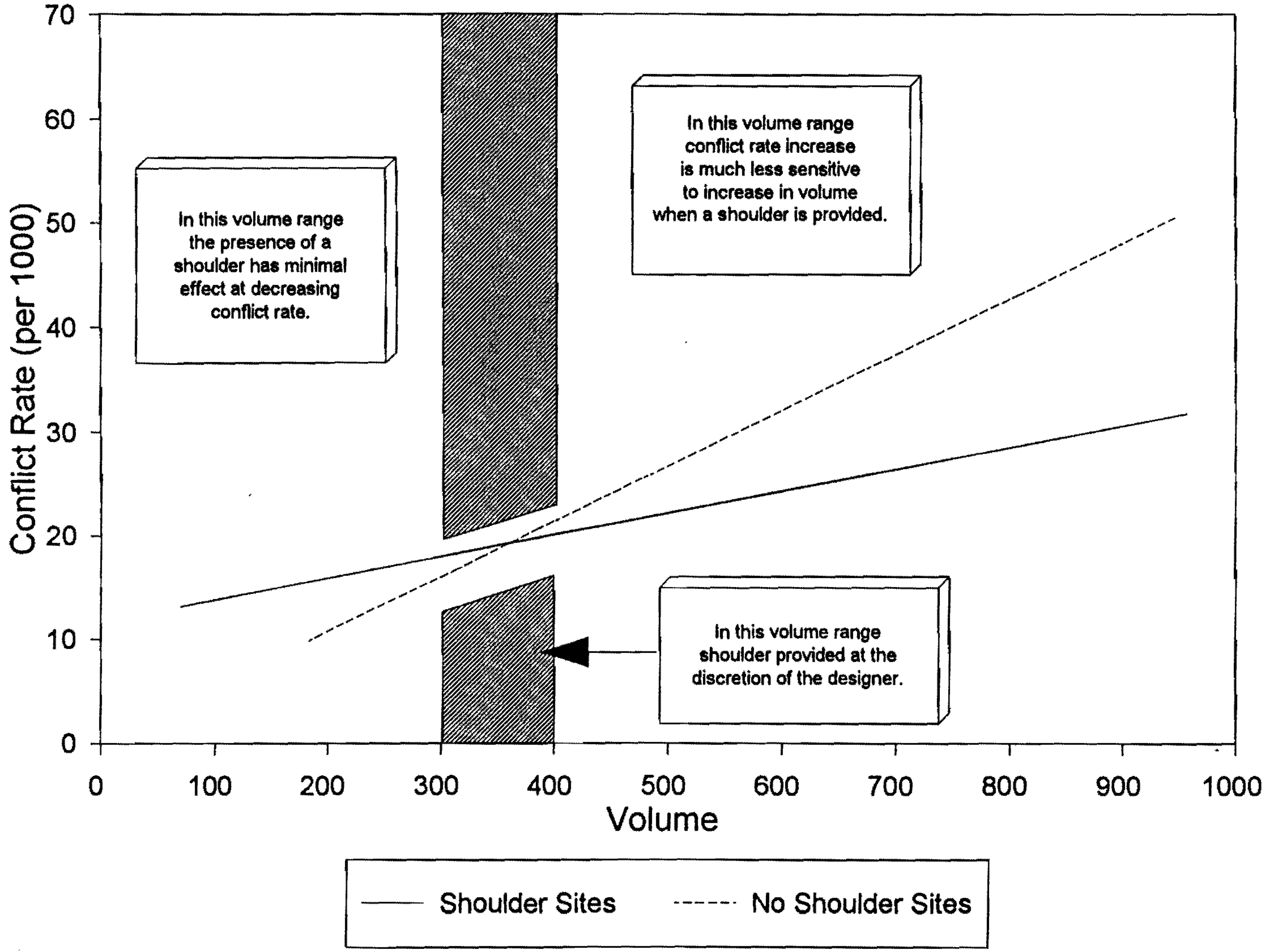

Figure 8. Providing Shoulders in High-Speed Curb Sections

4.5 TWO-WAY LEFT-TURN LANE REQUIREMENTS

Most suburban roadways have a relatively high density of driveway access locations and side road intersections with significant left-turn maneuvers; therefore, the potential for accidents involving through and left-turning vehicles may be high. TWLTLs can reduce this accident potential by removing left-turning vehicles from the through lane. To develop criteria warranting the use of a TWLTL on high-speed curb sections, a study involving a conflict analysis was conducted.

From the 16 sites evaluated in the previous traffic conflict analysis (see Table 7), 13 were selected for this study - 3 sites were not used for this study because they contained divided highway sections. All roadway sites had a minimum posted speed limit of 80.5 km/h (50 mph) and included two through travel lanes in each direction. The lengths varied from approximately 150 to 300 m (500 to 1000 ft). All sites were through travel sections with no traffic control devices in the direction of travel being observed. Table 14 lists the 13 study sites selected for this study and their geometric attributes.

The study design was divided up into two parts: (1) a field study, and (2) a theoretical analysis. The first part involved determining the effects of traffic volume on conflict rates for sites with and without TWLTLs. The second part involved developing a probabilistic model to predict left-turn, same direction conflicts. The goal of this study was to define the limits in which a TWLTL would provide significant benefits (reduced accident potential) for a high-speed suburban roadway with no existing median.

The only conflicts examined for this study were left-turn, same direction. As described earlier, a left-turn, same direction conflict occurs when a vehicle slows in a through travel lane to perform a left turn, causing a following vehicle to perform an evasive maneuver. The left-turn opposing conflicts (a vehicle approaching from the opposite direction performs a left turn, causing another vehicle to make an evasive maneuver) were not considered because these conflicts would occur whether a TWLTL was present or not.

4.5.1 Field Study

The first study was conducted to determine the volume range in which TWLTLs would be justified. This study involved a conflict analysis similar to that used for developing paved shoulder guidelines (see Section 4.4.3). However, the only conflict rates analyzed for this study were those that involved left-turn, same direction conflicts. The study sites were selected from those used in the analysis concerning paved shoulders. Nine of the selected sites had TWLTLs, and four of the sites had no median (undivided). The conflict rates were calculated by determining the total volume and total left-turn same direction conflicts for each three hour study period (i.e., morning peak and afternoon peak). From this, a mean conflict rate was established for those locations without a TWLTL and those with a TWLTL. These data are tabulated in Tables 15 and 16, respectively.

A pooled t-test was applied to the data to determine whether or not there was a higher conflict rate for those sites without a TWLTL. The research hypothesis was that the mean conflict rates are higher for those sites without a TWLTL. The results from the test yielded a p-value that was not

Table 14. Study Sites for TWLTL Study

County	Highway Number	Cross-Section	ADT	Posted Speed Limit km/h (mph)
Bastrop	SH 21	4-lane, Undivided	21,000	80 (50)
Comal	SH 46	4-lane, TWLTL	8,200	89 (55)
Henderson	SH 31	4-lane, TWLTL	13,000	89 (55)
Lamb	US 84	4-lane, TWLTL	4,700	80 (50)
Lamb	Loop 430	4-lane, Undivided	1,750	80 (50)
Mills	US 84	4-lane, Undivided	3,500	89 (55)
Nueces	FM 2444	4-lane, TWLTL	9,200	80 (50)
San Patricio	SH 35	4-lane, TWLTL	10,900	89 (55)
Smith	SH 64	4-lane, TWLTL	8,900	89 (55)
Smith	SH 155	4-lane, TWLTL	11,200	89 (55)
Tom Green	RM 584	4-lane, TWLTL	5,900	89 (55)
Victoria	SH 185	4-lane, TWLTL	10,200	89 (55)
Williamson	US 79	4-lane, Undivided	5,500	80 (50)

Table 15. Conflict Rates for Sites without TWLTLs

Location	Peak Hour Volume	Left-Turn, Same Direction Conflicts	Conflict Rate per 1000 Vehicles
Bastrop A.M.	716	2	0.98
Bastrop P.M.	717	3	1.97
Lamb (430) A.M.	86	0	0.00
Lamb (430) P.M.	70	0	0.00
Mills A.M.	187	0	0.00
Mills P.M.	247	1	2.11
Williamson A.M.	205	1	1.20
Williamson P.M.	293	1	1.52
Observation Periods (n_1)	8		
Mean Conflict Rate (y_1)	0.97		
Standard Deviation (s_1)	0.88		
Variance (s_1^2)	0.78		

Table 16. Conflict Rates for Sites with TWLTLs

Location	Peak Hour Volume	Left-Turn, Same Direction Conflicts	Conflict Rate per 1000 Vehicles
Comal A.M.	356	0	0.00
Comal P.M.	676	0	0.00
Henderson A.M.	897	2	0.86
Henderson P.M.	480	1	0.98
Lamb (84) A.M.	184	0	0.00
Lamb (84) P.M.	211	1	1.73
Nueces A.M.	338	0	0.00
Nueces P.M.	810	0	0.00
Tom Green A.M.	284	0	0.00
Tom Green P.M.	485	0	0.00
San Patricio A.M.	388	1	1.13
San Patricio P.M.	447	2	2.16
Smith (64) A.M.	648	3	1.72
Smith (64) P.M.	375	0	0.00
Smith (155) A.M.	402	1	1.01
Smith (155) P.M.	873	3	1.43
Victoria A.M.	425	1	0.84
Victoria P.M.	366	1	1.22

Observation Periods (n_2)	18
Mean Conflict Rate (y_2)	0.73
Standard Deviation (s_2)	0.74
Variance (s_2^2)	0.55

significant at the 95 percent confidence level. Therefore, it was concluded that there is no significant difference between the mean conflict rates of those selected study sites with TWLTLs and sites without TWLTLs. This may be due to the limited study sites that had no TWLTLs. Many of these sites had very low volume levels which may have resulted in the low conflict rates.

A regression analysis was conducted on the collected data to determine if any correlation existed between traffic volume and conflict rate. The Quattro Pro spreadsheet program yielded the results shown in Table 17.

From the results of the analysis, there is a moderate indication of a positive linear relationship between traffic volume and conflict rate for those sites without TWLTLs, but little indication for those sites with TWLTLs. The r-values for both data sets are between 0 and 1, indicating a positive relationship; however, the relationship appears to be very weak for those sites with TWLTLs (i.e., the r-value is close to 0). A relatively stronger relationship exists for those sites without TWLTLs.

To determine if a relationship existed between traffic volume and conflict rate for sites with TWLTLs, a t-test was conducted. The research hypothesis was that traffic volume is a useful predictor of conflict rates. Applying this test to the data for those locations with TWLTLs produced a p-value that was not significant at the 95 percent confidence level. Therefore, it was concluded that for those sites with a TWLTL, conflict rates are relatively insensitive to increases in traffic volume.

Conducting the same test on the data for those locations without TWLTLs produced a p-value of 0.02. This translates to a 98 percent confidence that there is a positive linear relationship between traffic volume and conflict rate for those sites without TWLTLs.

Table 17. Results of Regression Analysis for TWLTL Study

	Study Sites Without TWLTLs	Study Sites With TWLTLs
Observation Periods (n)	8	18
y-intercept (β_0)	0.3839	0.5413
Slope of Prediction Line (β_1)	0.0019	0.0004
Standard Error (S_ε)	0.0012	0.0009
Correlation Coefficient (r)	0.5462	0.1115

An additional test was run to determine if the y-intercept of the prediction line was zero for those sites without TWLTLs. The research hypothesis was that the y-intercept is not equal to zero. The results from the test yielded a p-value that was not significant at the 95 percent confidence level. Therefore, it was concluded that the y-intercept of the prediction line is zero.

Assuming that the y-intercept of the prediction line is zero, it was necessary to recompute the slope, β_1. This resulted in the following prediction equation for the study sites without TWLTLs: $y = 0 + 0.0026\ (x)$, where y = conflict rate and x = peak hour volume. The prediction line for sites without TWLTLs was plotted along with the results from the analysis for those sites with TWLTLs (see Figure 9).

By observing Figure 9, it is seen that both types of cross section yield approximately equal conflict rates at a peak hour volume of approximately 250. Above this volume, the conflict rate continues to increase linearly for sites without a TWLTL. Below this volume, the use of a TWLTL demonstrates little effectiveness in reducing conflict rates. A peak hour volume of plus or minus 50 provides some flexibility in deciding where TWLTLs are most effective.

4.5.2 Theoretical Analysis

To determine when a TWLTL would provide significant benefits over no median, another study was conducted in which a probabilistic model was developed to predict left-turn, same direction conflicts. For a left-turn conflict to occur between two vehicles traveling in the same direction, it was assumed the following two events must occur: the headway between the vehicles must be such that if the leading vehicle slows to turn left, the following vehicle will have to make an evasive maneuver to avoid an accident; and the leading vehicle will make a left turn. Assumptions were used to develop a mathematical model to predict the probability of a left-turn, same direction conflict based on through volume and driveway density. Following is a more in-depth description of these assumptions and a discussion of the methodology used to develop this model.

Predicting Time Headway

The first step in developing the model was to predict the probability that the time headway between two successive vehicles was less than some value. This value was the maximum time headway required for a leading, left-turning vehicle to be in conflict with a following, through vehicle traveling in the same direction. If the actual time headway was greater than this value, then the following, through vehicle would not have to make an evasive maneuver to avoid a collision with the leading, left-turning vehicle.

To estimate the maximum time headway, calculations were made assuming a conservative, worse case scenario. The overall objective of this study was to reduce traffic accidents involving left turns from suburban highways, and it was felt that using this conservative approach would result in a minimization of left-turn conflicts. The calculations for the maximum time headway are shown in Appendix D.

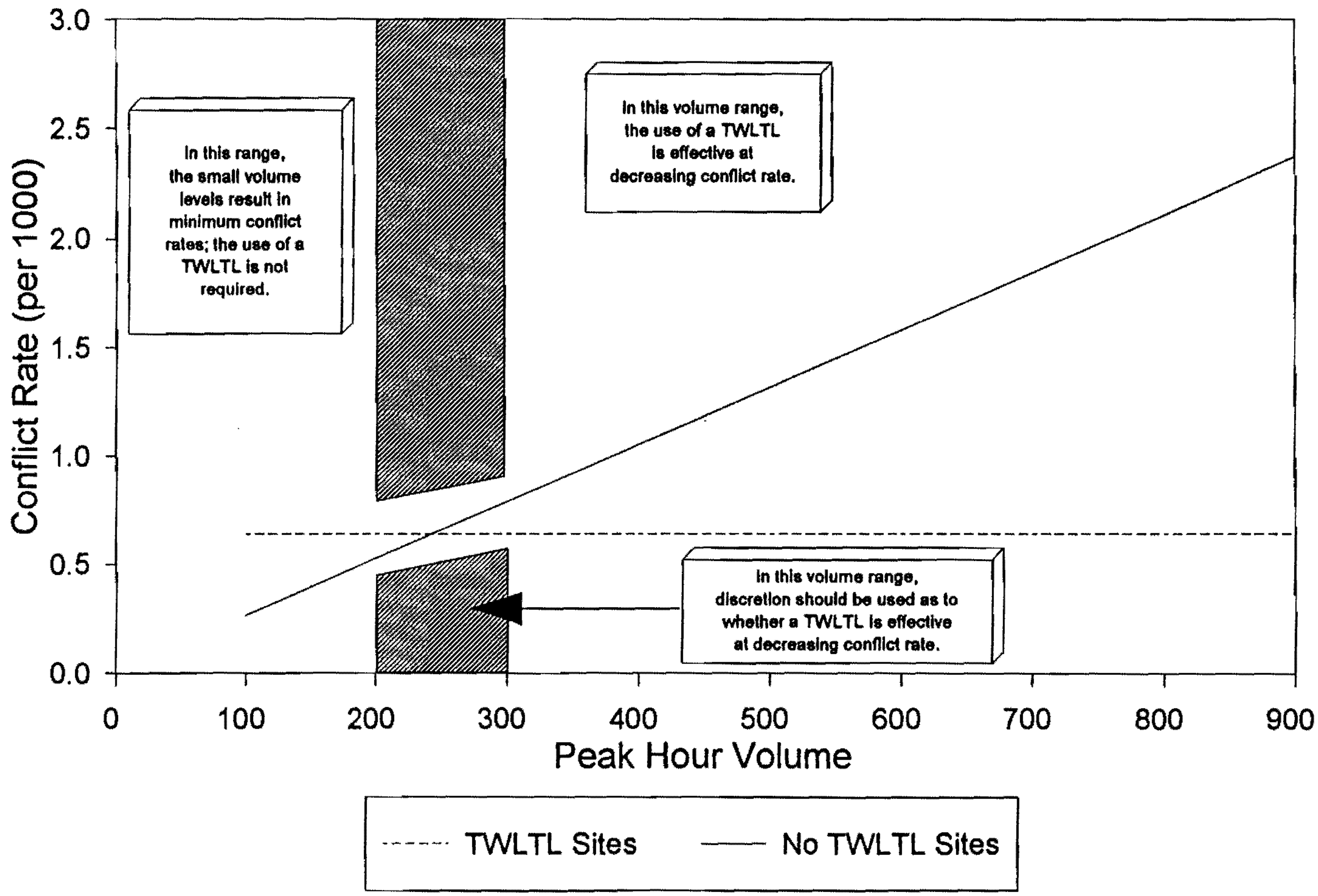

Figure 9. Effects of Traffic Volume on Left-Turn, Same Direction Conflict Rate

It was assumed that the maximum time headway was composed of two time periods. The first time period was the minimum time required for the leading, left-turning vehicle to slow from its operating speed to zero (assuming vehicle stops for oncoming traffic) to make a left turn. The operating speed was assumed to be between 80.5 and 88.5 km/h (50 and 55 mph). The time to decelerate from operating speed to zero was calculated using the deceleration rates recommended by AASHTO (1). The second time period was the minimum time required for the following, through vehicle to detect the leading, left-turning vehicle and make an evasive maneuver. This time was calculated using the stopping sight distance requirements by AASHTO for operating speeds between 80.5 and 88.5 km/h (50 and 55 mph). Combining these two time periods resulted in an estimated maximum time headway of 17.0 seconds. In other words, if two vehicles were traveling in a time headway of 17.0 seconds or less, and the leading vehicle stopped to make a left turn, then the two vehicles would be in conflict.

To predict the probability of vehicles arriving in a given time headway, it was assumed that the arrival headway between successive vehicles could be estimated using a theoretical Poisson distribution. This distribution assumes that vehicles are arriving randomly. This assumption is valid because traffic volumes in suburban areas are typically low to moderate, and traffic signals are not usually employed. Assuming a Poisson distribution, the probability of two vehicles arriving in a headway of 17.0 seconds or less was estimated over a given volume range. The results are shown in Appendix D.

Predicting Left Turns

To predict the probability of left-turning vehicles, data collected from the study sites were used. Videotapes and photographs for each site (taken earlier) were viewed to obtain the necessary data. The sites were videotaped during the morning peak (7:00 A.M. to 10:00 P.M.) and the evening peak (3:00 P.M. to 6:00 P.M.). These videotapes were used to obtain the total through volume and left-turning volume in one direction. Driveway densities were estimated using the videotapes and photographs. The results are shown in Table 18. This information was used to develop a mathematical model to predict the percent (or probability) of left-turning vehicles based on through volume and driveway density.

A regression analysis was undertaken in an attempt to develop a linear relationship between the observed variables. Again, the method of least squares, which chooses the prediction line $y = \beta_0 + \beta_1 x_1 + \beta_2 x_2$ that minimizes the sum of the squared errors of prediction, was used. By performing a linear regression analysis on the observed data, using the percent left-turning vehicles as the dependent variable and the through volume and driveway density as independent variables, it was possible to predict the percent left-turning vehicles from observed traffic volumes and driveway densities.

The regression analysis was performed using the Quattro Pro spreadsheet program (see Table 19). The results of the analysis indicated a positive linear relationship between the percent left-turning vehicles and through volume/driveway density. This was supported by calculating the correlation coefficient (r). As stated earlier, r provides some insight into the relationship between the observed variables; an r-value between 0 and 1 indicates that there is a positive linear

Table 18. Attributes of Study Sites for TWLTL Study

Location	Length of Site m (ft)	Number of Drives	Drive Dens. Drv/Km (Drv/Mi)	Total Volume	Number of Left-Turns	Percent Left-Turn Vehicles
Gregg Loop 281	274.3 (900)	7	25 (41)	A.M. 1858 P.M. 2406	176 256	9.47 10.64
Henderson SH 31	289.6 (950)	4	14 (22)	A.M. 2325 P.M. 1020	126 63	5.42 6.18
Lamb US 84	243.8 (800)	2	8 (13)	A.M. 429 P.M. 578	31 21	4.90 2.60
Mills US 84	213.4 (700)	3	14 (23)	A.M. 672 P.M. 475	40 18	5.95 3.79
San Patricio SH 35	225.6 (740)	5	22 (36)	A.M. 887 P.M. 927	76 29	8.57 9.39
Smith SH 64	237.7 (780)	2	9 (14)	A.M. 1748 P.M. 741	79 33	4.52 4.45
Victoria SH 185	161.5 (530)	2	6 (9)	A.M. 1185 P.M. 820	40 24	3.38 2.93

Table 19. Results of Regression Analysis to Predict Left Turns

Number of Observations (n)	14
y-Intercept (β_0)	0.737
Slope of Prediction Line	
Traffic Volume (β_1), PHV	0.001
Drive Density (β_2), drv/km (drv/mi)	0.324 (0.202)
Correlation Coefficient (r)	0.952
Standard Error	
Traffic Volume ($S_{\hat{\epsilon}1}$), PHV	0.001
Drive Density ($S_{\hat{\epsilon}2}$), drv/km (drv/mi)	0.039 (0.024)

relationship. From the data sets, an r-value of 0.952 was computed, indicating that there is strong evidence of a positive linear relationship between percent left-turning vehicles and traffic volume/driveway density.

The coefficient of determination (r^2) for a regression model is the proportion of variability in the dependent variable accounted for by the independent variable. This value can help determine how much influence each independent variable (traffic volume and driveway density) has on the dependent variable (percent left-turns). When driveway density was used as the only independent variable, r^2 was calculated as 0.895. When driveway density and traffic volume were both considered as independent variables, r^2 was calculated as 0.905. A significant jump between these two values would indicate that both independent variables have a strong influence on the dependent variable; however, the difference between the r^2-values was very small, indicating that traffic volume does not strongly influence the percent of left-turning vehicles. Therefore, it was concluded that the percent of left-turning vehicles is strongly influenced by driveway density, but it is relatively insensitive to traffic volume.

The computations of the regression analysis are included in Appendix D. The results yielded the following equation (Note: English units are in parentheses): $y = 0.001(x_1) + 0.325(x_2) + 0.737$ ($y = 0.001(x_1) + 0.202(x_2) + 0.737$), where y = percent left-turning volume, x_1 = peak hour volume, and x_2 = driveway density, drives/km (drives/mi). For the range of data collected for this study, this equation can be used to estimate the amount of left-turning volume; however, the data observed were limited. Applying these results to all suburban highways can help predict left turning volume based on through traffic volume and driveway density.

Predicting Left-Turn, Same Direction Conflicts

As stated previously, the probability of a left-turn, same direction conflict depends on the probability that two successive vehicles are in a given headway (17.0 seconds) and the probability that the leading vehicle will turn left. Mathematical models have been selected and developed to predict the latter two probabilities; therefore, knowing these probabilities, the likelihood of a left-turn, same direction conflict can be estimated. By assuming the event of two vehicles arriving in a certain headway and the event of the leading vehicle turning left are independent (i.e., the probability of one event does not depend on the occurrence of the other event), the probability of each event can be multiplied to predict the probability of a left-turn, same direction conflict. This process was carried out to predict the likelihood of a left-turn, same direction conflict given peak hour volume and driveway density. The computations are included in Appendix D, and the results are illustrated in Figure 10.

Observing Figure 10, it is noted that the probability of a left-turn, same direction conflict increases with increasing traffic volume and increasing driveway density; however, there are volume ranges where the rates of increase (slopes) of the prediction lines are greater than others. Finding the critical points (where the slope of a prediction line changes dramatically) helped to determine when TWLTLs should be provided.

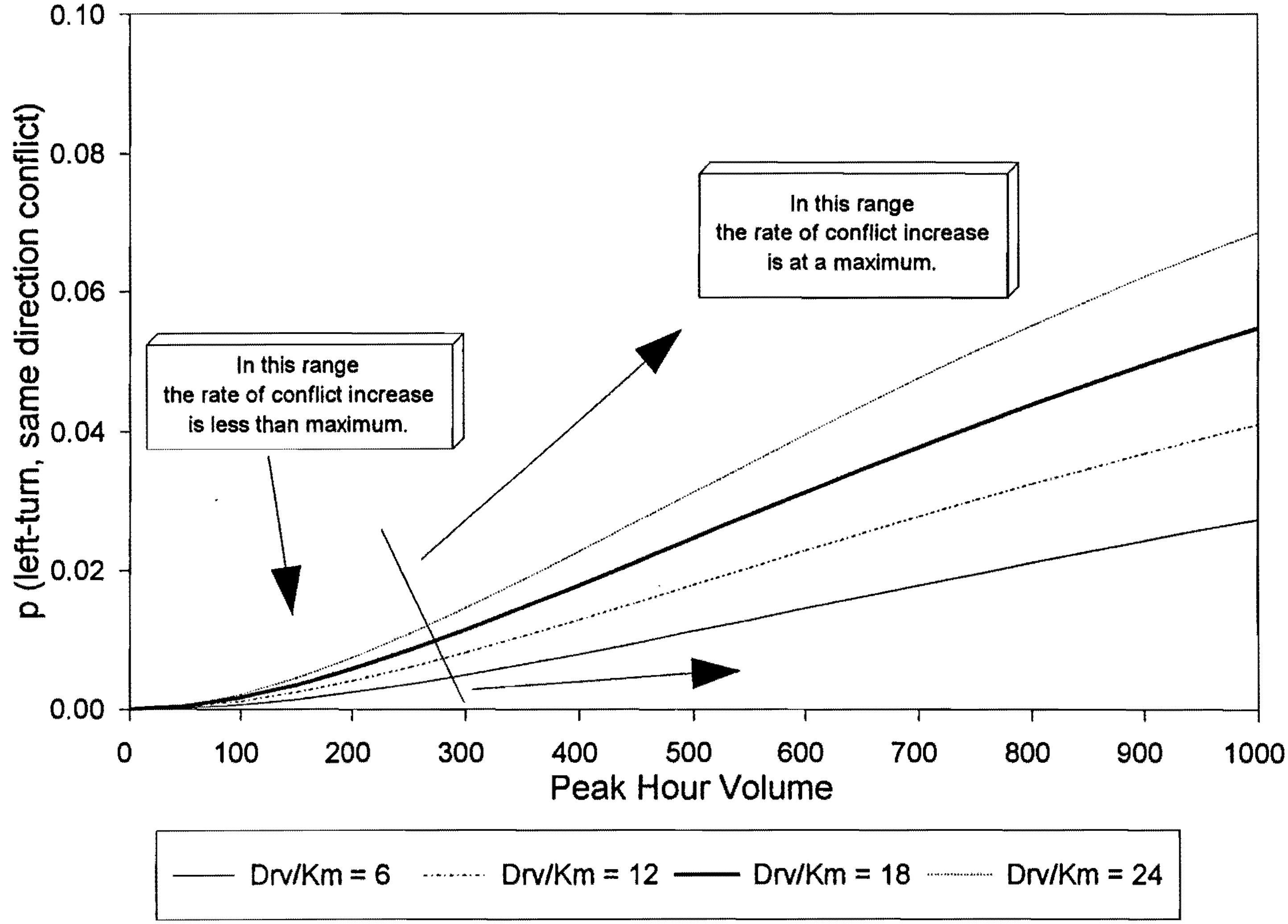

Figure 10. Probability of Left-Turn, Same Direction Conflict

As shown in Figure 10, for given driveway densities, the probability of a left-turn, same direction conflict increases slowly for low volumes (0 < PHV < 150), and then begins to increase dramatically (150 < PHV < 400). Then at some point, for each driveway density, the slopes of the corresponding prediction lines reach a maximum and become relatively constant. The points where the slopes reached maximum were used to determine the minimum volumes at which a TWLTL should be provided for given driveway densities. These criterion helped to develop TWLTL guidelines for given driveway densities and traffic volumes.

4.5.3 TWLTL Guidelines

The results from the theoretical analysis (see Figure 10) helped to determine the ranges of volume and driveway density where TWLTLs were most effective at reducing traffic conflicts. From Figure 10, the critical volumes (where the rate of conflict increase (slope) reached a maximum for each corresponding driveway density) were estimated to develop guidelines (see Table 20). These guidelines present a range of driveway densities and corresponding minimum volume levels in which TWLTLs were most beneficial. For Table 20, peak hour volumes were converted to ADT using a K-factor of 10 to 12 percent.

The results in Table 20 show that as driveway density increases, the minimum ADT decreases. This is reasonable because as the driveway density increases, the demand for left turns will increase. This increase in left-turn demand will result in a higher potential for left-turn, same direction conflicts. As traffic volumes become higher, the left-turn conflict potential will continue to increase. Therefore, as driveway densities become higher, TWLTLs are justified at lower volume levels.

Table 20. Providing TWLTLs in High-Speed Curb Sections

Driveway Density drv/km (drv/mi)	Minimum ADT
6 (10)	3,000
12 (20)	2,900
18 (30)	2,800
24 (40)	2,700

The guidelines from Table 20 suggest that TWLTLs should be provided on high-speed curb and gutter roadways with ADTs above 2,700 to 3,000, depending on driveway density. These results are very similar to those obtained from the field study (see Figure 9). From Figure 9, it is seen that TWLTLs were effective at decreasing conflict rate above a peak hour volume of approximately 300. Converting the peak hour volume to ADT using a K-factor of 10 to 12 percent results in a range in ADT of approximately 2,500 to 3,000. This volume range is very close to that obtained from the theoretical analysis, increasing the credibility of the results from the theoretical analysis.

It is the research team's belief that TWLTL facilities on high-speed suburban roadways should be warranted at lower levels of traffic volumes and driveway densities than those for urban roadways. Current TxDOT guidelines state that TWLTLs are warranted on urban facilities with 3,000 or more vehicles per day and 12 or more drives per kilometer (20 drives per mile). Therefore, TWLTLs on suburban roadways should be warranted at a traffic volume in the range of 3,000 vehicles per day or less with a corresponding driveway density in the range of 12 drives per kilometer (20 drives per mile) or less.

As shown in Table 20, the resulting TWLTL guidelines for suburban roadways fit the above criteria. For example, a suburban roadway with a driveway density of 12 drives per kilometer (20 drives per mile) should include a TWLTL at a minimum ADT of 2,900 (compared to 3,000 for urban roadways). This relationship further increases the credibility of the final TWLTL guidelines for high-speed curb and gutter roadways.

The field data used for this study was limited; therefore, as with most design guidelines, designers should apply engineering judgement in using these recommendations. As stated above, however, the developed guidelines are supported and seem reasonable. Therefore, it was concluded that applying the results in Table 20 to all high-speed curb and gutter roadways can help determine when a TWLTL should be provided.

5.0 CLEAR ZONE STUDY

The objective for this portion of the study was to determine an appropriate and cost-beneficial clear zone requirement for the upgrading of suburban high-speed arterial highways with curb and gutter cross sections. Under current design guidelines, low-speed (i.e., 72.4 km/h (45 mph) or less) urban roadways with curb and gutter cross sections and no shoulders are required to have a minimum of 0.46 m (18 in) clear zone beyond the face of the curb. On the other hand, high-speed rural highways with shoulders and parallel drainage ditches are required to have a 9.1 m (30 ft) clear zone width beyond the edge of the travelway, i.e., edgeline or edge of pavement. There is a big difference between the clear zone requirements for urban and rural highways and there need to be some intermediate design requirements of clear zone width for suburban high-speed curb and gutter roadways which serve as a transition zone between urban and rural highways.

These suburban arterial highway sections are typically not newly-constructed, but reconstructed from existing two-lane rural highways. As the growth of an urban area extends outward along a major arterial highway, the nature of the land use along the highway changes from rural and agricultural use to strip commercial developments, such as service stations, fast food restaurants, and strip shopping centers. The resulting growth in traffic volume and frequent turning movements necessitates the widening of the two-lane highway to a four-lane highway, oftentimes with two-way left-turn center lane. The addition of lanes to the highway reduces the available clear zone width unless additional right-of-way is purchased. In other words, the typical clear zone width of 9.1 m (30 ft) common to rural highways may not be available after the highway is widened to provide more travel lanes. The question now becomes what clear zone width is required for such suburban high-speed highways for the improvement to be cost-beneficial.

This chapter first presents a brief overview of the benefit/cost analysis procedure used for the analysis of the clear zone width requirement, followed by a discussion on the study approach. The study results are then presented with appropriate conclusions and recommendations.

5.1 BENEFIT/COST ANALYSIS

The analysis of the clear zone requirement for suburban arterials was based on the cost-effectiveness or benefit/cost approach. The basic concept behind the benefit/cost analysis is that public funds should be invested only in projects where the expected benefits would equal or exceed the expected direct costs of the project. Benefits are measured in terms of reductions in accident or societal costs associated with decreases in the frequency or severity of accidents. Direct highway agency costs are comprised of initial installation, maintenance, and accident repair costs. An incremental benefit/cost ratio between the additional benefits and costs associated with an improvement option over the existing conditions or another improvement option is normally used as the primary measure of whether or not a safety improvement investment is appropriate. The formulation of the incremental benefit/cost ratio is expressed below:

$$B/CRatio_{2-1} = \frac{B_2 - B_1}{C_2 - C_1} \tag{1}$$

where

BC_{2-1}	=	Incremental B/C ratio of alternative 2 compared with alternative 1
B_1,B_2	=	Annualized accident or societal cost of alternatives 1 and 2
C_1,C_2	=	Annualized direct cost of alternative 1 and 2

When comparing safety improvement alternative 2 to alternative 1, if the incremental benefit/cost ratio is greater than 1, it indicates that the benefits of alternative 2 are greater than the increased costs associated with that improvement.

The benefit/cost procedure used in this study is known as the ABC model, developed previously at the Texas Transportation Institute (TTI) for similar applications. The procedure is based on the encroachment probability model which is unique to roadside safety cost-effectiveness procedures. It is based on the concept that the run-off-road accident frequency can be directly related to the encroachment frequency, i.e., the number of vehicles inadvertently leaving the traveled portion of the roadway, which is a function of roadway and traffic characteristics, and that the severity of run-off-road accidents is related to encroachment characteristics, such as the speed and angle of encroachment.

The basic formulation of the encroachment model is expressed by the following equation:

$$E(C) = \sum_{i=1}^{n} P(E)*P(A|E)*P(I_i|A)*C(I_i) \tag{2}$$

where

E(C)	=	Expected accident cost
P(E)	=	Probability of an encroachment
P(A|E)	=	Probability of an accident given an encroachment
$P(I_i\|A)$	=	Probability of injury severity i, given an accident
$C(I_i)$	=	Cost associated with injury severity i
n	=	Number of injury severity levels

There are four key modules for the encroachment model-based analysis procedure:

1. Encroachment module,
2. Accident prediction module,
3. Accident severity module, and
4. Benefit/cost module.

5.1.1 Encroachment Module

The encroachment module utilizes roadway and traffic information to estimate the expected encroachment frequency, P(E), along any highway segment. A two-step process is used to estimate encroachment frequencies. The first step involves estimating a base encroachment frequency based on the highway type and traffic volume. The encroachment frequency-traffic volume relationships were established from encroachment data collected by Cooper during the late 1970's (26). This study involved weekly observations of wheel tracks on grass-covered roadsides of rural highways with speed limits in the 80.5 to 96.6 km/h (50 to 60 mph) range.

Base encroachment rates are then modified to account for specific highway characteristics, such as horizontal and vertical alignment. The rationale for these adjustment factors is that encroachments are affected by certain geometric and roadway cross-sectional characteristics, and the base encroachment rates should, therefore, be adjusted to account for these characteristics. For example, previous studies have found that vehicle encroachments are more likely on the outside of curves and the encroachment rate should thus be increased to account for the presence and the degree of curvature of the horizontal curve at such locations (27,28).

5.1.2 Accident Prediction Module

Given that an encroachment occurs, the accident prediction module estimates the conditional probability, P(A|E), that an accident will result. The determination of the probability that the encroaching vehicle will impact a roadside feature is based on factors such as lateral extent of vehicle encroachment, the encroachment speed and angle, vehicle trajectory, and hazard size.

The model first determines the probability that the vehicle would encroach far enough laterally to impact the roadside feature under consideration based on a distribution of the lateral extent of vehicle encroachment. This accounts for the probability that the vehicle may stop or steer back to the roadway before encroaching far enough to impact the roadside feature. The distribution of lateral extent of vehicle encroachment used in the model is derived from the Cooper study cited previously. As may be expected, the percentage of vehicles encroaching beyond a given lateral distance decreases as the distance from the edge of the travelway increases. In other words, a roadside feature located further away from the edge of the travelway is less likely to be impacted than one that is closer to the travelway.

If the vehicle is predicted to encroach far enough laterally, the model then estimates the probability that the vehicle will impact the roadside feature using an approach known as hazard imaging. An impact envelope, which is defined as the region along the roadway within which a vehicle leaving the travelway at a prescribed angle will impact the roadside object or feature, is shown in Figure 11. Given an encroachment by a vehicle of a particular type and size, the probability that the vehicle will leave the highway within the hazard envelope of a particular roadside obstacle is given by the following equation:

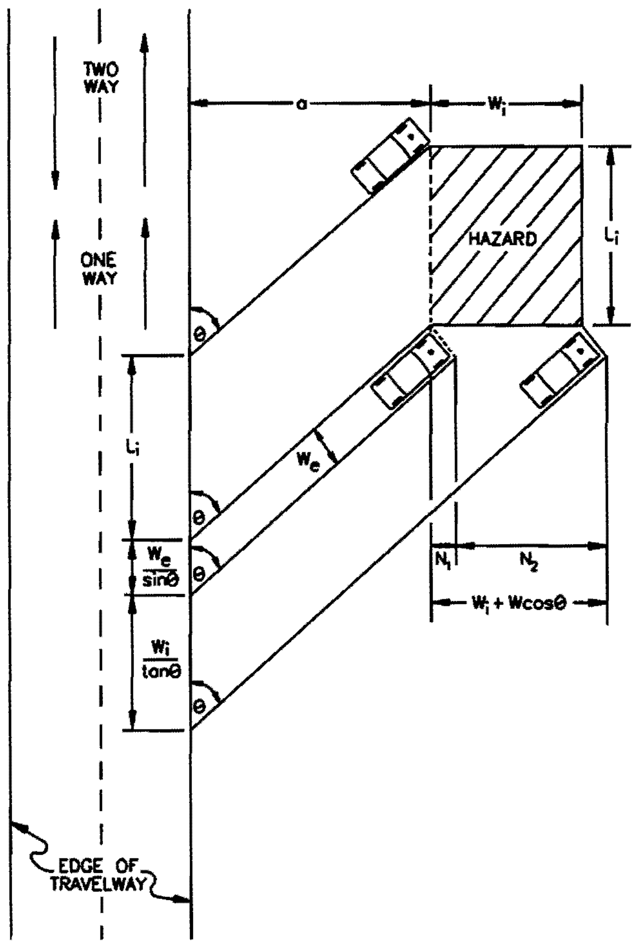

Figure 11. Hazard Imaging

$$P(H^{w,i}_{v,\theta} \mid E^{w}_{v,\theta}) = \frac{1}{5280}\left(L_i + \frac{W_e}{\sin\theta} + W_i\cos\theta\right) \qquad \textbf{(3)}$$

where:

$P(H^{w,i}_{v,\theta} \mid E^{w}_{v,\theta})$	=	Probability that an errant vehicle of size, W, encroaching at speed, V, and angle, θ, will be within the impact envelope of hazard, i, given that a vehicle of size, W, has encroached at speed, V, and angle, θ.
L_i	=	Length of hazard i
W_e	=	Effective width of vehicle size W
θ	=	Encroachment angle (deg.).
W_i	=	Width of hazard i

The hazard imaging algorithm assumes that the errant vehicles encroach along a straight path, i.e., no steering input. It takes into account the encroachment angle, the length and width of the hazard, and the vehicle width. The module also determines the impact speed and angle associated with the predicted impacts, which is used for estimation of the accident severity in the accident severity prediction module. These impact conditions are established from real-world accident data (29).

5.1.3 Accident Severity Prediction Module

Given that an accident has occurred, the accident severity prediction module estimates the severity of the accident and its associated costs. The severity of an accident is a function of many factors, including impact conditions (i.e., impact speed and angle), the size and weight of the impacting vehicle, and the nature of the impacted roadside object or feature. For a given roadside object or feature and impacting vehicle, the conditions under which the vehicle impacts the roadside object or feature determine the outcome and severity of the accident. In the case of a roadside safety device (e.g., guardrail, crash cushion, etc.), the performance limit of the safety device is also taken into account. When the impact conditions exceed the performance limit of the safety device, some catastrophic outcome could occur, and the severity of the impact is usually a function of the catastrophic outcome. For example, if the impact loading on a bridge railing is greater than its structural capacity, the impacting vehicle would penetrate the bridge railing and fall into the river or traffic below. The severity of the accident is determined not only by the impact with the bridge railing, but also by the fall of the vehicle off of the bridge.

The model uses a severity index (SI) as a surrogate measure for accident severity. The severity index scale of 0 to 10 was developed under the 1977 AASHTO *Guide for Selecting, Locating, and Designing Traffic Barriers* (30) through surveys of transportation and law enforcement experts as a tool for estimating the severity of roadside hazards. A 0 represents an accident with no significant property damage or injury, while a 10 corresponds to an accident with a 100 percent chance of a fatality. A probability of injury and fatality is assigned to each index as shown in Table 21.

SI values are assigned to each roadside hazard type and vary as a function of the impact speed and angle. The accident severity is then converted to accident or societal costs, $C(I_i)$, based on accident cost figures contained in the 1988 AASHTO *Roadside Design Guide* (2) as shown below.

Injury Severity	Accident Cost
Fatality	$ 500,000
Severe Injury	110,000
Moderate Injury	10,000
Slight Injury	3,000
Property Damage Only (Level 2)	2,000
Property Damage Only (Level 1)	500

Table 21. Severity Index by Accident Type Distribution (2)

Severity Index	Property Damage (1)	Property Damage (2)	Slight Injury	Moderate Injury	Severe Injury	Fatal Injury	Accident Cost, $
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
0.5	100.0	0.0	0.0	0.0	0.0	0.0	500
1.0	66.7	23.7	7.3	2.3	0.0	0.0	1,375
2.0	0.0	71.0	22.0	7.0	0.0	0.0	3,135
3.0	0.0	43.0	34.0	21.0	1.0	1.0	10,295
4.0	0.0	30.0	30.0	32.0	5.0	3.0	25,350
5.0	0.0	15.0	22.0	45.0	10.0	8.0	56,535
6.0	0.0	7.0	16.0	39.0	20.0	18.0	116,555
7.0	0.0	2.0	10.0	28.0	30.0	30.0	186,150
8.0	0.0	0.0	4.0	19.0	27.0	50.0	281,720
9.0	0.0	0.0	0.0	7.0	18.0	75.0	395,500
10.0	0.0	0.0	0.0	0.0	0.0	100.0	500,000

The accident cost figures include estimates of direct costs, such as wage loss, medical expense, insurance administration, legal/litigation cost, and property damage, but do not account for indirect costs, such as the consideration of a person's natural desire to live longer or protect the quality of one's life.

The cost of repairing roadside safety hardware is also estimated by the accident severity module. The process involves estimating the extent of damage based on impact energy. The repair cost for any given accident is then estimated from the extent of damage and unit repair costs supplied by the user. For example, results from full-scale crash testing and computer simulations can be used to determine the relationship between impact energy terms and length of guardrail damage. The repair cost is then the product of the length of damaged rail and the unit cost for repair.

5.1.4 Benefit/Cost Module

Benefits derived from a safety improvement are measured in terms of reduced accident or societal costs resulting from reduced accident frequency and/or severity. Costs associated with a safety improvement include the cost of initial installation, normal maintenance, and repair of damages from accidents. Computation of the incremental benefit/cost ratios is very straightforward once the benefits and costs are determined. The accident or societal costs and the construction and maintenance costs are first annualized, then the incremental benefit/cost ratio of each pair of alternatives under consideration is calculated. The formulation for determining the incremental benefit/cost ratio between two alternatives was shown previously in equation 1.

5.2 STUDY DESIGN

The major activities undertaken to determine the appropriate and cost-beneficial clear zone requirement for suburban high-speed curb sections were as follows:

1. Define typical site conditions for study;
2. Conduct benefit/cost analysis on the various clear zone widths for the typical site conditions; and
3. Develop clear zone guidelines.

The following sections present brief descriptions of these activities and the results.

5.2.1 Typical Site Conditions

An effort was made to define the typical site conditions for high-speed curb sections from review of data, e.g., photographs and videotapes, field measurements, traffic counts, etc., of highway sections sampled for the operational portion of the study. Table 22 summarizes the pertinent information on the sampled highway sections. Table 23 shows the typical site conditions selected for use with the benefit/cost analysis.

Review of the sampled highway sections showed that the highway types can be categorized into one of the following three types:

1. 4-lane, two-way undivided highway;
2. 4-lane, two-way undivided highway with two-way left-turn lane; and
3. 4-lane divided highway.

The more prevalent highway types were 4-lane, two-way undivided highways with or without two-way left-turn lanes, which were thus selected for the analysis. Note that the encroachment probability model accounts for the highway type but does not distinguish between highway sections with and without two-way left-turn lanes.

Speed limits on these sampled highway sections were between 80.5 to 88.5 km/h (50 to 55 mph). The highways typically had 3.7 m (12 ft) lane widths with curb and gutter cross sections. Most of the sampled highway sections had no shoulder and a few had shoulders 2.4 to 3.0 m (8 to 10 ft) in width. The alignment of the highways was typically straight and level. The traffic volumes ranged from approximately 2,000 to 20,000 AADT.

Table 22. Study Sites for Clear Zone Study

District	County	Highway	Description	AADT	Shoulder Width m (ft)	Clear Zone m (ft)	ROW Cost $\$/m^2$ ($\$/ft^2$)
5	Lamb	Loop 430	4-lane undivided	1,750	2.4 (8)	3.0 (10)	64.58 (6.0)
5	Lamb	US 84	4-lane TWLTL[1]	4,700	None	6.1 (20)	168.99 (15.7)
7	Tom Green	RM 584	4-lane TWLTL	5,900	None	≥9.1 (≥30)	
10	Henderson	SH 31	4-lane TWLTL	13,000	None		
10	Rusk	US 79	4-lane TWLTL	5,700	None	7.6 (25)	322.92 (30.0)
10	Smith	SH 64	4-lane TWLTL	8,900	3.0 (10)	7.6 (25)	122.71 (11.4)
10	Smith	SH 155	4-lane TWLTL	11,200	None	≥9.1 (≥30)	
10	Gregg	Loop 281	5-lane TWLTL	18,300	None	3.0 (10)	25.83 (2.4)
13	Victoria	SH 185	4-lane TWLTL	10,200	2.4 (8)		
13	Calhoun	SH 35	4-lane divided	12,500	3.0 (10)		
14	Bastrop	SH 21	4-lane divided, LT[2] bays	21,000	3.0 (10)	≥9.1 (≥30)	
14	Williamson	US 79	4-lane undivided	5,500	3.0 (10)		
15	Comal	SH 46	4-lane TWLTL	8,200	None	8.2 (27)	69.97 (6.5)
16	Nueces	FM 2444	4-lane TWLTL	9,200	None		
16	San Patricio	SH 35	4-lane divided	10,900	None		
23	Mills	US 84	4-lane undivided	3,500	3.0 (10)	5.8 (19)	24.76 (2.3)

[1] TWLTL -- Two-Way, Left-Turn Lane
[2] LT -- Left-Turn

Table 23. Typical Site Conditions for Clear Zone Study

- 4-lane, two-way undivided highway with or without two-way left-turn lane
- 3.7 m (12 ft) lane width, curb and gutter section
- No shoulder/3.0 m (10 ft) shoulder
- Straight and level alignment
- 80.5 to 88.5 km/h (50 to 55 mph) speed limit
- AADT - 2,000 to > 25,000
- Clear Zone Width - extends to right-of-way line
  - 3.0 m (10 ft)
  - 4.6 m (15 ft)
  - 6.1 m (20 ft)
  - 7.6 m (25 ft)
  - 9.1 m (30 ft)
- Roadside Conditions
  - Flat terrain beyond curb
  - Utility poles
  - Trees
- Roadside Hazard Rating
  - Low - Utility poles at ROW line, 76.2 m (250 ft) spacing
  - Medium - Utility poles at ROW line 76.2 m (250 ft) spacing + line of trees 1.5 m (5 ft) beyond ROW line, spaced 30.5 m (100 ft) apart
  - High - Utility poles at ROW line 76.2 m (250 ft) spacing + line of trees 1.5 m (5 ft) beyond ROW line, spaced 15.2 m (50 ft) apart
- Estimates of Direct Costs
  - Unit right-of-way acquisition cost = $\$21.53/m^2$ to $\$52.49/m^2$ ($\$2/ft^2$ to $\$16/ft^2$)
  - Unit clearing and grading cost = $\$0.25/m^2$ ($1,000/acre)
  - Unit relocation of utility pole cost = $1,500 per pole
- Traffic Growth Factor = 2.5% annually
- Percent Trucks = 0% (i.e., all passenger car traffic)
- Life of Project = 10 years
- Discount Rate = 4%

For the purpose of the benefit/cost analysis, the speed limit was set at 80.5 to 88.5 km/h (50 to 55 mph). Lane width was selected to be 3.7 m (12 ft) with curb and gutter. The alignment was assumed to be straight and level. The traffic volume was varied as appropriate to arrive at an incremental benefit/cost ratio of 1.0. Note that the encroachment probability model does not distinguish between highway sections with shoulders and those without.

The roadside conditions for these highway sections typically had flat terrain beyond the curb. There was generally a line of utility poles at the right-of-way line on one side of the highway, with trees, fences, commercial signs, and buildings beyond the right-of-way line. The density of roadside objects beyond the right-of-way line varied from highway section to highway section. There were numerous driveways and access points along the highway. The clear zone width typically varied with the right-of-way width and was clear of obstacles except for occasional sign supports.

For the purpose of the benefit/cost analysis, the clear zone width was assumed to extend to the right-of-way line and varied in width, starting with a minimum of 3.0 m (10 ft) and increased in 1.5 m (5 ft) increments. Flat terrain was assumed beyond the curb. A line of utility poles was assumed at the right-of-way line. A spacing of 76.2 m (250 ft) was used with the utility poles, which is a conservative estimate since utility pole spacing on rural highways can range up to 121.9 or 152.4 m (400 or 500 ft).

As discussed previously, the type, size, location, and density of roadside objects beyond the right-of-way line varied greatly among the sampled highway sections, ranging from a relatively clear, hazard-free condition to one cluttered with hazards such as trees, commercial signs, buildings, etc. As one would expect, the density, location, and other attributes of hazards present on the roadside affect the accident frequency calculated by the accident prediction module which, in turn, affects the results of the benefit/cost analysis. For the purpose of accounting for these effects of these variations in the benefit/cost analysis, it was decided to use a line of trees as a surrogate for the various roadside objects that may be present in a given location. Three levels of roadside hazard ratings were defined as follows:

Low	-	A line of utility poles at right-of-way line with 76.2 m (250 ft) spacing and clear roadside beyond right-of-way line.
Medium	-	A line of utility poles at right-of-way line with 76.2 m (250 ft) spacing and a line of trees 1.52 m (5 ft) beyond right-of-way line spaced 30.5 m (100 ft) apart.
High	-	A line of utility poles at right-of-way line with 76.2 m (250 ft) spacing and a line of trees 1.52 m (5 ft) beyond right-of-way line spaced 15.2 m (50 ft) apart.

These three roadside hazard ratings represent varying roadside conditions, from a relatively clear roadside (low rating) to a roadside cluttered with hazards (high rating). As the distance between trees (hazards) decreases, the probability of an accident increases. For a high hazard rating, with trees (hazards) spaced only 15.2 m (50 ft) apart, there is a probability of 85.2 percent that an encroaching vehicle would impact a tree (hazard). For a medium hazard rating, with trees (hazards) spaced 30.5 m (100 ft) apart, the probability of an impact with one of the trees (hazards) decreases to 42.6 percent.

Photographs illustrating the three roadside hazard ratings are shown in Figure 12. These photographs were selected from the roadway sections sampled for this study. Note that Figure 12(a), which represents a low roadside hazard rating, has a roadside relatively free of hazards beyond the line of utility poles. The roadside depicted in Figure 12(b), which was selected to represent a medium roadside hazard rating, is considerably more cluttered beyond the line of utility poles with hazards such as commercial signs, buildings, and trees. However, these hazards have a more discrete spacing than that of the high roadside hazard rating shown in Figure 12(c), which is densely populated with trees. By using these surrogate roadside hazard ratings, the effect of hazard frequency can be quantified in the benefit/cost analysis and a designer can select the rating that best describes the roadside condition for the specific highway section under study.

(a) Low

(b) Medium

(c) High

Figure 12. Typical Roadside Hazard Ratings

Note that the hazards associated with curbs, driveways, and small sign supports within the clear zone were not included in the analysis. The rationale for excluding these hazards in the benefit/cost analysis was twofold. First, since the analysis is comparative or incremental in nature, the effects of these hazards would be the same for all clear zone widths and, thus, would cancel each other out. Second, the severity associated with these hazards is relatively low and their presence is independent of the clear zone width.

The severity indices shown in Table 24 are proposed for inclusion in an update of the 1988 AASHTO *Roadside Design Guide* currently in preparation. As shown in this table, the severity associated with an accident increases with impact speed. For the B/C analysis, the trees were considered to be rigid point objects and were assigned severity values corresponding to the upper end of the range shown in Table 24. Although utility poles can also be considered rigid point objects for some impact conditions, accident studies (31) and crash tests (32) have shown that utility poles will fracture at ground level when the impact energy exceeds a certain level. The impact energy is a function of the mass and speed of the impacting vehicle. Based on a distribution presented by Mak and Mason (31) for all vehicle types, 50 percent of utility poles are knocked down when impacted at a speed of 64.4 km/h (40 mph). For this reason, the utility pole hazards were assigned average severity values for the ranges shown in Table 24.

There are other assumptions made for the inputs to the benefit/cost model, including a traffic growth factor of 2.5 percent annually, zero (0) percent trucks, i.e., all passenger car traffic, 10 years for the life of the project, and a 4 percent discount rate. The traffic growth factor of 2.5 percent annually represents the upper bound for traffic growth on such highways. The vehicle mix, i.e., percent truck, was believed to have little or no effect on the clear zone width and was thus assumed to be all passenger car traffic to simplify the analysis. The rationale for selecting a project life of 10 years was that the development and traffic growth on these suburban arterial highways will be such that they will effectively become urban roadways, with high traffic volume and lower speed limits, in 10 years. Thus, the costs for higher clear zone width requirements were amortized over a period of 10 years, which may or may not be the actual life of the project. The discount rate of 4 percent is a typical value used with benefit/cost analyses.

Table 24. Proposed Severity Indices

Type of Hazard	Impact Speed							
	64.4 km/h (40 mph)		80.5 km/h (50 mph)		96.5 km/h (60 mph)		112.6 km/h (70 mph)	
	Range	Avg.	Range	Avg.	Range	Avg.	Range	Avg.
Tree, Diameter > 102 mm (4 in)	2.6-5.0	3.8	3.2-6.0	4.6	3.8-7.2	5.5	4.4-8.6	6.5
Utility Pole	2.6-5.0	3.8	3.2-6.0	4.6	3.8-7.2	5.5	4.4-8.6	6.5

The direct costs associated with increasing the clear zone width included: right-of-way purchase cost, clearing cost, and the cost to relocate the utility poles. The cost for purchase of additional right-of-way for the sampled highway sections varied from a low of $21.53/m^2$ ($2/ft^2$) to a high of $322.92/m^2$ ($30/ft^2$) with a median of approximately $64.58/m^2$ ($6/ft^2$). The cost to clear and grade the additional clear zone was assumed to be $0.25/m^2$ ($1,000/acre). The cost to relocate the utility poles to the new right-of-way line was estimated to be $1,500 per pole based on best available estimates.

5.2.2 Benefit/Cost Analysis

The next activity was to determine the incremental benefits/costs associated with the various clear zone widths based on the typical site conditions. Incremental benefit/cost ratios were calculated for various combinations of the following:

1. Clear zone width;
2. Traffic volume (AADT);
3. Roadside hazard rating; and
4. Right-of-way purchase cost.

As mentioned previously, suburban arterial, high-speed, curb and gutter highway sections are typically not newly constructed, but are rather widened or reconstructed from existing two-lane rural highways. For a given right-of-way width, the addition of lanes to an existing highway will reduce the available clear zone to some value less than that previously existing along the roadway section. A baseline clear zone width was therefore assumed for each analysis, where the baseline clear zone is defined as the available clear zone width after widening of the highway if no additional right-of-way is purchased. For example, assume it is proposed to widen an existing 2-lane rural highway to include two additional 3.7 m (12 ft) lanes of traffic. If the existing roadway has a clear zone of 9.1 m (30 ft), and assuming no additional right-of-way is purchased, the existing clear zone would be reduced to 5.5 m (18 ft). Thus, for purposes of the benefit/cost analysis, this proposed 4-lane, suburban roadway would be said to have a baseline clear zone of 5.5 m (18 ft).

Data from the sampled highway sections indicated that a minimum of at least 3.0 m (10 ft) was typically available for use as a baseline clear zone width after widening. This should generally be the case given that most 2-lane rural highways have at least a 9.1 m (30 ft) clear zone before widening. If the existing roadway is widened to include 4 travel lanes and a two-way left-turn lane, the existing clear zone would be reduced to a baseline value of approximately 3.7 m (12 ft), assuming 3.7 m (12 ft) lane widths. Thus, the benefit/cost analysis begins with a baseline clear zone width of 3.0 m (10 ft).

For purposes of the analysis, the alternatives which were considered included widening of the baseline clear zone width in 1.5 m (5 ft) increments. In other words, if the baseline clear zone width was 3.0 m (10 ft) for alternative 1, the clear zone width would be 4.6 m (15 ft) for alternative 2, 6.1 m (20 ft) for alternative 3, 7.6 m (25 ft) for alternative 4, and 9.1 (30 ft) for alternative 5. The analysis is then repeated for baseline clear zone widths of 4.6 m (15 ft), 6.1 m (20 ft), and 7.6 m (25 ft).

The objective of the analysis was to determine under what conditions (i.e., ADT, roadside hazard rating, right-of-way acquisition cost) these alternatives are cost-effective. In other words, when considering a widening or reconstruction project, when is it cost-beneficial to make capital outlays to purchase additional right-of-way and provide additional clear zone width over the baseline value that would already be available after the roadway is improved?

For each baseline clear zone width, the analysis covered various combinations of roadside hazard rating (i.e., low, medium, and high) and right-of-way purchase cost. Analysis of these options and determination of appropriate clear zones were based on an incremental benefit/cost analysis. For a given roadside hazard rating and right-of-way purchase cost, the ADT value at which the incremental benefit/cost ratio becomes 1.0 was determined for each pair of alternatives under consideration. The appropriate alternative was then determined by first comparing each alternative to the baseline clear zone option, and then to each other. A typical input file used in the B/C analysis is given in Appendix E. The results are summarized in tabular format and discussed in the following section.

5.3 STUDY RESULTS

Results of the benefit/cost (B/C) analysis were used to develop tables which identify the most cost-beneficial clear zone width option for given combinations of baseline clear zone width, roadside hazard rating, and unit right-of-way (ROW) acquisition cost. Tables 25 through 28 show the range of traffic volumes (ADT) for which providing additional clear zone width is cost-beneficial for baseline clear zone widths of 3.0 m (10 ft), 4.6 m (15 ft), 6.1 m (20 ft), and 7.6 m (25 ft), respectively. The baseline clear zone width is defined as the clear zone width that would be available after a roadway is widened, assuming no additional right-of-way is acquired. The data in each of these tables is further subdivided according to high, medium, and low roadside hazard ratings which are denoted as (a), (b), and (c), respectively.

As shown in Table 22, the cost for purchase of additional right-of-way for the sampled highway sections varies from a low of $\$21.53/m^2$ ($\$2/ft^2$) to a high of $\$323/m^2$ ($\$30/ft^2$). When developing Tables 25 through 28, the unit ROW acquisition cost was varied in increments of $\$21.53/m^2$ ($\$2/ft^2$), starting at $\$21.53/m^2$ ($\$2/ft^2$), until the ADT at which the baseline clear zone width ceased to be cost-beneficial or exceeded 20,000 vehicles per day, which was the upper limit of the range obtained for the sampled sections of highways. For unit ROW acquisition costs in excess of those shown in the tables, it is not cost-effective to purchase additional ROW for purpose of providing additional clear zone width.

Some general observations can be made from review of these tables. First, it can be seen that, as the ROW acquisition cost increases, the ADT required to justify a particular clear zone width also increases. This is to be expected when one considers that, as the direct costs increase, it is necessary to have a corresponding increase in benefits in order to maintain a B/C ratio of 1.0. In the B/C analysis, benefits are measured in terms of reductions in accident costs which are directly related to the traffic volume. In other words, the same safety improvement can result in more benefits (i.e., reduced accident costs) when implemented on a roadway with a higher ADT.

Table 25. ADT Range for Which Providing Additional Clear Zone Width is Cost-Beneficial Based on a Baseline Clear Zone Width of 3.0 m (10 ft)

(a) High Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
3.0 (10)*	<7,200	<16,000	<22,300
4.6 (15)	N/A	16,000-17,400	22,300-25,700
6.1 (20)	7,200-11,400	17,400-24,400	25,700-34,000
7.6 (25)	11,400-17,100	24,400-32,200	34,000-43,700
9.1 (30)	≥17,100	≥32,200	≥43,700

(b) Medium Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
3.0 (10)*	<12,000	<22,000	
4.6 (15)	--	22,000-23,500	
6.1 (20)	12,000-16,700	23,500-31,500	
7.6 (25)	16,700-23,200	31,500-40,800	
9.1 (30)	≥23,200	≥48,800	

(c) Low Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
3.0 (10)*	<37,800		
4.6 (15)	--		
6.1 (20)	37,800-45,300		
7.6 (25)	45,300-57,000		
9.1 (30)	≥57,000		

* Baseline condition - clear zone width available after widening without any additional right-of-way purchase

Table 26. ADT Range for Which Providing Additional Clear Zone Width is Cost-Beneficial Based on a Baseline Clear Zone Width of 4.6 m (15 ft)

(a) High Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
4.6 (15)*	<12,600	<22,600	
6.1 (20)	--	22,600-24,300	
7.6 (25)	12,600-17,200	24,300-32,300	
9.1 (30)	≥17,200	≥32,300	

(b) Medium Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
4.6 (15)*	<18,000	<29,500	
6.1 (20)	--	29,500-31,500	
7.6 (25)	18,000-23,200	31,500-40,700	
9.1 (30)	≥23,200	≥48,700	

(c) Low Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
4.6 (15)*	<47,800		
6.1 (20)	--		
7.6 (25)	47,800-57,000		
9.1 (30)	≥57,000		

* Baseline condition - clear zone width available after widening without any additional right-of-way purchase

Table 27. ADT Range for Which Providing Additional Clear Zone Width is Cost-Beneficial Based on a Baseline Clear Zone Width of 6.1 m (20 ft)

(a) High Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
6.1 (20)*	<18,700	<30,500	
7.6 (25)	--	30,500-32,200	
9.1 (30)	≥18,700	≥32,200	

(b) Medium Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
6.1 (20)*	<25,000		
7.6 (25)	--		
9.1 (30)	≥25,000		

(c) Low Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $\$/m^2$ ($\$/ft^2$)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
6.1 (20)*	<60,000		
7.6 (25)	--		
9.1 (30)	≥60,000		

* Baseline condition - clear zone width available after widening without any additional right-of-way purchase

Table 28. ADT Range for Which Providing Additional Clear Zone Width is Cost-Beneficial Based on a Baseline Clear Zone Width of 7.6 m (25 ft)

(a) High Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
7.6 (25)*	<26,800		
9.1 (30)	≥26,800		

(b) Medium Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
7.6 (25)*	<34,500		
9.1 (30)	≥34,500		

(c) Low Roadside Hazard Rating

Clear Zone Width, m (ft)	Unit Right-of-Way Acquisition Cost, $/m² ($/ft²)		
	21.53 (2.0)	43.06 (4.0)	64.58 (6.0)
7.6 (25)*	<76,500		
9.1 (30)	≥76,500		

* Baseline condition - clear zone width available after widening without any additional right-of-way purchase

Second, it is evident from the tables that, for a given unit ROW acquisition cost, higher ADT values are required to justify additional clear zone width. This observation is similar to the first in that an increase in direct costs must be offset by a corresponding increase in benefits. However, in this case, the increase in direct costs is the result of purchasing additional ROW instead of higher unit acquisition price.

Another observation is that, for all the baseline clear zone widths considered, it is not cost-beneficial to purchase 1.5 m (5 ft) or less of additional ROW. As shown in Tables 25 through 28, a 1.5 m (5 ft) increase in clear zone width is either not cost-effective or has such a small range of ADT for which it could be considered cost-effective that it would be impractical to implement. This is due to the fact that the incremental benefits achieved over the baseline clear zone width are too small to justify the additional costs. For such small ROW purchases, the direct costs are driven by the utility pole relocation cost, which is a fixed cost based on the number of utility poles. As the clear zone width is further increased, the utility pole relocation cost becomes a smaller percentage of the direct costs and the incremental benefits become large enough to justify the increased expenditures. However, it should be noted that there may be other benefits associated with the acquisition of additional ROW, such as utility accommodation and the ability to tie in driveways, which were not accounted for in the B/C analysis. These considerations may affect the decision to purchase additional right-of-way regardless of the clear zone requirement and should be evaluated on a case-by-case basis.

It is also interesting to observe that, for unit ROW acquisition costs of $64.58/m² ($6/ft²) or greater, it is not cost-beneficial to provide additional clear zone width through the purchase of additional ROW. Since $64.58/m² ($6/ft²) was found to be the median ROW cost for the sampled highway sections, this would indicate that it is not cost-beneficial to provide additional clear zone width beyond the existing baseline condition for the majority of roadways.

The use of these tables to select a suitable clear zone width requires only basic information such as ADT, baseline clear zone width, unit ROW acquisition cost, and roadside hazard rating. For example, consider a highway section which has an ADT of 9,000, a baseline clear zone width of 3.0 m (10 ft), a unit ROW acquisition cost of $43.06/m² ($4/ft²), and a high roadside hazard rating, which correspond to the conditions of the upper shaded cell in Table 25(a). The table indicates that a clear zone width of 3.0 m (10 ft) is cost-beneficial for the specified conditions. Since this is equivalent to the baseline clear zone width, no additional ROW purchase would be required. If the same highway section had an ADT of 18,000, the lower shaded cell in Table 25(a) shows that a 6.1 m (20 ft) clear zone width would be cost-beneficial and, therefore, the purchase of an additional 3.0 m (10 ft) of ROW would be justified.

The data presented in Tables 25 through 28 are further condensed to provide some general clear zone width guidelines for high-speed curb sections. These guidelines are presented in Tables 29 through 31 for high, medium, and low roadside hazard ratings, respectively. Use of these tables requires the same basic roadway and roadside data, but is presented in a different format. For instance, consider a highway section with an ADT of 14,000, a baseline clear zone width of 4.6 m (15 ft), a unit ROW acquisition cost of $21.53/m² ($2/ft²), and a high roadside hazard rating. Table

Table 29. Clear Zone Requirements for High Roadside Hazard Rating

(a) Unit Right-of-Way Acquisition Cost = \$21.53/m^2 (\$2/ft^2)

Baseline Clear Zone Width m (ft)	AADT <8,000	AADT 8,000-12,000	AADT 12,000-16,000	AADT >16,000
3.0 (10)		6.1 (20)	7.6 (25)	9.1 (30)
4.6 (15)			7.6 (25)	9.1 (30)
6.1 (20)	No Additional Right of Way Required			9.1 (30)
7.6 (25)				

(b) Unit Right-of-Way Acquisition Cost = \$43.06/m^2 (\$4/ft^2)

Baseline Clear Zone Width m (ft)	AADT <8,000	AADT 8,000-12,000	AADT 12,000-16,000	AADT >16,000
3.0 (10)				6.1 (20)
4.6 (15)		No Additional Right of Way Required		
6.1 (20)				
7.6 (25)				

(c) Unit Right-of-Way Acquisition Cost > \$43.06/m^2 (\$4/ft^2)

Baseline Clear Zone Width m (ft)	AADT <8,000	AADT 8,000-12,000	AADT 12,000-16,000	AADT >16,000
3.0 (10)				
4.6 (15)			No Additional Right of Way Required	
6.1 (20)				
7.6 (25)				

Table 30. Clear Zone Requirements for Medium Roadside Hazard Rating

(a) Unit Right-of-Way Acquisition Cost = \$21.53/m^2 (\$2/ft^2)

Baseline Clear Zone Width m (ft)	AADT			
	<8,000	8,000-12,000	12,000-16,000	>16,000
3.0 (10)			6.1 (20)	7.6 (25)
4.6 (15)				7.6 (25)
6.1 (20)	No Additional Right of Way Required			
7.6 (25)				

(b) Unit Right-of-Way Acquisition Cost ≥ \$43.06/m^2 (\$4/ft^2)

Baseline Clear Zone Width m (ft)	AADT			
	<8,000	8,000-12,000	12,000-16,000	>16,000
3.0 (10)	No Additional Right of Way Required			
4.6 (15)				
6.1 (20)				
7.6 (25)				

Table 31. Clear Zone Requirements for Low Roadside Hazard Rating

All Unit Right-of-Way Acquisition Costs

Baseline Clear Zone Width m (ft)	AADT			
	<8,000	8,000-12,000	12,000-16,000	>16,000
3.0 (10)	No Additional Right of Way Required			
4.6 (15)				
6.1 (20)				
7.6 (25)				

29 (a) indicates that a 7.6-m (25-ft) clear zone is cost-beneficial. With a baseline clear zone width of 4.6 m (15 ft), the purchase of an additional 3.0 m (10 ft) of ROW would be required to attain a clear zone width of 7.6 m (25 ft).

Although the use of these tables is rather straightforward, it requires the use of different tables depending on the specific site conditions. It may be desirable to select one or two tables and use them as statewide guidelines for establishing clear zone width requirements for high-speed curb sections. Further discussions on Tables 29 through 31 may be helpful in the selection process and are presented as follows.

For highway sections with a high roadside hazard rating, Table 29 shows that additional clear zone width is not cost-beneficial when the unit ROW acquisition cost exceeds $43.06/m^2 ($4/ft^2). For highway sections with a medium hazard rating, Table 30 indicates that it is not cost-beneficial to provide additional clear zone width when the unit ROW acquisition cost equals or exceeds $43.06/m^2 ($4/ft^2). For highway sections with a low roadside hazard rating, Table 31 shows that keeping the existing baseline clear zone width is the most cost-beneficial option for all unit ROW acquisition costs considered in the analysis.

Based on data collected on the sampled highway sections, the typical or average high-speed curb section would have a medium roadside hazard rating and a median unit ROW acquisition cost of $64.58/m^2 ($6/ft^2), which corresponds to the conditions specified in Table 30(b). It is interesting to note that, for these average site conditions, the purchase of additional clear zone width is not cost-beneficial regardless of the ADT or baseline clear zone width.

The most conservative conditions would be a combination of a high roadside hazard rating and the lowest unit ROW acquisition cost, i.e., $21.53/m^2 ($2/ft^2), which corresponds to the conditions specified in Table 29(a). This table indicates that, for traffic volumes greater than 16,000 ADT, a 9.1 m (30 ft) clear zone width is cost-beneficial for baseline clear zone widths up to and including 6.1 m (20 ft). For traffic volumes between 12,000 and 16,000 ADT, a 7.6 m (25 ft) clear zone width is cost-beneficial for baseline clear zone widths of 3.0 m (10 ft) and 4.6 m (15 ft). For traffic volumes between 8,000 and 12,000 ADT, a 6.1 m (20 ft) clear zone width is cost-beneficial for a baseline clear zone width of 3.0 m (10 ft).

The conditions depicted in Table 29(b), i.e., high roadside hazard rating and unit ROW acquisition cost of $43.06/m^2 ($4/ft^2), are somewhere between the average and the most conservative conditions. The roadside hazard rating is high while the unit ROW acquisition cost of $43.06/m^2 ($4/ft^2) is between the lowest cost of $21.53/m^2 ($2/ft^2) and the median of $64.58/m^2 ($6/ft^2). The only instance in which additional clear zone width is cost-beneficial is for traffic volumes of greater than 16,000 ADT and a baseline clear zone width of 3.0 m (10 ft).

Similarly, the conditions specified in Table 30(a), i.e., medium roadside hazard rating and an unit ROW acquisition cost of $21.53/m^2 ($2/ft^2), are also somewhere between the average and the most conservative site conditions. The roadside hazard rating is medium while the unit ROW acquisition cost is the lowest cost at $21.53/m^2 ($2/ft^2). The table indicates that, for traffic volumes greater than 16,000 ADT, a 7.6 m (25 ft) clear zone width is cost-beneficial for baseline clear zone widths of 3.0 m (10 ft) and 4.6 m (15 ft). For traffic volumes between 12,000 and 16,000 ADT, a 6.1 m (20 ft) clear zone width is cost-beneficial for a baseline clear zone width of 3.0 m (10 ft).

When establishing the clear zone guidelines, it is desirable to be conservative so that the site conditions upon which the guidelines are based are valid for a vast majority of the roadways for which they will be applied. On the other hand, guidelines that are overly conservative could lead to too many applications that are not cost-beneficial. To maintain uniformity with existing guidelines contained in TxDOT's Design Manual, and to offer designers some flexibility in applying new guidelines to the widely varying conditions that will be encountered in the field, it was deemed appropriate to select two conditions for purposes of establishing minimum acceptable and desirable clear zone guidelines for suburban highways.

After careful consideration, it is felt that the conditions under Table 29(a), i.e., a high roadside hazard rating and a unit ROW acquisition cost of $21.53/m^2 ($2/ft ^{2}), would be an appropriate choice for establishing the desirable clear zone guidelines. This selection, which represents the most conservative conditions analyzed in the benefit/cost analysis, was made for three reasons. First, the clear zone widths indicated by this table should be conservative (i.e., greater than or equal to the most cost-effective value) for virtually all suburban roadways across the state. Second, due to the probabilistic nature of the B/C analysis and the assumptions inherent therein, a conservative selection is made to account for possible variations in some of the required input parameters discussed in Section 5.2. Third, it is well known that accident cost figures have a significant effect on the outcome of the B/C analysis, and the use of conservative site conditions would offset some of the effects of future increases in accident costs.

It is recognized that site conditions vary considerably and the attainment of the clear zone requirements of Table 29(a) may not be practical for all roadways. For instance, it is anticipated that right-of-way acquisition costs along some roadways will be as high as $322.92/m^2 ($30/ft^2), which is the maximum value obtained for the sampled highway sections (see Table 22). For such situations, it may not be economically feasible to obtain sufficient right-of-way to satisfy desirable clear zone guidelines (i.e., those contained in Table 29(a)) which are based on a conservative right-of-way acquisition cost of $21.53/m^2 ($2/ft^2). Thus, in order to afford designers some latitude in evaluating such situations, the establishment of a minimum acceptable set of clear zone requirements was considered necessary.

Upon further review of the results of the B/C analysis, Table 29(b) is recommended as the basis for establishing minimum acceptable clear zone guidelines for high-speed suburban highways. As discussed previously, the results in this table are still conservative considering they are based on the highest roadside hazard rating studied and a below-median ROW acquisition cost of $43.06/m^2 ($4/ft^2). However, the requirements of Table 29(b) are not as demanding as those in Table 29(a), and should provide some flexibility to help account for site variations.

For purposes of simplifying presentation, the results of Tables 29(a) and 29(b) have been combined into a single set of clear zone guidelines which are presented in Table 32. Implicit in these results are the various conservative site conditions upon which the original tables were based. By formulating the clear zone guidelines in this manner (i.e., using preselected conservative assumptions), variables such as roadside hazard rating, unit right-of-way acquisition cost, and baseline clear zone width are eliminated from the analysis. This will assure that multiple designers will arrive at similar solutions to specific clear zone questions.

The recommendations contained in Table 32 are rather straightforward to apply, requiring only the ADT of the roadway. For example, if a roadway which is programmed for widening has an ADT of 9,000, the recommended clear zone guidelines in Table 32 would suggest a minimum clear zone distance of 3 m (10 ft) and a desirable distance of 6.1 m (20 ft). The designer would then use the baseline clear zone projected after reconstruction to determine whether additional clear zone is required.

When making such a determination, one should recall that the results of the benefit/cost analysis indicated that, for all the baseline clear zone widths considered, it is not cost-beneficial to purchase 1.5 m (5 ft) or less of additional right-of-way strictly for the purpose of providing additional clear zone. This is due to the fact that, for such small ROW purchases, the direct costs associated with land acquisition, clearing, and utility pole relocation, exceed the incremental benefits achieved in terms of reduced accident costs. Thus, if a roadway has an ADT greater than 16,000 and a baseline clear zone width of 4.6 m (15 ft), it would not be cost effective to purchase an additional 1.5 m (5 ft) of right-of-way strictly for the purpose of providing the minimum clear zone distance of 6.1 m (20 ft) recommended in Table 32. However, there may be other benefits associated with the acquisition of additional ROW, such as the ability to tie in driveways, which were not accounted for in the B/C analysis and which should be evaluated on a case-by-case basis.

It should also be noted that, due to the probabilistic nature of the B/C analysis and the assumptions inherent therein, a certain degree of judgment should be exercised in the application of this data. The information presented in this section is intended to provide general guidance to the highway engineer in the selection of appropriate clear zones and some flexibility in the application of this information in establishing guidelines is considered acceptable.

Table 32. Recommended Clear Zone Guidelines

ADT	Recommended Clear Zone Distance, m (ft)	
	Minimum	Desirable
< 8,000	3.0 (10)	3.0 (10)
8,000 - 12,000	3.0 (10)	6.1 (20)
12,000 - 16,000	3.0 (10)	7.6 (25)
> 16,000	6.1 (20)	9.1 (30)

6.0 CONCLUSIONS AND RECOMMENDATIONS

The objective of this study was to develop geometric and roadside safety design guidelines for suburban, high-speed curb and gutter sections. The research conducted for this report included safety studies, operational studies, and clear roadside studies. Field data collection sites for the studies were selected from various areas throughout the state of Texas. The conclusions and recommendations are discussed in the following sections.

6.1 CONCLUSIONS

6.1.1 Safety Studies

The first study addressed in this report involved analyzing the safety effects of high-speed curb and gutter roadway sections. The safety effects were analyzed through accident rates, accident severities, and accident characteristic frequencies. The sample population included accident data from ten Texas suburban, high-speed curb and gutter sites before and after the site was modified to a curb and gutter cross section. Following are the conclusions that were drawn based on data available for this study.

1. Driveway density appears to have an effect on the safety of high-speed curb and gutter sections especially when coupled with the effect of increasing ADT. In areas of high driveway density due to high roadside development, the installation of a curb and gutter section did not increase the accident rate. This result is probably because drivers are aware of roadside development and expect vehicles to be frequently entering and exiting the roadway. Thus, drivers are more cognizant of potential interactions along the edge of the roadway and tend to decrease their speeds.

   When low density driveway sites were coupled with effects of varying ADT, the accident rate increased with increasing ADT. Drivers tend to drive at higher speeds along low driveway density roadway sections. These higher speeds coupled with high volumes resulted in higher accident rates for curb and gutter roadway sections.

2. The field data suggests that as ADT increases, curb and gutter on a high-speed roadway without shoulders causes an increase in accident rates and, therefore, a less safe driving environment.

3. Storm water ponding presents a problem for high-speed curb and gutter sections. For many of the study sites, the rate of accidents occurring on a slick roadway surface increased.

4. Visibility of the curb on high-speed curb and gutter sections is also a problem according to the study data. The results showed that the proportion of accidents involving impaired visibility were significantly higher for sites with curb and gutter than without curb and gutter.

5. The data also indicated that the rate of run-off-the-road accidents increased with the installation of curb and gutter. The results suggested that in addition to the rate, the severity of these accidents may be worse when curb and gutter is used on a high-speed suburban roadway than when parallel drainage ditches are utilized.

6.1.2 Operational Studies

The operational studies included two separate studies: (1) a study concerning shoulder requirements; and (2) a study concerning two-way left-turn lane (TWLTL) requirements. The first study evaluated the operational effects of a paved shoulder in high-speed, suburban curb sections. The operational effects studied included conflict rates, lane distributions and free-flow speeds. Independent samples from sections with and without shoulders were compared statistically to determine any differences in the operational characteristics of vehicles in those sections.

The second operational study analyzed the effects that a TWLTL would have on reducing the accident potentials for suburban highways with no existing medians. The study involved conducting a conflict analysis (left-turn, same direction) for sites with and without TWLTLs. Also, a probabilistic model was developed to predict left-turn, same direction conflicts based on traffic volume and driveway density. The results from this research were used to determine when a TWLTL would provide significant benefits (reduced accident potential) over no median. The conclusions drawn from the operational studies are as follows.

1. For a peak hour volume range of 0 to 1000, a higher conflict rate was observed in those sites without shoulders.

2. For those sites without shoulders, regression analysis indicated a positive linear relationship between conflict rate and traffic volume, defined by the equation $y = 0.0533\ (x)$, where y = conflict rate and x = peak hour traffic volume.

3. For those sites with shoulders, increases in traffic volume had little effect on the mean conflict rate of 19.96 conflicts per 1000 vehicles.

4. The point at which both types of cross section exhibit approximately equal conflict rates was near a peak hour volume of 350. Utilizing the K-factor range of 10 to 15 percent for suburban locations, this peak hour volume translates to an approximate ADT range of 3000 to 5000 vehicles per day. Above 5000 ADT, conflict rates continued to increase linearly if no shoulder was provided. Below 3000 ADT, the presence of shoulders had little effect on conflict rates.

5. For the range of data collected, those sites with shoulders produced a significantly higher proportion of vehicles in the right lane than did those without shoulders.

6. For the range of data collected, the 85th percentile speeds were accurate representations of the posted speed limits. Further, there was insufficient evidence to indicate that the presence or absence of shoulders influenced the difference in 85th percentile speeds and the posted speed limits.

7. For a peak hour volume range of 0 to 1000, there was no significant difference between the mean conflict rates of those study sites with TWLTLs and sites without TWLTLs; however, this may have been due to the limited study sites without TWLTLs. Many of these sites had very low volume levels which may have resulted in the low conflict rates.

8. A regression analysis indicated a positive linear relationship between the percent left turning vehicles and through volume / driveway density. The results yielded the following equation: $y = 0.001(x_1) + 0.325(x_2) + 0.737$, where y = percent left-turning vehicles, x_1 = peak hour volume, and x_2 = driveway density (drives per kilometer).

9. For those sites without TWLTLs, a regression analysis indicated a positive linear relationship between left-turn, same direction conflict rate, and traffic volume. This was defined by the equation $y = 0 + 0.0026\,(x)$, where y = conflict rate and x = peak hour volume.

10. For those sites with TWLTLs, increasing traffic volume had little effect on the conflict rate.

11. The results from this study indicated that TWLTLs provide significant reductions in accident potential for suburban highways with ADTs above approximately 2700 to 3000, depending on driveway density.

6.1.3 Clear Zone Study

The final research study addressed in this report was a clear roadside study. This study was undertaken to determine the most appropriate and cost-beneficial clear zone width requirements for suburban high-speed curb and gutter sections. Typical site conditions for this class of roadway were defined based on field data obtained from a selected sample of highway sections. An incremental benefit/cost (B/C) analysis was used to determine incremental B/C ratios for various combinations of clear zone width, traffic volume (ADT), roadside hazard rating, and unit right-of-way (ROW) acquisition cost. The results of this analysis were tabulated to identify ADT ranges for which different clear zone widths become cost-beneficial. Based on these results, the following conclusions were made:

1. It is not cost-beneficial to purchase 1.5 m (5 ft) or less of additional ROW strictly for the purpose of providing additional ROW;

2. For unit ROW acquisition costs greater than $43.06/m^2 ($4/ft^2), it is not cost-beneficial to provide additional clear zone width through the purchase of additional ROW; and

3. For roadways with a low roadside hazard rating, it is not cost-beneficial to provide additional clear zone width beyond the existing baseline clear zone width.

6.2 RECOMMENDATIONS

6.2.1 Safety Studies

From the safety studies, it appears that suburban, high-speed curb and gutter sections present several safety problems with respect to ponding and visibility. It also appears that ADT and driveway density are two indicators of the safety of high-speed curb and gutter sections, and, thus, these two operational variables should be considered when modifying a high-speed site to a curb and gutter cross section.

1. When driveway density is low and ADT is high, curb and gutter may not be a wise option as it may increase accident rates, and make the road less safe. On roadways with high driveway densities, however, curb and gutter may help the road to operate more safely.

2. It is recommended that when curb and gutter is installed on a high-speed suburban roadway, special care be given to the design of adequate drainage so as to prevent storm water ponding. This prevention of storm water ponding would ensure the safety of the section during inclement weather, and may be accomplished through many means including placement of inlets, adequate cross section sloping, and minimum grade requirements.

3. Nighttime lighting used to increase the visibility of the section should be considered. This lighting would allow the nighttime driver to see the line of the curb.

4. It is recommended that a future study be performed with specific emphasis on the safety concerns addressed above. The threshold where driveway density and ADT cause curb and gutter on a high-speed roadway to change from a safety advantage to an impediment should be more thoroughly studied.

6.2.2 Operational Studies

From the operational study concerning shoulder requirements and TWLTL requirements, the following recommendations were made.

1. The recommendations concerning the implementation of paved shoulders on suburban, high-speed curb sections are illustrated in Figure 13. It is recommended that a minimum paved shoulder width of 2.4 m (8 ft) be incorporated into the design of these sections for locations where ADT is expected to be in excess of 5000. Figure 13 presents evidence to indicate a reduction in conflict rate when a shoulder is provided in this volume range.

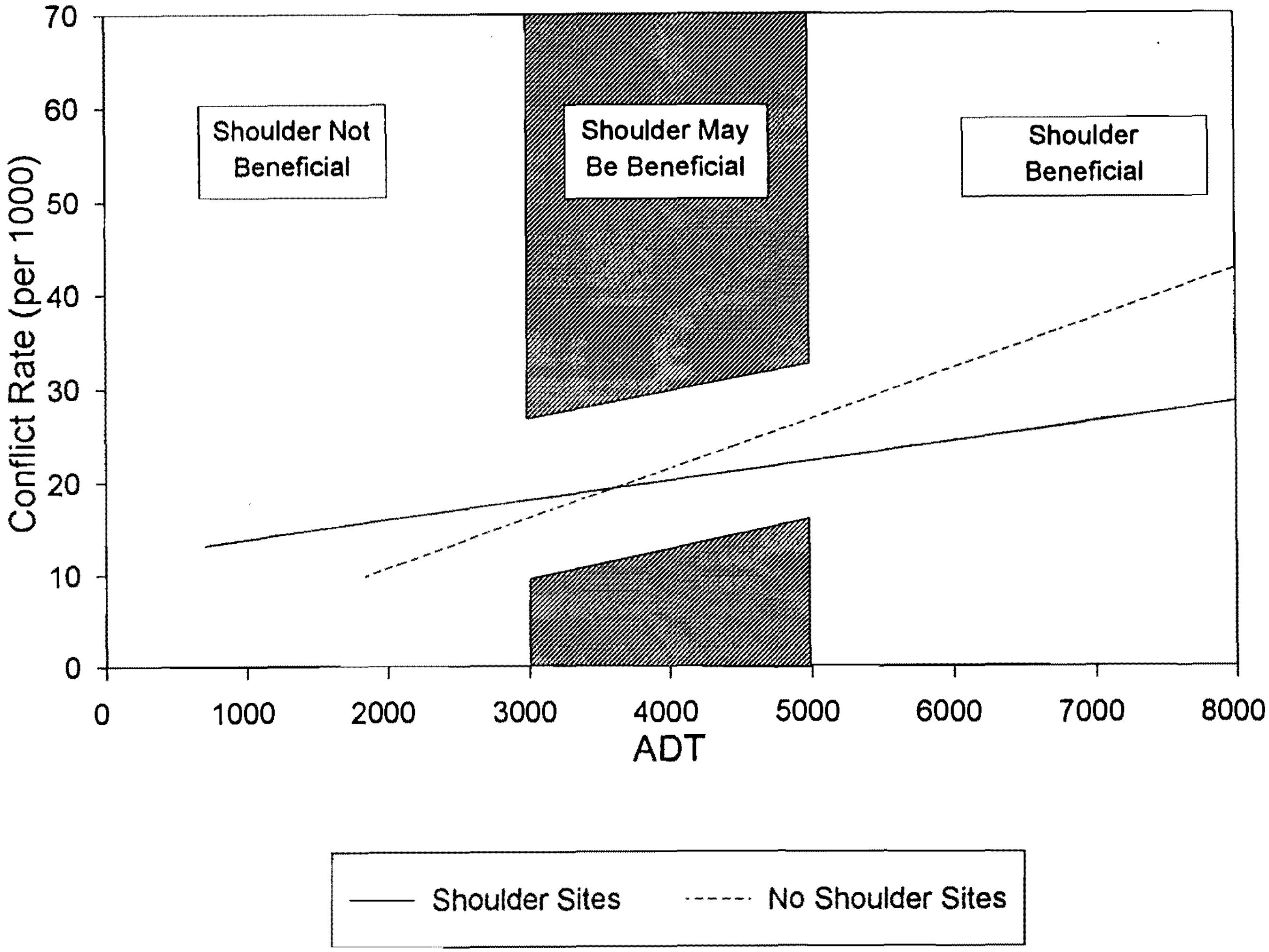

Figure 13. Guidelines for Shoulders in High-Speed Curb Sections

2. There is no indication that a significant reduction in conflict rate would be achieved through the use of a paved shoulder in those locations where an ADT of less than 3000 is expected.

3. For the ADT range of 3000 to 5000 vehicles per day, flexibility should be provided for the inclusion of a shoulder, depending on other circumstances, such as intensity of access and severity of right of way restriction.

4. The results of this study indicate that the proportion of vehicles in the right lane in those sites with shoulders is consistently greater than 50 percent. This would suggest that the concept of assuming a 50-50 lane distribution in design is flawed and further study could be warranted.

5. The recommendations concerning the implementation of TWLTLs on high-speed curb and gutter roadways are shown in Table 33. This table depicts where TWLTLs provide significant benefits (reduced accident potential) based on traffic volume (ADT) and driveway density. It is recommended that a TWLTL be incorporated into the design of high-speed curb and gutter roadways where ADT is expected to be greater than 2,700 to 3,000 (based on driveway density).

6. Below approximately 2,000 ADT, TWLTLs provide little benefit for reducing accident potential.

6.2.3 Clear Zone Study

For purposes of establishing a general clear zone policy, the results contained in Table 34 are recommended. The recommendations contained in this table are rather straightforward. For each of the four different ADT ranges, the minimum and desirable clear zone widths are provided. For example, for roadways with ADT between 8,000 and 12,000, the recommended minimum and desirable clear zone widths are 3.0 m (10 ft) and 6.1 m (20 ft), respectively. The designer would then use the baseline clear zone projected after reconstruction to determine whether additional clear zone is required.

As indicated by the footnote of Table 34, it is not cost-beneficial to purchase 1.5 m (5 ft) or less of additional right-of-way strictly for the purpose of providing additional clear zone. This is due to the fact that, for such small ROW purchases, the direct costs associated with land acquisition, clearing, and utility pole relocation, exceed the incremental benefits achieved in terms of reduced accident costs. Thus, for example, if a roadway has an ADT greater than 16,000 and a baseline clear zone width of 4.6 m (15 ft), it would not be cost-effective to purchase an additional 1.5 m (5 ft) of right-of-way strictly for the purpose of providing the minimum clear zone distance of 6.1 m (20 ft) recommended in Table 32. However, there may be other benefits associated with the acquisition of additional ROW, such as utility accommodation and the ability to tie in driveways, which were not accounted for in the B/C analysis and which should be evaluated on a case-by-case basis.

Table 33. Guidelines for Installing TWLTLs

Minimum Driveway Density drv/km (drv/mi)	Minimum ADT
6 (10)	3,000
12 (20)	2,900
18 (30)	2,800
24 (40)	2,700

Table 34. General Clear Zone Requirements

ADT	Recommended Clear Zone Distance[1], m (ft)	
	Minimum	Desirable
< 8,000	3.0 (10)	3.0 (10)
8,000 - 12,000	3.0 (10)	6.1 (20)
12,000 - 16,000	3.0 (10)	7.6 (25)
> 16,000	6.1 (20)	9.1 (30)

[1]Note: Purchase of 1.5 m (5 ft) or less of additional right-of-way strictly for satisfying clear-zone provisions is not cost-beneficial and, thus, not required.

It should be noted that the site conditions upon which the recommended table is based are conservative by design. This is necessary considering the wide range of roadway and roadside conditions for which these guidelines will be applied. However, considering the range of site conditions encountered among the highway sections which were sampled in this study, it is obvious that there will be some sites for which these recommendations will be very conservative and for which a reduced clear zone width may be justified. For these situations, it may be desirable to make a more precise determination of an appropriate clear zone width based on the actual characteristics of the roadway under consideration. The data tabulated in Tables 29 through 31 can be used for this purpose given that the roadside hazard rating, ADT, baseline clear zone width, and unit ROW acquisition cost are known.

Consider for example a roadway which has a baseline clear zone of 3.0 m (10 ft), a low roadside hazard rating, a unit ROW acquisition cost of $\$21.53/m^2$ ($\$2/ft^2$), and an ADT of 20,000. The clear zone guidelines shown in Table 34 would recommend clear zone width of 6.1 m (20 ft) based strictly on ADT. However, a more site-specific evaluation using Table 31 indicates that the baseline clear zone width of 3.0 m (10 ft) is satisfactory and no additional ROW purchase is required.

The recommendations from this report are outlined in Appendices F and G. These outlines are in a format similar to the urban street design section in TxDOT's Design Manual and contain the complete design guidelines for suburban highways. These guidelines were written in a manner such that they could be inserted into the Design Manual.

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APPENDIX A

STATE QUESTIONNAIRE

SUBURBAN DESIGN CRITERIA SURVEY
Texas Transportation Institute/Texas Department of Transportation

Name: ____________________

Agency/District: ____________________

Title/Responsibilities: ____________________

Address: ____________________

____

Phone: ____________________

1. Please note any existing roadways within your jurisdiction which utilize curb and gutter sections for drainage in combination with a posted speed 50 mph or greater, (attach additional pages if necessary).

Highway Destination	County	Control - Section	Speed Limits	Median/ Shoulder Type	Completion Date	Milepost Limits

1. ____________________
2. ____________________
3. ____________________
4. ____________________
5. ____________________
6. ____________________
7. ____________________
8. ____________________
9. ____________________
10. ____________________

2. Please describe, if indicated, any operational or safety problems associated with these designated type of roadways.

3. When confronted with these type of facilities in design, which do not specifically conform to urban street criteria nor multilane rural highway criteria, what guidelines do you follow in application? Why?

4. For high speed, suburban roadways with curb and gutter, please rank the following factors as to importance in the section design policy for these facilities (1 - most important, 10 - least important).

_____ Traffic Demand Volume	_____ Vehicle Turning Movements
_____ Accident Experience	_____ Adjacent Land Development
_____ Available Right-of-Way	_____ Design/Posted Speed
_____ Drainage Requirements	_____ Mail Boxes
_____ Utility Accommodation	_____ School Bus Route
_____ Intersection Sight Distance	_____ Bicycles
_____ Driveway Locations and Frequency	

5. Please state any suggestions or recommendations you feel are appropriate and feasible for roadside "clear zone" to be associated with suburban, high speed, curb and gutter facilities.

APPENDIX B

SUPPORTING DATA FOR SAFETY STUDIES

ACCIDENT RATE DATA

RAW ACCIDENTS: BEFORE											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	584	30	89	45	55	40	120	66	82	20	37
Acc2	175	11	22	5	12	16	32	26	36	9	6
Acc3	68	4	9	9	4	3	5	13	11	3	7
Acc4	61	3	8	9	4	3	4	12	10	3	5
Acc5	221	13	27	13	15	18	34	35	44	10	12
Acc6	22	0	3	0	0	1	9	2	6	1	0
Acc7	12	0	1	0	0	1	4	1	4	1	0
Acc8	67	6	5	2	8	4	23	2	5	3	9
Acc9	8	0	1	3	0	1	1	1	0	1	0
Acc10	75	11	11	3	5	1	14	11	13	3	3
Acc11	66	11	11	2	5	1	11	8	11	3	3
Acc12	84	2	11	11	5	5	8	19	13	4	6
Acc13	5	1	1	1	0	1	1	0	0	0	0

RAW ACCIDENTS: AFTER											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	814	42	161	35	97	79	69	123	138	54	16
Acc2	291	20	57	4	22	25	22	50	70	15	6
Acc3	118	2	25	1	15	18	8	22	18	6	3
Acc4	101	2	21	0	14	16	8	21	13	3	3
Acc5	361	21	71	5	34	36	29	60	78	18	9
Acc6	44	0	10	3	3	7	2	8	7	3	1
Acc7	28	0	7	2	3	4	1	6	3	1	1
Acc8	85	7	16	8	17	6	16	3	3	8	1
Acc9	9	3	0	1	0	1	1	0	2	1	0
Acc10	85	4	20	3	9	6	7	9	17	7	3
Acc11	79	4	18	3	7	6	7	9	15	7	3
Acc12	184	2	69	3	18	21	12	29	21	5	4
Acc13	13	0	7	0	2	2	0	1	0	1	0

ACCIDENT RATES: BEFORE											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	7.50	4.85	9.61	16.6	8.79	7.67	10.8	8.25	5.13	5.01	7.91
Acc2	2.25	1.78	2.38	1.84	1.92	3.07	2.82	3.25	2.25	2.25	1.28
Acc3	0.87	0.65	0.97	3.32	0.64	0.58	0.44	1.63	0.69	0.75	1.50
Acc4	0.78	0.48	0.86	3.32	0.64	0.58	0.35	1.50	0.63	0.75	1.07
Acc5	2.84	2.10	2.92	4.79	2.39	3.45	2.99	4.38	2.75	2.51	2.57
Acc6	0.28	0.00	0.32	0.00	0.00	0.19	0.79	0.25	0.38	0.25	0.00
Acc7	0.15	0.00	0.11	0.00	0.00	0.19	0.35	0.13	0.25	0.25	0.00
Acc8	0.86	0.97	0.54	0.74	1.28	0.77	2.03	0.25	0.31	0.75	1.92
Acc9	0.10	0.00	0.11	1.11	0.00	0.19	0.09	0.13	0.00	0.25	0.00
Acc10	0.96	1.78	1.19	1.11	0.80	0.19	1.23	1.38	0.81	0.75	0.64
Acc11	0.85	1.78	1.19	0.74	0.80	0.19	0.97	1.00	0.69	0.75	0.64
Acc12	1.08	0.32	1.19	4.05	0.80	0.96	0.70	2.38	0.81	1.00	1.28
Acc13	0.06	0.16	0.11	0.37	0.00	0.19	0.09	0.00	0.00	0.00	0.00

ACCIDENT RATES: AFTER											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	7.06	4.54	8.69	6.45	7.75	7.58	9.11	10.3	5.75	9.02	13.69
Acc2	2.52	2.16	3.08	0.74	0.76	2.40	2.91	4.17	2.92	2.51	5.13
Acc3	1.02	0.22	1.35	0.18	1.20	1.73	1.06	1.83	0.75	1.00	2.57
Acc4	0.88	0.22	1.13	0.00	1.12	1.53	1.06	1.75	0.54	0.50	13.69
Acc5	3.13	2.26	3.83	0.92	2.72	3.45	3.83	5.00	3.25	3.01	7.70
Acc6	0.38	0.00	0.54	0.55	0.24	0.67	0.26	0.67	0.29	0.50	0.86
Acc7	0.24	0.00	0.38	0.37	0.24	0.38	0.13	0.50	0.13	0.17	0.86
Acc8	0.74	0.76	0.86	1.47	1.36	0.58	2.11	0.25	0.13	1.34	0.00
Acc9	0.08	0.32	0.00	0.18	0.00	0.10	0.13	0.00	0.08	0.17	2.57
Acc10	0.74	0.43	1.08	0.55	0.72	0.58	0.92	0.75	0.71	1.17	2.57
Acc11	0.68	0.43	0.97	0.55	0.56	0.58	0.92	0.75	0.63	1.17	3.42
Acc12	1.60	0.22	3.72	0.55	1.44	2.01	1.58	2.42	0.88	0.84	0.00
Acc13	0.11	0.00	0.38	0.00	0.16	0.19	0.00	0.08	0.00	0.17	

DIFFERENCE IN ACCIDENT RATES											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	-0.4	-0.3	-0.9	-10.1	-1.0	-0.1	-1.5	2.0	0.6	4.0	5.8
Acc2	0.3	0.4	0.7	-1.1	-0.2	-0.7	0.1	0.9	0.7	0.3	3.9
Acc3	0.2	-0.4	0.4	-3.1	0.6	1.2	0.6	0.2	0.1	0.3	1.1
Acc4	0.1	-0.3	0.3	-3.3	0.5	1.0	0.7	0.3	-0.1	-0.3	1.5
Acc5	0.3	0.2	0.9	-3.8	0.3	0.0	0.8	0.6	0.5	0.5	5.1
Acc6	0.1	0.0	0.2	0.6	0.2	0.5	-0.5	0.4	-0.1	0.3	0.9
Acc7	0.1	0.0	0.3	0.4	0.2	0.2	-0.2	0.4	-0.1	-0.1	0.9
Acc8	-0.1	-0.2	0.3	0.7	0.1	-0.2	0.1	0.0	-0.2	0.6	-1.1
Acc9	-0.0	0.3	-0.1	-0.9	0.0	-0.1	0.0	-0.1	0.1	-0.1	0.0
Acc10	-0.2	-1.4	-0.1	-0.6	-0.1	0.4	-0.3	-0.6	-0.1	0.4	1.9
Acc11	-0.2	-1.4	-0.2	-0.2	-0.2	0.4	-0.0	-0.3	-0.1	0.4	1.9
Acc12	0.5	-0.1	2.5	-3.5	0.6	1.1	0.9	0.0	0.1	-0.2	2.1
Acc13	0.1	-0.2	0.3	-0.4	0.2	0.0	-0.1	0.1	0.0	0.2	0.0

PERCENT DIFFERENCE IN ACCIDENT RATES											
Site	All	1	2	3	4	5	6	7	8	9	10
Acc1	-6	-6	-10	-61	-12	-1	-14	24	12	80	73
Acc2	12	22	30	-60	-8	-22	3	28	30	11	300
Acc3	17	-67	39	-94	88	200	140	13	9	33	71
Acc4	12	-55	31	-100	75	167	200	17	-13	-33	140
Acc5	10	8	31	-81	14	0	28	2	18	17	200
Acc6	35	X	67	X	X	250	-67	167	-22	100	X
Acc7	58	X	250	X	X	100	-63	300	-50	-33	X
Acc8	-14	-22	60	100	6	-25	4	0	-60	78	-56
Acc9	-24	X	-100	-83	X	-50	50	-100	X	-33	X
Acc10	-24	-76	-9	-50	-10	200	-25	-46	-13	56	300
Acc11	-19	-76	-18	-25	-30	200	-5	-25	-9	56	300
Acc12	48	-33	213	-86	80	110	125	2	8	-17	167
Acc13	76	-100	250	-100	X	0	-100	X	X	X	X

FREQUENCY VALUES FOR ACCIDENT CHARACTERISTIC VARIABLES

RAW ACCIDENTS BEFORE BY SITE

Severity	1	2	3	4	5	6	7	8	9	10	Total
1	21	48	35	29	19	71	41	56	12	15	347
2	3	21	5	13	6	14	5	14	3	10	94
3	2	11	2	9	8	23	16	8	3	8	90
4	3	8	3	4	4	11	2	4	2	4	45
5	1	1	0	0	3	1	2	0	0	0	8
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Severity	1	2	3	4	5	6	7	8	9	10	Total
1	31	91	19	61	43	38	71	70	26	14	464
2	4	34	7	16	16	13	25	27	19	0	161
3	6	27	8	12	12	12	13	30	3	2	125
4	1	7	1	7	7	4	10	9	5	0	52
5	0	2	0	1	1	2	4	2	1	0	12
Total	42	161	35	79	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Severity	1	2	3	4	5	6	7	8	9	10	11	12	13
1	347	103	43	38	347	14	9	42	0	37	30	53	3
2	94	20	13	12	94	2	0	12	0	20	19	16	0
3	90	31	10	9	90	5	3	6	3	8	7	13	2
4	45	17	1	1	45	1	0	6	4	4	4	1	0
5	8	4	1	1	8	0	0	1	1	0	0	1	0
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Severity	1	2	3	4	5	6	7	8	9	10	11	12	13
1	464	147	66	59	464	24	15	49	0	36	31	82	8
2	161	44	29	24	161	3	1	18	0	26	26	31	3
3	125	64	20	17	125	9	7	13	5	17	16	22	1
4	52	29	2	0	52	4	2	5	4	4	4	1	1
5	12	7	1	1	12	4	3	0	0	2	2	2	0
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Light	1	2	3	4	5	6	7	8	9	10	Total
0	19	67	40	43	24	88	40	46	11	31	409
1	0	1	0	0	0	1	0	2	0	0	4
2	5	17	0	9	12	12	18	27	7	3	110
3	5	4	4	2	3	18	8	6	2	3	55
4	1	0	1	1	1	1	0	1	0	0	6
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Light	1	2	3	4	5	6	7	8	9	10	Total
0	22	104	31	75	54	47	73	68	39	10	523
1	1	0	1	0	2	1	3	1	0	0	9
2	2	37	1	9	20	2	30	55	11	1	168
3	16	19	2	12	2	19	12	13	4	5	104
4	1	1	0	1	1	0	5	1	0	0	10
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Light	1	2	3	4	5	6	7	8	9	10	11	12	13
0	409	0	46	42	409	6	2	53	7	49	42	55	2
1	4	4	1	1	4	2	1	0	1	0	0	1	0
2	110	110	13	10	110	9	7	6	0	13	12	17	2
3	55	55	8	8	55	5	2	7	0	5	4	11	1
4	6	6	0	0	6	0	0	1	0	2	2	0	0
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Light	1	2	3	4	5	6	7	8	9	10	11	12	13
0	523	0	70	60	523	18	11	61	7	57	53	80	6
1	9	9	2	2	9	3	2	1	1	0	0	2	0
2	168	168	31	24	168	18	12	9	1	14	12	33	6
3	104	104	14	14	104	4	2	14	0	13	13	21	1
4	10	10	1	1	10	1	1	0	0	1	1	2	0
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Visib.	1	2	3	4	5	6	7	8	9	10	Total
1	17	62	32	40	22	86	31	38	10	25	363
2	1	0	1	1	1	2	0	3	0	0	9
3	4	15	0	8	11	11	15	24	6	3	97
4	4	3	3	2	3	16	7	6	1	2	47
5	2	5	8	3	2	2	9	8	1	6	46
6	0	1	0	0	0	0	0	0	0	0	1
7	1	2	0	1	1	1	3	3	1	0	13
8	1	1	1	0	0	2	1	0	1	1	8
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Visib.	1	2	3	4	5	6	7	8	9	10	Total
1	21	90	30	63	43	40	63	60	36	7	453
2	2	1	1	1	2	1	6	2	0	0	16
3	2	29	1	8	14	2	25	47	8	1	137
4	15	16	2	10	2	18	7	11	4	5	90
5	1	14	1	12	11	7	10	8	3	3	70
6	0	0	0	0	1	0	2	0	0	0	3
7	0	8	0	1	6	0	5	8	3	0	31
8	1	3	0	2	0	1	5	2	0	0	14
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Visib.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	363	0	0	0	363	4	2	49	7	46	40	14	2
2	9	9	0	0	9	1	1	1	0	2	2	0	0
3	97	97	0	0	97	7	5	6	0	10	10	6	2
4	47	47	0	0	47	5	2	6	0	4	4	3	1
5	46	0	46	42	46	2	0	4	0	3	2	41	0
6	1	1	1	1	1	1	0	0	0	0	0	1	0
7	13	13	13	10	13	2	2	0	1	3	2	11	0
8	8	8	8	8	8	0	0	1	0	1	0	8	0
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Visib.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	453	0	0	0	453	10	6	52	6	54	50	16	5
2	16	16	0	0	16	4	3	1	0	1	1	1	0
3	137	137	0	0	137	15	10	5	1	13	11	8	5
4	90	90	0	0	90	4	2	13	1	12	12	7	0
5	70	0	70	60	70	8	5	9	1	3	3	64	1
6	3	3	3	3	3	0	0	0	0	0	0	3	0
7	31	31	31	24	31	3	2	4	0	1	1	25	1
8	14	14	14	14	14	0	0	1	0	1	1	14	1
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Weather	1	2	3	4	5	6	7	8	9	10	Total
0	26	80	36	51	37	115	53	71	17	30	516
1	3	8	9	4	3	4	12	10	3	5	61
2	0	0	0	0	0	0	0	0	0	0	0
3	0	1	0	0	0	0	1	0	0	2	4
7	1	0	0	0	0	1	0	1	0	0	3
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Weather	1	2	3	4	5	6	7	8	9	10	Total
0	40	136	34	82	61	61	101	120	48	13	696
1	2	21	0	14	16	8	21	13	3	3	101
2	0	1	0	0	1	0	0	0	0	0	2
3	0	1	1	0	1	0	1	5	3	0	12
7	0	2	0	1	0	0	0	0	0	0	3
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Weather	1	2	3	4	5	6	7	8	9	10	11	12	13
0	516	153	0	0	516	17	10	62	7	62	56	23	5
1	61	19	61	61	61	5	2	4	1	7	4	60	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0
3	4	2	4	0	4	0	0	1	0	0	0	1	0
7	3	1	3	0	3	0	0	0	0	0	0	0	0
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Weather	1	2	3	4	5	6	7	8	9	10	11	12	13
0	696	243	0	0	696	33	21	71	8	80	74	32	10
1	101	41	101	101	101	10	7	11	1	4	4	98	1
2	2	0	2	0	2	0	0	0	0	1	1	0	1
3	12	7	12	0	12	1	0	2	0	0	0	7	1
7	3	0	3	0	3	0	0	1	0	0	0	1	0
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Inter.	1	2	3	4	5	6	7	8	9	10	Total
0	10	20	21	17	10	29	11	19	5	10	152
1	3	31	6	13	4	19	14	19	1	12	122
2	3	12	10	16	7	24	0	2	4	12	90
3	14	26	8	9	19	48	41	42	10	3	220
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Inter.	1	2	3	4	5	6	7	8	9	10	Total
0	11	46	11	32	10	16	45	37	18	7	223
1	4	46	6	20	10	8	18	33	6	6	157
2	4	23	10	27	14	18	1	1	13	2	113
3	23	46	8	18	45	27	59	67	17	1	311
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Inter.	1	2	3	4	5	6	7	8	9	10	11	12	13
0	152	33	11	11	152	0	0	8	2	2	1	13	0
1	122	34	17	15	122	3	1	11	1	11	9	19	1
2	90	18	11	10	90	1	1	29	0	9	8	14	0
3	220	90	29	25	220	18	10	19	5	47	42	38	4
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Inter.	1	2	3	4	5	6	7	8	9	10	11	12	13
0	233	60	21	19	233	0	0	4	3	5	5	26	1
1	157	57	22	21	157	3	2	7	0	24	45	31	2
2	113	27	17	12	113	1	0	50	0	7	22	16	1
3	311	147	58	49	311	40	26	24	6	49	7	65	9
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Rd. Rel.	1	2	3	4	5	6	7	8	9	10	Total
0	23	82	41	53	26	104	37	57	18	35	476
1	7	4	3	2	12	7	26	17	1	1	80
2	0	3	1	0	2	9	3	8	1	1	28
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Rd. Rel.	1	2	3	4	5	6	7	8	9	10	Total
0	31	137	31	84	60	61	81	93	47	14	639
1	11	14	1	10	12	6	32	36	4	1	127
2	0	10	3	3	7	2	10	9	3	1	48
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Rd. Rel.	1	2	3	4	5	6	7	8	9	10	11	12	13
0	476	119	43	38	476	0	0	65	8	62	58	55	0
1	80	38	19	17	80	0	0	2	0	5	1	20	5
2	28	18	6	6	28	22	12	0	0	2	1	9	0
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Rd. Rel.	1	2	3	4	5	6	7	8	9	10	11	12	13
0	639	204	73	62	639	0	0	79	9	78	78	86	1
1	127	59	32	27	127	0	0	6	0	6	1	37	12
2	48	28	13	12	48	44	28	0	0	1	0	15	0
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

RAW ACCIDENTS BEFORE BY SITE

Surface	1	2	3	4	5	6	7	8	9	10	Total
0	26	78	34	50	33	111	47	67	15	31	492
1	2	11	11	5	5	8	19	13	4	6	84
2	0	0	0	0	0	0	0	0	1	0	1
4	1	0	0	0	0	0	0	0	0	0	1
5	1	0	0	0	2	1	0	2	0	0	6
Total	30	89	45	55	40	120	66	82	20	37	584

RAW ACCIDENTS AFTER BY SITE

Surface	1	2	3	4	5	6	7	8	9	10	Total
0	40	131	32	78	54	57	93	117	48	12	662
1	2	23	3	18	21	12	29	21	5	4	138
2	0	0	0	0	0	0	0	0	0	0	0
4	0	7	0	1	4	0	1	0	1	0	14
5	0	0	0	0	0	0	0	0	0	0	0
Total	42	161	35	97	79	69	123	138	54	16	814

RAW ACCIDENTS BEFORE BY ACCIDENT TYPE

Surface	1	2	3	4	5	6	7	8	9	10	11	12	13
0	492	145	3	0	492	14	8	62	7	60	54	0	4
1	84	29	61	60	84	8	4	5	1	8	5	84	0
2	1	0	0	0	1	0	0	0	0	0	0	0	0
4	1	1	1	0	1	0	0	0	0	0	0	0	0
5	6	0	3	1	6	0	0	0	0	1	1	0	1
Total	584	175	68	61	584	22	12	67	8	69	60	84	5

RAW ACCIDENTS AFTER BY ACCIDENT TYPE

Surface	1	2	3	4	5	6	7	8	9	10	11	12	13
0	662	226	5	0	662	31	20	71	8	76	71	138	10
1	138	58	106	98	138	13	8	14	1	7	6	0	2
2	0	0	0	0	0	0	0	0	0	0	0	0	0
4	14	7	7	3	14	0	0	0	0	2	2	0	1
5	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	814	291	118	101	814	44	28	85	9	85	79	138	13

ANOCAT TABLES FOR LOGLINEAR ANALYSIS

ANOCAT TABLE: All Accident Types				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	38.01	3.84	Reject
Severity	4	1256.46	9.49	Reject
Interaction	4	3.77	9.49	Accept
Total	9	1298.24	16.92	Accept

ANOCAT TABLE: Accident Type = 6				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	5.3	3.84	Reject
Severity	4	46.59	9.49	Reject
Interaction	3	0.65	7.81	Accept
Total	8	52.54	15.51	Reject

ANOCAT TABLE: Accident Type = 7				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	2.98	3.84	Accept
Severity	4	24.33	9.49	Reject
Interaction	1	0.18	3.84	Accept
Total	6	27.49	12.59	Reject

ANOCAT TABLE: All Accident Types				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	38.02	3.84	Reject
Visibility	1	39.36	3.84	Reject
Interaction	1	5.94	3.84	Reject
Total	3	83.32	7.81	Reject

ANOCAT TABLE: Accident Type = 6				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	23.29	3.84	Reject
Visibility	1	7.48	3.84	Reject
Interaction	1	0.18	3.84	Accept
Total	3	30.95	7.81	Reject

ANOCAT TABLE: Accident Type = 7				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	15.42	3.84	Reject
Visibility	1	6.58	3.84	Reject
Interaction	1	0.12	3.84	Accept
Total	3	22.12	7.81	Reject

ANOCAT TABLE: All Accidents				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	158.34	3.84	Reject
Lighting	1	38.01	3.84	Reject
Interaction	1	5.15	3.84	Reject
Total	3	201.50	7.81	Reject

ANOCAT TABLE: Accident Type = 6				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	4.97	3.84	Reject
Lighting	1	7.47	3.84	Reject
Interaction	1	1.21	3.84	Accept
Total	3	13.65	7.81	Reject

ANOCAT TABLE: Accident Type = 7				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	5.01	3.84	Reject
Lighting	1	6.59	3.84	Reject
Interaction	1	2.11	3.84	Accept
Total	3	13.71	7.81	Reject

ANOCAT TABLE: All Accident Types				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	38.01	3.84	Reject
Surface	2	1629.33	5.99	Reject
Interaction	2	2.05	5.99	Accept
Total	5	1669.39	11.07	Reject

ANOCAT TABLE: Accident Type = 6				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	7.48	3.84	Reject
Surface	1	8.93	3.84	Reject
Interaction	1	0.31	3.84	Accept
Total	3	16.72	7.81	Reject

ANOCAT TABLE: Accident Type = 7				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	6.58	3.84	Reject
Surface	1	6.58	3.84	Reject
Interaction	1	0.09	3.84	Accept
Total	3	13.25	7.81	Reject

ANOCAT TABLE: All Accident Types				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	38.02	3.84	Reject
Intersection	3	172.39	7.81	Reject
Interaction	3	1.84	7.81	Accept
Total	7	212.25	14.07	Reject

ANOCAT TABLE: Accident Types = 6				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	7.48	3.84	Reject
Intersection	2	87.27	5.99	Reject
Interaction	2	1.08	5.99	Accept
Total	5	95.83	11.07	Reject

ANOCAT TABLE: Accident Type = 7				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	7.67	3.84	Reject
Intersection	2	37.15	5.99	Reject
Interaction	1	0.04	3.84	Accept
Total	5	44.86	11.07	Reject

ANOCAT TABLE: All Accident Types				
Source	Df	L.R.X^2	SIG X^2	Decision
Road Type	1	38.02	3.84	Reject
Road Rel.	2	1333.92	5.99	Reject
Interaction	2	1.99	5.99	Accept
Total	5	1373.93	11.07	Reject

APPENDIX C

SUPPORTING DATA FOR CONFLICT ANALYSIS AND SPEED STUDY

County: Comal Highway: S.H. 46 Date: 6/17/93 Direction of Travel: Eastbound

Description: 4-lane TWLTL, No shld. COMACNAM.WQ1

Limits: 122 m (400 ft) west of Oelkers St. to 91.4 m (300 ft) east of Oelkers St.

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	41	0	27	4	72	-	-	-	-	-	-	-	-
7:15	48	1	33	3	85	-	-	-	-	-	-	-	-
7:30	57	0	38	8	103	-	-	1	1	-	-	-	-
7:45	52	2	32	3	89	-	-	-	-	-	-	-	-
8:00 A.M.	43	0	33	3	79	-	-	-	-	-	-	-	-
8:15	42	2	22	2	68	-	-	-	-	-	-	-	-
8:30	31	0	22	7	60	-	-	-	-	-	-	-	-
8:45	48	0	36	5	89	-	-	-	-	-	-	-	-
9:00 A.M.	47	2	29	1	79	-	-	-	-	-	-	-	-
9:15	40	2	25	5	72	-	-	-	-	-	-	-	-
9:30	46	0	27	4	77	-	-	-	-	-	-	-	-
9:45 A.M.	56	2	19	4	81	-	-	1	-	-	-	-	-
Totals	551	11	343	49	954	0	0	2	1	0	0	0	0

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	954	veh.	Total Volume	356	veh.
% Trucks	6.3%		% Trucks	5.6%	
% Left Lane	59%		% Left Lane	57%	
% Right Lane	41%		% Right Lane	43%	
Total Conf.	3		Total Conf.	2	
Conflict Rate	3.14	per 1000 veh.	Conflict Rate	5.62	per 1000 veh.

County: Comal Highway: S.H. 46 Date: 6/17/93 Direction of Travel: Eastbound

Description: 4-lane TWLTL, No shld. COMACNPM.WQ1

Limits: 122 m (400 ft) west of Oelkers St. to 91.4 m (300 ft) east of Oelkers St.

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	76	3	36	10	125	-	1	-	-	-	-	-	-
3:15	80	2	49	9	140	-	-	2	-	-	-	-	-
3:30	69	1	55	0	125	-	-	2	-	-	-	-	-
3:45	75	0	33	2	110	-	-	1	-	-	-	-	-
4:00 P.M.	97	2	45	5	149	-	-	-	-	-	-	-	-
4:15	83	1	46	1	131	-	-	-	-	-	-	-	-
4:30	83	2	46	4	135	-	-	-	-	-	-	1	-
4:45	106	4	33	3	146	-	-	1	-	-	-	-	-
5:00 P.M.	102	1	65	3	171	-	-	1	-	-	-	-	-
5:15	114	2	54	4	174	-	-	-	-	-	-	-	-
5:30	119	2	62	2	185	-	-	3	-	-	-	-	-
5:45 P.M.	91	0	48	4	143	-	-	-	-	-	-	-	-
Totals	1095	20	572	47	1734	0	1	10	0	0	0	1	0

Three Hour Observation Period			Peak Hour 4:45 P.M. to 5:45 P.M.		
Total Volume	1734	veh.	Total Volume	676	veh.
% Trucks	3.9%		% Trucks	3.1%	
% Left Lane	64%		% Left Lane	67%	
% Right Lane	36%		% Right Lane	33%	
Total Conf.	12		Total Conf.	5	
Conflict Rate	6.92	per 1000 veh.	Conflict Rate	7.40	per 1000 veh.

County: Gregg Highway: Loop 281 Date: 8/25/93 Direction of Travel: Northbound

Description: 5-lane, TWLTL, No Shldr. GREGCNAM.WQ1

Limits: From McDonald's to Gilmour Street 256 m (840 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	61	9	54	1	125	-	1	1	-	-	1	-	-
7:15	92	2	76	2	172	3	-	3	1	-	-	-	-
7:30	130	5	100	5	240	6	-	3	2	-	-	-	1
7:45	124	1	97	2	224	3	1	4	2	-	-	1	-
8:00 A.M.	66	7	73	6	152	3	-	5	1	-	-	-	1
8:15	67	6	55	2	130	3	1	1	1	-	-	-	1
8:30	58	13	55	3	129	-	1	1	1	-	-	1	1
8:45	73	7	62	4	146	2	-	1	1	1	-	-	-
9:00 A.M.	69	5	60	6	140	4	-	3	-	-	-	-	1
9:15	65	10	51	5	131	3	1	2	2	-	-	-	-
9:30	58	5	55	1	119	3	-	4	-	-	-	-	-
9:45 A.M.	70	12	66	2	150	-	-	1	-	1	-	-	-
Totals	933	82	804	39	1858	30	5	29	11	2	1	2	5

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	1858	veh.	Total Volume	788	veh.
% Trucks	6.5%		% Trucks	3.8%	
% Left Lane	55%		% Left Lane	54%	
% Right Lane	45%		% Right Lane	46%	
Total Conf.	85		Total Conf.	40	
Conflict Rate	45.75	per 1000 veh.	Conflict Rate	50.76	per 1000 veh.

County: Gregg Highway: Loop 281 Date: 8/25/93 Direction of Travel: Northbound

Description: 5-lane, TWLTL, No Shldr. GREGCNPM.WQ1

Limits: From McDonald's to Gilmour Street 256 m (840 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	75	9	63	5	152	1	-	5	2	-	-	-	-
3:15	73	10	50	2	135	-	-	-	-	-	-	-	-
3:30	95	6	71	5	177	3	1	3	1	-	-	-	-
3:45	119	6	79	3	207	5	-	7	2	-	1	2	1
4:00 P.M.	121	8	90	5	224	2	-	7	4	-	-	2	1
4:15	95	14	71	1	181	4	1	2	2	-	-	1	-
4:30	106	12	87	2	207	-	-	3	-	-	1	1	2
4:45	117	5	86	6	214	3	-	1	2	-	1	-	1
5:00 P.M.	154	6	110	3	273	3	1	8	3	-	1	-	1
5:15	140	9	98	3	250	5	-	6	6	-	-	-	1
5:30	124	3	82	3	212	5	-	4	4	-	-	1	2
5:45 P.M.	101	5	66	2	174	2	1	4	1	-	-	2	3
Totals	1320	93	953	40	2406	33	4	50	27	0	4	9	12

Three Hour Observation Period			Peak Hour 4:45 P.M. to 5:45 P.M.		
Total Volume	2406	veh.	Total Volume	949	veh.
% Trucks	5.5%		% Trucks	4.0%	
% Left Lane	59%		% Left Lane	59%	
% Right Lane	41%		% Right Lane	41%	
Total Conf.	139		Total Conf.	59	
Conflict Rate	57.77	per 1000 veh.	Conflict Rate	62.17	per 1000 veh.

County: Henderson Highway: SH 31 Date: 8/24/93 Direction of Travel: Eastbound

Description: 4-lane, TWLTL, No Shldr. HENDCNAM.WQ1

Limits: From Kidd Jones Shamrock to Rippy's Citgo 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	87	1	85	3	176	-	-	-	-	-	-	-	-
7:15	104	1	99	2	206	3	1	-	-	-	-	-	-
7:30	149	2	130	5	286	6	2	1	3	-	-	-	1
7:45	121	2	106	0	229	2	3	1	1	-	-	-	-
8:00 A.M.	91	1	82	2	176	5	1	-	-	-	-	-	-
8:15	66	4	82	3	155	4	-	-	1	-	-	-	-
8:30	79	3	88	5	175	3	-	1	-	-	-	-	-
8:45	68	1	74	7	150	2	-	-	-	-	-	-	-
9:00 A.M.	61	2	67	4	134	5	1	-	2	-	-	-	-
9:15	49	2	57	3	111	3	-	-	-	-	1	-	-
9:30	54	2	66	5	127	1	1	-	-	-	-	1	-
9:45 A.M.	59	3	70	7	139	3	1	-	1	-	-	-	-
Totals	988	24	1006	46	2064	37	10	3	8	0	1	1	1

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	2064	veh.	Total Volume	897	veh.
% Trucks	3.4%		% Trucks	1.7%	
% Left Lane	49%		% Left Lane	53%	
% Right Lane	51%		% Right Lane	47%	
Total Conf.	61		Total Conf.	30	
Conflict Rate	29.55	per 1000 veh.	Conflict Rate	33.44	per 1000 veh.

County: Henderson Highway: SH 31 Date: 8/24/93 Direction of Travel: Eastbound

Description: 4-lane, TWLTL, No Shldr. HENDCNPM.WQ1

Limits: From Kidd Jones Shamrock to Rippy's Citgo 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	50	2	58	4	114	1	-	-	-	1	-	-	-
3:15	42	7	58	9	116	2	-	1	1	-	-	-	-
3:30	51	5	52	7	115	1	-	1	1	-	-	-	-
3:45	63	7	50	9	129	1	-	1	-	-	-	-	1
4:00 P.M.	56	4	55	5	120	2	-	-	-	-	-	-	-
4:15	42	5	56	8	111	-	-	-	-	-	-	-	-
4:30	45	4	52	5	106	1	-	1	-	-	-	-	-
4:45	48	1	49	2	100	1	-	-	-	-	-	-	-
5:00 P.M.	65	6	66	3	140	1	-	-	-	-	-	-	-
5:15	53	1	61	4	119	2	-	-	-	-	-	-	-
5:30	42	2	53	7	104	1	-	-	-	-	-	-	-
5:45 P.M.	30	4	55	6	95	-	1	-	1	-	-	1	-
Totals	587	48	665	69	1369	13	1	4	3	1	0	1	1

Three Hour Observation Period			Peak Hour 3:15 P.M. to 4:15 P.M.		
Total Volume	1369	veh.	Total Volume	480	veh.
% Trucks	8.5%		% Trucks	11.0%	
% Left Lane	46%		% Left Lane	49%	
% Right Lane	54%		% Right Lane	51%	
Total Conf.	24		Total Conf.	12	
Conflict Rate	17.53	per 1000 veh.	Conflict Rate	25.00	per 1000 veh.

County: Lamb Highway: U.S. 84 Date: 8/16/93 Direction of Travel: Eastbound

Description: 4-lane, TWLTL, No Shld. LAM1CNAM.WQ1

Limits: From 76.2 m (250 ft) E. of Wilson St. to Austin St. 243.8 m (800 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	11	4	6	0	21	-	-	-	-	-	-	-	-
7:15	6	1	6	0	13	-	-	-	-	-	-	-	-
7:30	14	3	8	2	27	-	-	-	-	-	-	-	-
7:45	15	2	14	1	32	-	-	-	-	-	-	-	-
8:00 A.M.	24	3	12	0	39	-	-	-	-	-	-	-	-
8:15	18	5	17	3	43	-	-	-	-	-	-	-	-
8:30	26	0	17	1	44	-	-	-	-	-	-	-	-
8:45	18	0	18	2	38	-	1	-	-	-	-	-	-
9:00 A.M.	24	2	28	5	59	-	-	-	-	-	-	-	-
9:15	13	2	16	2	33	-	-	-	-	-	-	-	-
9:30	25	0	12	4	41	-	-	-	-	-	-	-	-
9:45 A.M.	16	1	20	2	39	-	-	-	-	-	-	-	-
Totals	210	23	174	22	429	0	1	0	0	0	0	0	0

Three Hour Observation Period			Peak Hour 8:15 A.M. to 9:15 A.M.		
Total Volume	429	veh.	Total Volume	184	veh.
% Trucks	10.5%		% Trucks	9.8%	
% Left Lane	54%		% Left Lane	51%	
% Right Lane	46%		% Right Lane	49%	
Total Conf.	1		Total Conf.	1	
Conflict Rate	2.33	per 1000 veh.	Conflict Rate	5.43	per 1000 veh.

County: Lamb Highway: U.S. 84 Date: 8/16/93 Direction of Travel: Eastbound

Description: 4-lane, TWLTL, No Shld. LAM1CNPM.WQ1

Limits: From 76.2 m (250 ft) E. of Wilson St. to Austin St. 243.8 m (800 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	20	2	18	5	45	-	-	-	-	-	-	-	-
3:15	21	3	20	5	49	1	-	-	-	-	-	-	-
3:30	26	2	26	2	56	-	-	-	-	-	-	-	1
3:45	19	1	18	1	39	-	-	-	-	-	-	-	-
4:00 P.M.	25	5	16	10	56	-	-	-	-	-	-	-	-
4:15	22	2	19	2	45	-	-	1	-	-	-	-	-
4:30	16	3	25	2	46	-	-	-	-	-	-	-	-
4:45	14	3	22	3	42	-	-	-	-	-	-	-	-
5:00 P.M.	26	7	29	2	64	-	-	1	-	-	-	-	-
5:15	19	5	27	5	56	1	-	-	-	-	-	-	-
5:30	13	3	21	12	49	-	-	-	-	-	-	-	-
5:45 P.M.	13	2	13	3	31	-	-	-	-	-	-	-	-
Totals	234	38	254	52	578	2	0	2	0	0	0	0	1

Three Hour Observation Period			Peak Hour 4:45 P.M. to 5:45 P.M.		
Total Volume	578	veh.	Total Volume	211	veh.
% Trucks	15.6%		% Trucks	19.0%	
% Left Lane	47%		% Left Lane	43%	
% Right Lane	53%		% Right Lane	57%	
Total Conf.	5		Total Conf.	2	
Conflict Rate	8.65	per 1000 veh.	Conflict Rate	9.48	per 1000 veh.

County: Nueces   Highway: F.M. 2444   Date: 8/30/93   Direction of Travel: Westbound

Description: 4-lane, TWLTL, No Shldr.   NUECCNAM.WQ1

Limits: Potter's Mill Apts. First Three Drives from the East. 170.7 m (560 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	24	0	21	1	46	1	-	-	-	-	-	-	-
7:15	38	1	33	2	74	2	-	-	-	-	-	-	-
7:30	33	1	39	1	74	1	-	1	-	-	-	-	-
7:45	42	2	52	0	96	2	-	1	1	-	-	-	-
8:00 A.M.	38	2	54	0	94	-	-	-	-	-	-	-	-
8:15	33	0	37	0	70	-	-	-	-	-	-	-	-
8:30	35	0	34	1	70	-	-	-	-	-	-	-	-
8:45	26	0	40	1	67	1	-	-	-	-	-	-	-
9:00 A.M.	26	0	33	1	60	1	-	-	1	-	-	-	-
9:15	31	0	27	2	60	-	-	-	-	-	-	-	-
9:30	25	1	34	1	61	-	-	-	-	-	-	-	-
9:45 A.M.	18	0	33	0	51	-	-	-	-	-	-	-	-
Totals	369	7	437	10	823	8	0	2	2	0	0	0	0

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	823	veh.	Total Volume	338	veh.
% Trucks	2.1%		% Trucks	2.7%	
% Left Lane	46%		% Left Lane	46%	
% Right Lane	54%		% Right Lane	54%	
Total Conf.	12		Total Conf.	8	
Conflict Rate	14.58	per 1000 veh.	Conflict Rate	23.67	per 1000 veh.

County: Nueces Highway: F.M. 2444 Date: 8/30/93 Direction of Travel: Westbound

Description: 4-lane, TWLTL, No Shldr. NUECCNPM.WQ1

Limits: Potter's Mill Apts. First Three Drives from the East. 170.7 m (560 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	29	1	42	0	72	2	-	-	-	-	-	-	-
3:15	31	3	65	1	100	2	-	1	1	-	-	-	-
3:30	44	1	49	2	96	2	-	-	-	-	-	-	-
3:45	52	0	61	1	114	3	-	-	1	-	-	-	-
4:00 P.M.	55	1	56	1	113	-	-	1	-	-	-	-	-
4:15	65	1	60	1	127	5	-	-	1	-	-	-	-
4:30	61	0	72	1	134	5	-	-	1	-	-	-	-
4:45	63	1	78	3	145	5	-	5	3	-	-	-	-
5:00 P.M.	94	0	102	1	197	8	-	3	2	-	-	-	-
5:15	84	0	106	0	190	6	-	2	3	-	-	-	-
5:30	100	0	133	0	233	10	-	1	7	-	-	-	-
5:45 P.M.	85	0	105	0	190	8	-	1	4	-	-	1	-
Totals	763	8	929	11	1711	56	0	14	23	0	0	1	0

Three Hour Observation Period			Peak Hour 5:00 P.M. to 6:00 P.M.		
Total Volume	1711	veh.	Total Volume	810	veh.
% Trucks	1.1%		% Trucks	0.1%	
% Left Lane	45%		% Left Lane	45%	
% Right Lane	55%		% Right Lane	55%	
Total Conf.	94		Total Conf.	56	
Conflict Rate	54.94	per 1000 veh.	Conflict Rate	69.14	per 1000 veh.

County: Rusk Highway: U.S. 79 Date: 8/26/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No Shldr. RUSKCNAM.WQ1

Limits: From St. Paul Street to People's State Bank 243.8 m (800 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	17	3	28	3	51	-	-	1	-	-	-	-	-
7:15	27	2	38	5	72	2	1	1	-	-	-	-	-
7:30	21	2	48	4	75	3	1	-	-	-	-	-	-
7:45	28	4	42	7	81	2	1	1	-	-	-	-	-
8:00 A.M.	24	2	42	3	71	2	-	-	1	-	-	-	1
8:15	36	1	23	4	64	1	-	-	-	-	-	1	-
8:30	31	0	32	3	66	-	-	1	-	-	-	-	-
8:45	37	3	27	6	73	-	-	-	-	-	-	-	-
9:00 A.M.	53	2	43	2	100	3	1	-	-	-	-	-	1
9:15	34	0	29	5	68	2	-	-	-	-	-	-	-
9:30	30	1	29	4	64	2	-	-	-	-	-	-	1
9:45 A.M.	43	3	33	4	83	2	-	1	-	-	-	-	-
Totals	381	23	414	50	868	19	4	5	1	0	0	1	3

Three Hour Observation Period			Peak Hour 9:00 A.M. to 10:00 A.M.		
Total Volume	868	veh.	Total Volume	315	veh.
% Trucks	8.4%		% Trucks	6.7%	
% Left Lane	47%		% Left Lane	53%	
% Right Lane	53%		% Right Lane	47%	
Total Conf.	33		Total Conf.	13	
Conflict Rate	38.02	per 1000 veh.	Conflict Rate	41.27	per 1000 veh.

County: Rusk Highway: U.S. 79 Date: 8/26/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No Shldr. RUSKCNPM.WQ1

Limits: From St. Paul Street to People's State Bank 243.8 m (800 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	45	2	35	4	86	-	-	1	-	-	1	-	1
3:15	45	1	42	3	91	1	-	-	-	-	-	-	1
3:30	66	2	48	8	124	2	-	2	2	-	-	-	1
3:45	66	2	46	5	119	5	1	1	3	-	-	-	2
4:00 P.M.	76	2	54	3	135	6	1	2	2	-	1	-	3
4:15	77	2	41	10	130	-	-	2	2	-	-	-	2
4:30	89	4	48	7	148	1	-	2	1	-	-	-	1
4:45	80	1	57	4	142	3	1	1	-	-	1	-	4
5:00 P.M.	99	0	47	3	149	2	1	2	-	-	-	-	1
5:15	84	1	58	5	148	3	-	1	1	-	-	-	5
5:30	92	0	47	4	143	5	1	3	2	-	-	1	6
5:45 P.M.	70	2	33	3	108	2	-	-	1	-	-	-	-
Totals	889	19	556	59	1523	30	5	17	14	0	3	1	27

Three Hour Observation Period			Peak Hour 4:30 P.M. to 5:30 P.M.		
Total Volume	1523	veh.	Total Volume	587	veh.
% Trucks	5.1%		% Trucks	4.3%	
% Left Lane	60%		% Left Lane	61%	
% Right Lane	40%		% Right Lane	39%	
Total Conf.	97		Total Conf.	31	
Conflict Rate	63.69	per 1000 veh.	Conflict Rate	52.81	per 1000 veh.

County: San Patricio Highway: S.H. 35 Date: 6/23/93 Direction of Travel: Northbound

Description: 4-lane divided , TWLTL, No Shldr. SANPCNAM.WQ1

Limits: Douglas Machine Works to Coastal Bend Bowling Lanes 225.6 m (740 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	20	1	23	2	46	-	-	-	-	-	-	-	-
7:15	23	2	26	3	54	-	-	-	-	-	-	-	-
7:30	51	0	46	5	102	-	-	1	-	-	-	-	-
7:45	50	1	54	3	108	1	-	-	-	-	-	-	-
8:00 A.M.	41	0	39	2	82	-	-	1	-	-	-	-	-
8:15	42	0	52	2	96	-	-	-	-	-	-	-	-
8:30	31	0	37	1	69	-	-	-	-	-	-	-	-
8:45	37	0	37	3	77	-	-	-	-	-	-	-	-
9:00 A.M.	38	0	31	3	72	-	-	1	-	-	-	-	-
9:15	31	1	30	1	63	-	-	-	-	-	-	-	-
9:30	41	0	35	1	77	-	-	-	-	-	-	-	-
9:45 A.M.	34	1	38	2	75	-	-	-	-	-	-	-	-
Totals	439	6	448	28	921	1	0	3	0	0	0	0	0

Three Hour Observation Period			Peak Hour 7:30 A.M. to 8:30 A.M.		
Total Volume	921	veh.	Total Volume	388	veh.
% Trucks	3.7%		% Trucks	3.4%	
% Left Lane	48%		% Left Lane	48%	
% Right Lane	52%		% Right Lane	52%	
Total Conf.	4		Total Conf.	3	
Conflict Rate	4.34	per 1000 veh.	Conflict Rate	7.73	per 1000 veh.

County: San Patricio Highway: S.H. 35 Date: 6/23/93 Direction of Travel: Northbound

Description: 4-lane divided , TWLTL, No Shldr. SANPCNPM.WQ1

Limits: Douglas Machine Works to Coastal Bend Bowling Lanes 225.6 m (740 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	49	0	50	3	102	-	-	-	-	-	-	-	-
3:15	36	4	54	0	94	-	1	-	-	-	-	1	-
3:30	47	4	51	1	103	-	-	-	-	-	-	-	-
3:45	44	0	60	3	107	-	-	1	-	-	-	-	-
4:00 P.M.	45	0	53	0	98	1	-	-	-	-	-	-	2
4:15	54	1	61	1	117	-	-	2	-	-	-	-	-
4:30	53	1	52	1	107	1	-	-	-	-	-	-	-
4:45	53	0	56	2	111	-	-	1	1	-	-	-	1
5:00 P.M.	50	0	58	1	109	-	-	-	-	-	-	-	-
5:15	55	1	63	1	120	-	-	2	1	1	-	-	-
5:30	44	0	56	0	100	-	-	-	-	-	-	-	-
5:45 P.M.	52	0	57	3	112	-	-	-	-	1	-	-	-
Totals	582	11	671	16	1280	2	1	6	2	2	0	1	3

Three Hour Observation Period			Peak Hour 4:30 P.M. to 5:30 P.M.		
Total Volume	1280	veh.	Total Volume	447	veh.
% Trucks	2.1%		% Trucks	1.6%	
% Left Lane	46%		% Left Lane	48%	
% Right Lane	54%		% Right Lane	52%	
Total Conf.	17		Total Conf.	8	
Conflict Rate	13.28	per 1000 veh.	Conflict Rate	17.90	per 1000 veh.

County: Smith Highway: SH 155 Date: 6/30/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No shldr. SMI1CNAM.WQ1

Limits: 146.3 m (480 ft) North of Westway St. to 167.6 m (550 ft) South of Westway Street 314 m (1030 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	13	0	23	1	37	1	-	-	-	-	-	-	-
7:15	35	1	31	5	72	-	-	1	-	-	-	-	-
7:30	30	0	35	3	68	2	-	-	-	-	-	-	1
7:45	40	1	50	4	95	1	-	-	-	-	-	-	-
8:00 A.M.	50	0	62	2	114	1	1	-	-	-	-	-	-
8:15	54	2	37	9	102	2	-	-	-	-	-	-	-
8:30	49	3	33	6	91	1	-	2	1	-	-	-	-
8:45	36	1	41	5	83	-	-	-	-	-	-	-	-
9:00 A.M.	34	2	37	2	75	-	-	-	-	-	-	-	-
9:15	38	2	37	8	85	1	-	1	-	-	-	-	-
9:30	35	3	33	7	78	-	-	1	-	-	-	-	-
9:45 A.M.	35	0	47	5	87	2	-	-	-	-	-	-	-
Totals	449	15	466	57	987	11	1	5	1	0	0	0	1

Three Hour Observation Period			Peak Hour 7:45 A.M. to 8:45 A.M.		
Total Volume	987	veh.	Total Volume	402	veh.
% Trucks	7.3%		% Trucks	6.7%	
% Left Lane	47%		% Left Lane	50%	
% Right Lane	53%		% Right Lane	50%	
Total Conf.	19		Total Conf.	9	
Conflict Rate	19.25	per 1000 veh.	Conflict Rate	22.39	per 1000 veh.

County: Smith Highway: SH 155 Date: 6/30/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No shldr. SMI1CNPM.WQ1

Limits: 146.3 m (480 ft) North of Westway St. to 167.6 m (550 ft) South of Westway Street 314 m (1030 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	64	2	63	2	131	2	-	1	1	-	-	-	1
3:15	64	0	70	4	138	1	-	1	-	-	-	-	-
3:30	87	0	67	8	162	5	3	3	-	-	-	-	-
3:45	73	1	66	4	144	1	2	1	-	-	-	-	-
4:00 P.M.	76	0	74	3	153	1	2	-	1	-	1	-	-
4:15	85	0	83	1	169	2	1	1	1	-	-	-	1
4:30	95	0	77	2	174	1	-	2	-	-	-	-	1
4:45	85	0	72	4	161	2	3	2	-	-	-	-	-
5:00 P.M.	114	1	110	2	227	4	-	1	1	-	-	-	-
5:15	130	2	118	2	252	5	1	3	1	-	-	-	-
5:30	112	0	106	1	219	4	-	3	1	-	-	-	-
5:45 P.M.	98	0	74	3	175	4	1	1	-	-	-	-	-
Totals	1083	6	980	36	2105	32	13	19	6	0	1	0	3

Three Hour Observation Period			Peak Hour 5:00 P.M. to 6:00 P.M.		
Total Volume	2105	veh.	Total Volume	873	veh.
% Trucks	2.0%		% Trucks	1.3%	
% Left Lane	52%		% Left Lane	52%	
% Right Lane	48%		% Right Lane	48%	
Total Conf.	74		Total Conf.	30	
Conflict Rate	35.15	per 1000 veh.	Conflict Rate	34.36	per 1000 veh.

County: Tom Green Highway: R.M. 584 Date: 8/19/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No Shld. TOMGCNAM.WQ1

Limits: From 30.5 m (100 ft) No. of Red Bluff Rd. to 167.6 m (550 ft) So. of Red Bluff Rd. 198.2 m (650 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	21	0	18	5	44	1	-	-	-	-	-	-	-
7:15	18	0	25	0	43	-	-	-	-	-	-	-	-
7:30	29	2	25	4	60	-	-	-	-	-	-	-	-
7:45	56	0	43	1	100	2	-	1	-	-	-	-	-
8:00 A.M.	37	0	28	1	66	1	-	-	-	-	-	-	-
8:15	35	0	22	1	58	-	-	-	-	-	-	-	-
8:30	25	0	28	0	53	-	-	-	-	-	-	-	-
8:45	20	0	22	1	43	-	-	-	-	-	-	-	-
9:00 A.M.	22	0	23	1	46	-	1	-	-	-	-	-	-
9:15	27	0	20	1	48	-	-	-	-	-	-	-	-
9:30	24	1	13	2	40	-	-	-	-	-	-	-	-
9:45 A.M.	23	0	21	3	47	-	-	-	-	-	-	-	-
Totals	337	3	288	20	648	4	1	1	0	0	0	0	0

Three Hour Observation Period			Peak Hour 7:30 A.M. to 8:30 A.M.		
Total Volume	648	veh.	Total Volume	284	veh.
% Trucks	3.5%		% Trucks	3.2%	
% Left Lane	52%		% Left Lane	56%	
% Right Lane	48%		% Right Lane	44%	
Total Conf.	6		Total Conf.	4	
Conflict Rate	9.26	per 1000 veh.	Conflict Rate	14.08	per 1000 veh.

County: Tom Green Highway: R.M. 584 Date: 8/19/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, No Shld. TOMGCNPM.WQ1

Limits: From 30.5 m (100 ft) No. of Red Bluff Rd. to 167.6 m (550 ft) So. of Red Bluff Rd. 198.1 m (650 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	41	1	30	0	72	2	-	-	-	-	-	-	-
3:15	33	1	25	1	60	-	-	-	-	-	-	-	-
3:30	39	0	30	3	72	-	-	-	-	-	-	-	-
3:45	47	2	18	3	70	-	-	-	-	-	-	-	-
4:00 P.M.	36	1	29	1	67	-	-	-	-	-	-	-	-
4:15	62	0	47	0	109	-	-	-	-	-	-	-	-
4:30	34	0	29	1	64	1	-	-	-	-	-	-	-
4:45	67	1	36	1	105	-	-	1	-	-	-	-	-
5:00 P.M.	79	0	51	4	134	-	-	-	-	-	-	-	-
5:15	72	0	58	1	131	2	-	-	1	-	-	-	-
5:30	68	0	43	0	111	-	-	-	-	-	-	-	-
5:45 P.M.	65	0	44	0	109	3	-	1	1	-	-	-	-
Totals	643	6	440	15	1104	8	0	2	2	0	0	0	0

Three Hour Observation Period			Peak Hour 5:00 P.M. to 6:00 P.M.		
Total Volume	1104	veh.	Total Volume	485	veh.
% Trucks	1.9%		% Trucks	1.0%	
% Left Lane	59%		% Left Lane	59%	
% Right Lane	41%		% Right Lane	41%	
Total Conf.	12		Total Conf.	8	
Conflict Rate	10.87	per 1000 veh.	Conflict Rate	16.49	per 1000 veh.

County: Bastrop Highway: SH 21 Date: 6/16/93 Direction of Travel: Westbound

Description: 4-lane divided , 3.0 m (10 ft) shld., LT ba BASTCNAM.WQ1

Limits: Smith Street to Eskew Street 225.6 m (740 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	68	2	82	10	162	-	-	-	-	-	-	-	-
7:15	81	0	116	8	205	1	1	-	-	-	-	-	-
7:30	74	2	106	5	187	1	1	-	-	-	-	-	-
7:45	61	2	87	9	159	1	1	-	1	-	1	-	-
8:00 A.M.	69	3	88	5	165	-	1	-	-	-	-	-	-
8:15	59	1	71	9	140	-	-	-	-	-	-	-	-
8:30	54	0	78	4	136	1	-	-	-	-	-	-	-
8:45	52	0	71	12	135	-	2	-	-	-	-	-	-
9:00 A.M.	54	3	74	11	142	-	-	1	-	-	-	-	-
9:15	55	1	91	6	153	2	2	-	1	-	-	-	-
9:30	57	2	80	11	150	-	1	-	1	-	-	-	-
9:45 A.M.	53	0	85	6	144	-	1	-	-	-	-	-	-
Totals	737	16	1029	96	1878	6	10	1	3	0	1	0	0

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	1878	veh.	Total Volume	716	veh.
% Trucks	6.0%		% Trucks	4.7%	
% Left Lane	40%		% Left Lane	41%	
% Right Lane	60%		% Right Lane	59%	
Total Conf.	21		Total Conf.	9	
Conflict Rate	11.18	per 1000 veh.	Conflict Rate	12.57	per 1000 veh.

County: Bastrop Highway: SH 21 Date: 6/16/93 Direction of Travel: Westbound

Description: 4-lane divided , 3.0 m (10 ft) shld., LT ba BASTCNPM.WQ1

Limits: Smith Street to Eskew Street 225.6 (740 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	43	1	72	5	121	-	1	-	-	-	-	-	1
3:15	56	0	87	3	146	2	-	1	-	-	-	-	-
3:30	57	0	106	3	166	-	-	-	-	2	-	-	1
3:45	55	1	89	7	152	-	-	-	-	-	-	-	-
4:00 P.M.	48	2	94	6	150	-	2	-	-	-	-	-	-
4:15	69	4	94	10	177	1	1	3	1	2	-	-	1
4:30	55	1	91	5	152	-	1	-	1	2	-	-	-
4:45	74	0	95	11	180	1	1	3	-	-	-	-	-
5:00 P.M.	59	3	103	7	172	1	1	1	-	-	-	-	-
5:15	63	2	90	3	158	-	3	1	-	-	-	-	-
5:30	72	0	110	7	189	-	2	-	-	-	-	-	-
5:45 P.M.	69	3	124	2	198	1	2	-	1	-	-	-	-
Totals	720	17	1155	69	1961	6	14	9	3	6	0	0	3

Three Hour Observation Period			Peak Hour 5:00 P.M. to 6:00 P.M.		
Total Volume	1961	veh.	Total Volume	717	veh.
% Trucks	4.4%		% Trucks	3.8%	
% Left Lane	38%		% Left Lane	38%	
% Right Lane	62%		% Right Lane	62%	
Total Conf.	41		Total Conf.	13	
Conflict Rate	20.91	per 1000 veh.	Conflict Rate	18.13	per 1000 veh.

County: Calhoun Highway: SH 35 Date: 6/24/93 Direction of Travel: Southbound

Description: 4-lane divided , 3.0 m (10 ft) shldr. CALHCNAM.WQ1

Limits: 39.6 m (130 ft) North of S.H. 238 to 173.7 m (570 ft) South of S.H. 238 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Cross Traffic		
	PC	HV	PC	HV							From Lt.	From Rt.	
7:00 A.M.	33	2	28	0	63	-	-	-	-	-	-	-	-
7:15	20	1	33	1	55	-	-	-	-	-	-	-	-
7:30	27	1	34	4	66	-	-	-	-	-	-	-	-
7:45	31	1	31	2	65	1	-	1	-	-	-	1	-
8:00 A.M.	20	0	41	3	64	-	-	-	-	-	-	-	-
8:15	18	1	43	1	63	-	-	-	-	1	-	-	-
8:30	18	2	34	5	59	-	-	-	-	-	-	-	-
8:45	15	3	39	2	59	-	-	-	-	-	-	-	-
9:00 A.M.	21	4	41	5	71	-	-	-	-	-	-	-	-
9:15	11	1	33	3	48	-	-	-	-	-	-	-	-
9:30	22	0	41	4	67	-	1	1	1	-	-	-	-
9:45 A.M.	27	1	36	8	72	1	-	-	-	-	-	-	-
Totals	263	17	434	38	752	2	1	2	1	1	0	1	0

Three Hour Observation Period			Peak Hour 9:00 A.M. to 10:00 A.M.		
Total Volume	752	veh.	Total Volume	258	veh.
% Trucks	7.3%		% Trucks	10.1%	
% Left Lane	37%		% Left Lane	34%	
% Right Lane	63%		% Right Lane	66%	
Total Conf.	8		Total Conf.	4	
Conflict Rate	10.64	per 1000 veh.	Conflict Rate	15.50	per 1000 veh.

County: Calhoun Highway: SH 35 Date: 6/24/93 Direction of Travel: Southbound

Description: 4-lane divided , 3.0 m (10 ft) shldr. CALHCNPM.WQ1

Limits: 39.6 m (130 ft) North of S.H. 238 to 173.7 m (570 ft) South of S.H. 238 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Cross Traffic		
	PC	HV	PC	HV							From Lt.	From Rt.	
3:00 P.M.	18	1	35	9	63	-	1	-	-	-	-	-	-
3:15	27	0	39	3	69	-	-	-	-	-	-	-	-
3:30	36	1	49	6	92	-	1	-	-	1	-	-	-
3:45	27	1	53	1	82	1	1	-	-	1	-	-	-
4:00 P.M.	52	1	68	4	125	-	-	1	1	-	-	-	-
4:15	55	0	74	7	136	1	2	1	2	1	-	-	-
4:30	62	1	91	6	160	-	-	-	-	-	-	-	-
4:45	134	0	152	6	292	2	2	2	-	1	-	1	-
5:00 P.M.	109	0	120	3	232	3	5	1	1	1	-	-	-
5:15	84	1	106	2	193	4	3	-	2	3	-	-	-
5:30	106	1	129	4	240	4	3	4	5	2	1	-	-
5:45 P.M.	92	0	102	4	198	2	1	-	1	-	1	-	-
Totals	802	7	1018	55	1882	17	19	9	12	10	2	1	0

Three Hour Observation Period			Peak Hour 4:45 P.M. to 5:45 P.M.		
Total Volume	1882	veh.	Total Volume	957	veh.
% Trucks	3.3%		% Trucks	1.8%	
% Left Lane	43%		% Left Lane	45%	
% Right Lane	57%		% Right Lane	55%	
Total Conf.	70		Total Conf.	50	
Conflict Rate	37.19	per 1000 veh.	Conflict Rate	52.25	per 1000 veh.

County: Lamb   Highway: Loop 430   Date: 8/17/93   Direction of Travel: Eastbound

Description: 4-lane, Undivided, 2.4 m (8 ft) Shld.   LAM2CNAM.WQ1

Limits: From Austin St. to Westside Ave.189 m (620 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	6	0	5	0	11	-	-	-	-	-	-	-	-
7:15	3	0	9	0	12	-	-	-	-	-	-	-	-
7:30	4	0	10	0	14	-	-	-	-	-	-	-	-
7:45	12	0	12	0	24	-	-	-	-	-	-	-	-
8:00 A.M.	8	0	14	0	22	-	-	-	-	-	-	-	-
8:15	5	2	13	1	21	-	-	-	-	-	-	-	-
8:30	11	2	5	1	19	-	-	-	-	-	-	-	-
8:45	8	0	14	1	23	-	-	-	-	-	-	-	-
9:00 A.M.	6	0	8	0	14	-	-	-	-	-	-	-	-
9:15	3	0	10	0	13	-	-	-	-	-	-	-	-
9:30	7	0	11	0	18	-	-	-	-	-	-	-	-
9:45 A.M.	7	0	6	0	13	-	-	-	-	-	-	-	-
Totals	80	4	117	3	204	0	0	0	0	0	0	0	0

Three Hour Observation Period			Peak Hour 7:45 A.M. to 8:45 A.M.		
Total Volume	204	veh.	Total Volume	86	veh.
% Trucks	3.4%		% Trucks	7.0%	
% Left Lane	41%		% Left Lane	47%	
% Right Lane	59%		% Right Lane	53%	
Total Conf.	0		Total Conf.	0	
Conflict Rate	0.00	per 1000 veh.	Conflict Rate	0.00	per 1000 veh.

County: Lamb Highway: Loop 430 Date: 8/17/93 Direction of Travel: Eastbound

Description: 4-lane, Undivided, 2.4 m (8 ft) Shld. LAM2CNPM.WQ1

Limits: From Austin St. to Westside Ave. 189 m (620 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	10	0	8	1	19	-	-	-	-	-	-	-	-
3:15	6	0	5	0	11	-	-	-	-	-	-	-	-
3:30	7	0	6	0	13	-	-	-	-	-	-	-	-
3:45	1	2	10	0	13	-	-	-	-	-	-	-	-
4:00 P.M.	4	2	3	0	9	-	-	-	-	-	-	-	-
4:15	4	0	7	0	11	-	-	-	-	-	-	-	-
4:30	6	0	8	0	14	-	-	-	-	-	-	-	-
4:45	6	1	16	1	24	-	-	-	-	-	-	-	-
5:00 P.M.	6	1	11	0	18	-	-	-	-	-	-	-	-
5:15	2	0	11	1	14	1	-	-	-	-	-	-	-
5:30	1	0	7	0	8	-	-	-	-	-	-	-	-
5:45 P.M.	5	0	8	0	13	-	-	-	-	-	-	-	-
Totals	58	6	100	3	167	1	0	0	0	0	0	0	0

Three Hour Observation Period			Peak Hour 4:30 P.M. to 5:30 P.M.		
Total Volume	167	veh.	Total Volume	70	veh.
% Trucks	5.4%		% Trucks	5.7%	
% Left Lane	38%		% Left Lane	31%	
% Right Lane	62%		% Right Lane	69%	
Total Conf.	1		Total Conf.	1	
Conflict Rate	5.99	per 1000 veh.	Conflict Rate	14.29	per 1000 veh.

County: Mills Highway: U.S. 84 Date: 8/20/93 Direction of Travel: Eastbound

Description: 4-lane, Undivided, 3.0 m (10 ft) Shld. MILLCNAM.WQ1

Limits: From Fina Minit Stop to Wylie Shamrock 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	3	0	8	3	14	1	-	-	-	-	-	-	-
7:15	12	1	16	4	33	-	-	-	-	-	-	-	-
7:30	13	1	31	1	46	-	-	-	-	-	-	-	-
7:45	10	0	45	4	59	-	1	-	-	-	-	-	-
8:00 A.M.	10	1	32	0	43	-	-	1	-	-	-	-	-
8:15	5	0	31	3	39	1	1	1	1	-	-	-	-
8:30	7	1	29	5	42	-	1	-	-	1	-	-	-
8:45	6	1	27	0	34	-	-	1	-	-	-	-	-
9:00 A.M.	12	0	27	2	41	-	-	-	-	-	-	-	-
9:15	12	2	25	5	44	1	-	1	-	-	-	-	-
9:30	7	1	27	3	38	-	-	-	-	-	-	-	-
9:45 A.M.	9	0	31	2	42	1	-	1	-	-	-	-	-
Totals	106	8	329	32	475	4	3	5	1	1	0	0	0

Three Hour Observation Period			Peak Hour 7:30 A.M. to 8:30 A.M.		
Total Volume	475	veh.	Total Volume	187	veh.
% Trucks	8.4%		% Trucks	5.3%	
% Left Lane	24%		% Left Lane	21%	
% Right Lane	76%		% Right Lane	79%	
Total Conf.	14		Total Conf.	6	
Conflict Rate	29.47	per 1000 veh.	Conflict Rate	32.09	per 1000 veh.

County: Mills Highway: U.S. 84 Date: 8/20/93 Direction of Travel: Eastbound

Description: 4-lane, Undivided, 3.0 m (10 ft) Shld. MILLCNPM.WQ1

Limits: From Fina Minit Stop to Wylie Shamrock 213.4 m (700 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	21	1	40	3	65	1	-	-	-	1	-	-	-
3:15	17	2	34	4	57	-	-	-	-	-	-	-	-
3:30	19	1	45	4	69	-	-	2	-	-	-	-	-
3:45	14	1	38	2	55	-	-	-	-	-	-	-	-
4:00 P.M.	16	0	36	5	57	-	1	1	-	-	-	-	1
4:15	10	0	50	2	62	1	-	2	1	-	-	-	-
4:30	11	4	34	5	54	-	-	1	-	-	-	-	-
4:45	17	0	51	1	69	1	-	-	-	1	-	-	2
5:00 P.M.	17	1	42	2	62	-	-	1	-	-	-	-	-
5:15	20	0	32	3	55	2	1	1	1	-	-	-	-
5:30	12	0	34	1	47	-	1	-	-	-	-	-	-
5:45 P.M.	22	0	52	5	79	2	1	2	-	-	-	-	-
Totals	196	10	488	37	731	7	4	10	2	2	0	0	3

Three Hour Observation Period			Peak Hour 4:15 P.M. to 5:15 P.M.		
Total Volume	731	veh.	Total Volume	247	veh.
% Trucks	6.4%		% Trucks	6.1%	
% Left Lane	28%		% Left Lane	24%	
% Right Lane	72%		% Right Lane	76%	
Total Conf.	28		Total Conf.	10	
Conflict Rate	38.30	per 1000 veh.	Conflict Rate	40.49	per 1000 veh.

County: Smith Highway: SH 64 Date: 7/1/93 Direction of Travel: Westbound

Description: 4-lane, TWLTL, 3.0 m (10 ft) shldr. SMI2CNAM.WQ1

Limits: Adam Henry Road to Nu-Way Chevron 237.7 m (780 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	42	0	62	1	105	-	-	-	-	-	-	-	-
7:15	43	0	86	3	132	2	2	-	-	-	-	-	-
7:30	77	0	103	7	187	-	3	1	-	-	-	-	-
7:45	83	0	102	1	186	1	-	1	-	-	-	-	-
8:00 A.M.	53	0	84	6	143	-	-	1	-	-	-	-	-
8:15	43	1	66	6	116	1	2	-	1	-	-	-	-
8:30	56	0	72	5	133	-	2	1	1	-	-	-	-
8:45	54	1	65	3	123	1	1	2	2	-	-	-	-
9:00 A.M.	43	0	57	7	107	-	-	1	-	-	-	-	-
9:15	26	0	47	4	77	1	-	1	-	-	-	-	-
9:30	41	1	49	4	95	-	-	1	-	-	-	-	-
9:45 A.M.	37	1	47	5	90	2	-	1	-	-	-	-	-
Totals	598	4	840	52	1494	8	10	10	4	0	0	0	0

Three Hour Observation Period			Peak Hour 7:15 A.M. to 8:15 A.M.		
Total Volume	1494	veh.	Total Volume	648	veh.
% Trucks	3.7%		% Trucks	2.6%	
% Left Lane	40%		% Left Lane	40%	
% Right Lane	60%		% Right Lane	60%	
Total Conf.	32		Total Conf.	11	
Conflict Rate	21.42	per 1000 veh.	Conflict Rate	16.98	per 1000 veh.

County: Smith Highway: SH 64 Date: 7/1/93 Direction of Travel: Westbound

Description: 4-lane, TWLTL, 3.0 m (10 ft) shldr. SMI2CNPM.WQ1

Limits: Adam Henry Road to Nu-Way Chevron 237.7 m (780 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	29	2	63	0	94	-	-	-	1	-	-	1	-
3:15	37	1	66	1	105	-	1	-	-	-	-	-	-
3:30	39	0	60	5	104	1	-	2	-	1	-	-	-
3:45	27	0	40	3	70	-	-	1	-	-	-	-	-
4:00 P.M.	34	1	55	6	96	-	-	-	-	-	-	-	-
4:15	30	0	47	4	81	-	-	-	-	-	-	-	-
4:30	47	0	52	4	103	-	1	-	-	-	-	-	-
4:45	23	1	47	3	74	-	-	-	-	-	1	-	-
5:00 P.M.	32	1	62	4	99	-	2	-	-	-	-	-	-
5:15	37	1	44	3	85	-	-	-	-	-	-	-	-
5:30	41	3	39	2	85	-	-	-	-	-	-	-	-
5:45 P.M.	36	0	41	0	77	-	-	-	-	-	-	-	-
Totals	412	10	616	35	1073	1	4	3	1	1	1	1	0

Three Hour Observation Period			Peak Hour 3:15 P.M. to 4:15 P.M.		
Total Volume	1073	veh.	Total Volume	375	veh.
% Trucks	4.2%		% Trucks	4.5%	
% Left Lane	39%		% Left Lane	37%	
% Right Lane	61%		% Right Lane	63%	
Total Conf.	12		Total Conf.	6	
Conflict Rate	11.18	per 1000 veh.	Conflict Rate	16.00	per 1000 veh.

County: Victoria Highway: SH 185 Date: 8/31/93 Direction of Travel: Southbound

Description: 4-lane, TWLTL, 2.4 m (8 ft) Shoulder VICTCNAM.WQ1

Limits: From Dudley Street to Canales Fina 161.5 m (530 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	80	0	81	2	163	-	-	1	1	-	-	-	-
7:15	61	1	45	6	113	-	-	3	1	-	-	-	-
7:30	38	3	41	3	85	-	-	-	-	-	-	-	-
7:45	28	1	33	2	64	-	-	-	-	1	-	-	-
8:00 A.M.	36	1	23	3	63	-	-	-	-	-	-	-	-
8:15	34	0	25	6	65	1	-	-	-	-	-	-	-
8:30	23	1	24	5	53	2	-	-	1	-	-	-	-
8:45	18	1	15	7	41	-	-	-	-	-	-	-	-
9:00 A.M.	22	2	19	2	45	-	-	-	-	-	-	-	-
9:15	27	1	16	5	49	1	-	-	-	-	-	-	1
9:30	20	2	11	4	37	-	-	-	-	-	-	-	-
9:45 A.M.	25	8	21	4	58	1	-	-	-	-	-	-	-
Totals	412	21	354	49	836	5	0	4	3	1	0	0	1

Three Hour Observation Period			Peak Hour 7:00 A.M. to 8:00 A.M.		
Total Volume	836	veh.	Total Volume	425	veh.
% Trucks	8.4%		% Trucks	4.2%	
% Left Lane	52%		% Left Lane	50%	
% Right Lane	48%		% Right Lane	50%	
Total Conf.	14		Total Conf.	7	
Conflict Rate	16.75	per 1000 veh.	Conflict Rate	16.47	per 1000 veh.

County: Victoria   Highway: SH 185   Date: 8/31/93   Direction of Travel: Southbound

Description: 4-lane, TWLTL, 2.4 m (8 ft) Shoulder   VICTCNPM.WQ1

Limits: From Dudley Street to Canales Fina 161.5 m (530 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	27	1	22	3	53	-	-	1	-	-	-	1	-
3:15	27	2	30	1	60	2	-	1	-	-	-	-	-
3:30	27	3	25	4	59	-	1	1	-	-	-	-	-
3:45	36	2	32	8	78	1	-	-	-	-	-	-	-
4:00 P.M.	39	3	28	1	71	1	-	1	1	-	-	-	-
4:15	32	1	34	4	71	2	1	-	-	-	1	-	-
4:30	52	2	36	10	100	1	-	1	-	-	-	-	-
4:45	35	3	35	2	75	-	-	-	-	-	-	-	-
5:00 P.M.	48	3	32	5	88	-	-	-	-	-	-	-	-
5:15	58	2	37	3	100	-	-	2	1	-	-	-	-
5:30	45	3	42	2	92	2	-	-	-	-	-	-	-
5:45 P.M.	51	0	33	2	86	1	-	-	-	-	-	-	-
Totals	477	25	386	45	933	10	2	7	2	0	1	1	0

Three Hour Observation Period			Peak Hour 5:00 P.M. to 6:00 P.M.		
Total Volume	933	veh.	Total Volume	366	veh.
% Trucks	7.5%		% Trucks	5.5%	
% Left Lane	54%		% Left Lane	57%	
% Right Lane	46%		% Right Lane	43%	
Total Conf.	23		Total Conf.	6	
Conflict Rate	24.65	per 1000 veh.	Conflict Rate	16.39	per 1000 veh.

County: Williamson Highway: U.S. 79 Date: 6/15/93 Direction of Travel: S.W. Bound

Description: 4-lane Undivided, 3.0 m (10 ft) shld., No WILLCNAM.WQ1

Limits: Beginning of School Zone on N.E. to Main Street 3.04 m (1000 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
7:00 A.M.	18	0	50	0	68	-	-	1	-	-	-	-	-
7:15	16	0	27	0	43	-	-	-	-	1	-	-	-
7:30	12	0	35	0	47	-	1	-	-	-	-	-	-
7:45	10	0	37	0	47	-	-	-	-	-	-	-	1
8:00 A.M.	10	0	28	0	38	-	1	-	-	-	-	-	-
8:15	10	0	34	0	44	-	1	-	-	-	-	-	-
8:30	14	0	41	0	55	-	-	-	-	-	-	-	-
8:45	18	0	26	0	44	-	-	1	-	-	-	-	-
9:00 A.M.	13	0	26	0	39	-	-	-	-	-	-	-	1
9:15	15	0	30	0	45	-	1	-	-	-	-	1	-
9:30	14	0	39	0	53	-	-	-	-	-	-	-	-
9:45 A.M.	5	0	20	0	25	-	-	-	-	-	-	-	-
Totals	155	0	393	0	548	0	4	2	0	1	0	1	2

Three Hour Observation Period			Peak Hour 7:00 A.M. to 8:00 A.M.		
Total Volume	548	veh.	Total Volume	205	veh.
% Trucks	0.0%		% Trucks	0.0%	
% Left Lane	28%		% Left Lane	27%	
% Right Lane	72%		% Right Lane	73%	
Total Conf.	8		Total Conf.	3	
Conflict Rate	14.60	per 1000 veh.	Conflict Rate	14.63	per 1000 veh.

County: Williamson Highway: U.S. 79 Date: 6/15/93 Direction of Travel: S.W. Bound

Description: 4-lane Undivided, 3.0 m (10 ft) shld., No WILLCNPM.WQ1

Limits: Beginning of School Zone on N.E. to Main Street 304.8 m (1000 ft)

Time Count Started	Volume					CONFLICTS							
	Left		Right		Total	Rt. Turn, Exit	Rt. Turn, Enter	Slow Vehicle	Secondary	Lt. Turn from Lt.	Lt. Turn Opposing	Lt. Turn from Rt.	Lt. Turn same direct
	PC	HV	PC	HV									
3:00 P.M.	19	1	36	2	58	-	-	1	-	3	-	-	-
3:15	18	1	24	4	47	1	-	-	-	-	-	-	-
3:30	11	3	27	3	44	-	-	-	-	-	1	-	-
3:45	12	1	43	1	57	-	-	1	1	-	-	-	-
4:00 P.M.	17	1	39	3	60	-	-	-	-	-	1	1	-
4:15	13	6	54	3	76	-	-	-	-	-	-	-	-
4:30	23	0	55	0	78	1	-	1	-	-	-	-	-
4:45	17	3	53	3	76	-	-	-	-	1	-	-	-
5:00 P.M.	15	0	45	3	63	-	-	-	-	-	-	-	1
5:15	12	0	35	1	48	2	-	-	-	-	-	-	1
5:30	16	1	42	7	66	-	-	-	-	-	-	-	-
5:45 P.M.	18	2	31	7	58	1	-	-	-	-	-	-	-
Totals	191	19	484	37	731	5	0	3	1	4	2	1	2

Three Hour Observation Period			Peak Hour 4:15 P.M. to 5:15 P.M.		
Total Volume	731	veh.	Total Volume	293	veh.
% Trucks	7.7%		% Trucks	6.1%	
% Left Lane	29%		% Left Lane	26%	
% Right Lane	71%		% Right Lane	74%	
Total Conf.	18		Total Conf.	4	
Conflict Rate	24.62	per 1000 veh.	Conflict Rate	13.65	per 1000 veh.

REGRESSION ANALYSIS WORKSHEETS

Regression Analysis (No Paved Shoulder)

Location	Peak Volume	Conflicts Observed	Conflicts Computed
Lamb (84) A.M.	184	5.43	9.81
Lamb (84) P.M.	211	9.48	11.25
Tom Green A.M.	284	14.08	15.14
Rusk A.M.	315	41.27	16.79
Nueces A.M.	338	23.67	18.02
Comal A.M.	356	5.62	18.97
San Patricio A.M.	388	7.73	20.68
Smith (155) A.M.	402	22.39	21.43
San Patricio P.M.	447	17.90	23.83
Henderson P.M.	480	25.00	25.58
Tom Green P.M.	485	16.49	25.85
Rusk P.M.	587	52.81	31.29
Comal P.M.	676	7.40	36.03
Gregg A.M.	788	50.76	42.00
Nueces P.M.	810	69.14	43.17
Smith (155) P.M.	873	34.36	46.53
Henderson A.M.	897	33.44	47.81
Gregg P.M.	949	62.17	50.58
	Average Rate	27.73	
	Standard Dev.	20.17	
	Variance	406.73	

Regression Output:

Constant		0.0000
Std Err of Y Est		14.6481
R Squared		0.4725
No. of Observations		18
Degrees of Freedom		17
X Coefficient(s)	0.0533	
Std Err of Coef.	0.0060	

Regression Analysis (No Paved Shoulder)

Location	Peak Volume	Conflicts Observed	Conflicts Computed
Lamb (84) A.M.	184	5.43	8.53
Lamb (84) P.M.	211	9.48	10.04
Tom Green A.M.	284	14.08	14.14
Rusk A.M.	315	41.27	15.88
Nueces A.M.	338	23.67	17.17
Comal A.M.	356	5.62	18.18
San Patricio A.M.	388	7.73	19.97
Smith (155) A.M.	402	22.39	20.76
San Patricio P.M.	447	17.90	23.28
Henderson P.M.	480	25.00	25.13
Tom Green P.M.	485	16.49	25.41
Rusk P.M.	587	52.81	31.14
Comal P.M.	676	7.40	36.13
Gregg A.M.	788	50.76	42.41
Nueces P.M.	810	69.14	43.65
Smith (155) P.M.	873	34.36	47.18
Henderson A.M.	897	33.44	48.53
Gregg P.M.	949	62.17	51.44
	Average Rate	27.73	
	Standard Dev.	20.17	
	Variance	406.73	

Regression Output:

Constant		-1.7949
Std Err of Y Est		15.0781
R Squared		0.4739
No. of Observations		18
Degrees of Freedom		16
X Coefficient(s)	0.0561	
Std Err of Coef.	0.0148	

Regression Analysis (Paved Shoulder)

Location	Peak Volume	Conflicts Observed	Conflicts Computed
Lamb (430) P.M.	70	14.29	13.14
Lamb (430) A.M.	86	0.00	13.48
Mills A.M.	187	32.09	15.59
Williamson A.M.	205	14.63	15.96
Mills P.M.	247	40.49	16.84
Calhoun A.M.	258	15.50	17.07
Williamson P.M.	293	13.65	17.80
Victoria P.M.	366	16.39	19.33
Smith (64) P.M.	375	16.00	19.52
Victoria A.M.	425	16.47	20.56
Smith (64) A.M.	648	16.98	25.22
Bastrop A.M.	716	12.57	26.64
Bastrop P.M.	717	18.13	26.66
Calhoun P.M.	957	52.25	31.68
	Average Rate	19.96	
	Standard Dev.	13.12	
	Variance	172.11	

Regression Output:

Constant		11.6777
Std Err of Y Est		12.3683
R Squared		0.1795
No. of Observations		14
Degrees of Freedom		12
X Coefficient(s)	0.0209	
Std Err of Coef.	0.0129	

DATA SHEETS FOR SPEED STUDIES

County: Bastrop Highway: S.H. 21
Date: 6-16-93 Direction of Travel: W.B.
Limits: Smith Street to Eskew Street 225.6 m (740 ft)
Start Time: 10:08 A.M. End Time: 10:25 A.M.
Posted Speed 50 BASTSPAM.WQ1

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	42		31	49	
2	49	Y	32	53	
3	45		33	54	
4	49		34	42	Y
5	55		35	53	
6	49		36	47	
7	50		37	51	
8	53		38	51	
9	54		39	53	
10	46		40	51	
11	54		41	46	
12	44	Y	42	54	
13	42	Y	43	56	
14	53		44	48	Y
15	58		45	47	Y
16	54		46	57	
17	52		47	56	
18	52		48	52	
19	55		49	51	
20	50		50	44	Y
21	52		51	54	
22	44		52	47	
23	59		53	50	Y
24	66		54	53	
25	60		55	53	
26	57		56	52	
27	52		57	44	
28	52		58	52	
29	54		59	52	
30	53		60	51	Y

Mean Speed: 51.30 P.C. Mean Speed: 52.18
Std. Dev. 4.64 Std. Dev. 4.28
85th Percentile Speed: 55

Note: 1 mph = 1.6 km/h

County: Bastrop Highway: S.H. 21
Date: 6-16-93 Direction of Travel: W.B.
Limits: Smith Street to Eskew Street 225.6 m (740 ft)
Start Time: 2:10 P.M. End Time: 2:25 P.M.
Posted Speed 50 BASTSPPM.WQ1

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	52		31	58	
2	45	Y	32	49	
3	49	Y	33	50	
4	51		34	56	
5	54		35	48	
6	48		36	44	Y
7	42	Y	37	57	
8	38		38	43	
9	54		39	42	
10	47		40	54	Y
11	51		41	45	
12	54		42	50	
13	46		43	52	
14	51		44	58	
15	45		45	51	
16	55		46	50	Y
17	45		47	53	
18	46	Y	48	49	Y
19	50		49	57	
20	42		50	58	
21	36		51	55	
22	52		52	53	
23	58		53	58	
24	49		54	48	
25	46		55	49	
26	48		56	62	
27	58		57	50	
28	54		58	42	Y
29	53		59	49	
30	55	Y	60	55	

Mean Speed: 50.32 P.C. Mean Speed: 50.86
Std. Dev. 5.43 Std. Dev. 5.46
85th Percentile Speed: 56

Note: 1mph = 1.6 km/h

County:	Calhoun	Highway:	SH 35
Date:	6-24-93	Direction of Travel:	SB
Limits:	39.6 m (130 ft) N of SH 238 to 173.7 m (570 ft) S of SH 238 213.4 m (700 ft)		
Start Time:	10:06 A.M.	End Time:	10:30 A.M.
Posted Speed	50	CALHSPAM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	44		31	46	
2	41		32	44	
3	45		33	49	Y
4	40		34	51	
5	44		35	49	
6	50		36	50	
7	51	Y	37	50	Y
8	53		38	46	
9	48	Y	39	46	Y
10	34		40	42	
11	50		41	50	
12	52		42	43	
13	48		43	44	
14	42		44	53	
15	43		45	50	
16	51		46	40	
17	48		47	50	
18	40		48	47	
19	42		49	46	
20	46		50	45	
21	47		51	51	Y
22	50		52	53	
23	41		53	49	
24	48		54	45	
25	49		55	53	
26	49		56	50	
27	52		57	48	Y
28	49		58	45	
29	53		59	53	
30	48		60	49	

Mean Speed:	47.25	P.C. Mean Speed:	47.02
Std. Dev.	4.10	Std. Dev.	4.27
85th Percentile Speed:	51		

Note: 1 mph = 1.6 km/h

County:	Calhoun	Highway:	SH 35
Date:	6-24-93	Direction of Travel:	SB
Limits:	39.6 m (130 ft) N of SH 238 to 173.7 m (570 ft) S of SH 238 213.4 m (700 ft)		
Start Time:	2:15 P.M.	End Time:	2:40 P.M.
Posted Speed	50	CALHSPPM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	47	Y	31	47	Y
2	45		32	47	
3	52		33	55	Y
4	45		34	44	
5	50		35	46	
6	45		36	40	
7	45	Y	37	46	Y
8	45		38	55	
9	50		39	54	
10	47		40	48	
11	47		41	54	
12	39		42	47	
13	45		43	44	
14	48		44	47	
15	44	Y	45	48	
16	45		46	59	
17	46	Y	47	52	
18	49	Y	48	46	
19	40		49	44	
20	47		50	43	
21	50		51	49	
22	44		52	50	
23	45		53	46	
24	47	Y	54	38	
25	50		55	48	
26	58		56	51	
27	42		57	50	
28	44		58	51	
29	44		59	55	
30	44		60	50	Y

Mean Speed:	47.38	P.C. Mean Speed:	47.34
Std. Dev.	4.31	Std. Dev.	4.53
85th Percentile Speed:	51		

Note: 1 mph = 1.6 km/h

County:	Lamb	Highway:	Loop 430
Date:	8-17-93	Direction of Travel:	E.B.
Limits:	From Austin St. to Westside Ave. 189 m (620 ft)		
Start Time:	10:05 A.M.	End Time:	11:05 A.M.
Posted Speed	50	LAM2SPAM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	35		31	37	
2	57		32	47	
3	40		33	44	
4	47		34	47	
5	43		35	39	
6	41		36	40	
7	45		37	49	
8	42		38	47	
9	52		39	58	
10	45		40	46	
11	45		41	44	
12	55		42	46	
13	40		43	47	
14	34		44	47	
15	31		45	53	
16	43		46	40	
17	50		47	51	
18	50		48		
19	44		49		
20	48		50		
21	40		51		
22	44		52		
23	36		53		
24	55		54		
25	51		55		
26	50		56		
27	45	Y	57		
28	40		58		
29	40		59		
30	56		60		

Mean Speed:	45.23	P.C. Mean Speed:	45.24
Std. Dev.	6.17	Std. Dev.	6.24
85th Percentile Speed:	51		

Note: 1 mph = 1.6 km/h

County: Lamb Highway: Loop 430
Date: 8-17-93 Direction of Travel: E.B.
Limits: From Austin St. to Westside Ave. 289 m (620 ft)
Start Time: 1:40 P.M. End Time: 2:36 P.M.
Posted Speed 50 LAM2SPPM.WQ1

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	46		31	45	
2	59		32	39	
3	54	Y	33	46	
4	42		34	46	
5	41		35	46	Y
6	49		36	49	
7	40		37	49	
8	34		38	38	
9	43		39	49	
10	53		40	53	
11	47		41	38	
12	38		42	42	
13	52		43	50	
14	43		44	46	
15	49		45	42	
16	41		46	44	
17	45		47	48	
18	45		48	47	
19	56		49	37	
20	46		50	57	
21	52		51	48	
22	52		52	46	
23	39		53	58	
24	46		54	60	
25	54		55	43	
26	46		56	56	
27	38		57	45	
28	50		58	38	
29	37	Y	59	42	
30	42		60	54	

Mean Speed: 46.33 P.C. Mean Speed: 46.37
Std. Dev. 6.15 Std. Dev. 6.11
85th Percentile Speed: 53

Note: 1 mph = 1.6 km/h

County:	Mills	Highway:	U.S. 84
Date:	8-20-93	Direction of Travel:	E.B.
Limits:	From Fina Minit Stop to Wylie Shamrock		
	213.4 m (700 ft)		
Start Time:	10:06 A.M.	End Time:	10:33 A.M.
Posted Speed	55	MILLSPAM.WQ1	

Vehicle	Speed (mph)		Vehicle	Speed (mph)	
Number	PC	HV	Number	PC	HV
1	38		31	51	
2	51		32	47	
3	51		33	50	
4	50	Y	34	50	
5	40		35	59	
6	45		36	53	
7	56		37	50	
8	51		38	50	
9	54		39	54	
10	56		40	45	
11	48		41	55	
12	50		42	62	
13	50		43	52	
14	47		44	54	
15	53		45	45	Y
16	38		46	48	
17	43		47	49	
18	57		48	51	
19	46		49	42	
20	54		50	58	
21	56		51	39	
22	51		52	49	
23	46		53	50	
24	44		54	53	
25	50		55	55	
26	49		56	53	Y
27	51		57	51	
28	57		58	52	
29	33		59	51	
30	54		60	46	

Mean Speed:	49.88	P.C. Mean Speed:	49.91
Std. Dev.	5.49	Std. Dev.	5.58
85th Percentile Speed:	55		

Note: 1 mph = 1.6 km/h

County:	Mills	Highway:	U.S. 84
Date:	8-20-93	Direction of Travel:	E.B.
Limits:	From Fina Minit Stop to Wylie Shamrock		
	213.4 m (700 ft)		
Start Time:	2:00 P.M.	End Time:	2:24 P.M.
Posted Speed	55	MILLSPPM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	53		31	54	
2	56		32	48	
3	36		33	53	
4	61		34	60	
5	51		35	59	
6	60		36	55	
7	46		37	51	
8	61		38	56	
9	49		39	52	
10	56		40	59	
11	49	Y	41	52	
12	55		42	49	Y
13	47		43	53	
14	52		44	47	
15	59		45	61	
16	49		46	48	
17	43		47	56	
18	48		48	54	
19	55		49	55	
20	54		50	52	
21	51		51	48	
22	56		52	43	
23	46	Y	53	52	
24	53	Y	54	40	
25	59		55	41	
26	46		56	55	
27	62	Y	57	50	Y
28	47		58	52	
29	51		59	49	
30	51		60	48	

Mean Speed:	51.90	P.C. Mean Speed:	51.94
Std. Dev.	5.53	Std. Dev.	5.58
85th Percentile Speed:	56		

Note: 1 mph = 1.6 km/h

County: Smith Highway: SH 64
Date: 7-1-93 Direction of Travel: WB
Limits: Adam Henry Rd. to NuWay Chevron 237.7 m (780 ft)
Start Time: 10:04 A.M. End Time: 10:24 A.M.
Posted Speed: 55 SMI2SPAM.WQ1

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	57		31	62	
2	44		32	59	
3	52		33	60	
4	55		34	52	
5	49		35	51	
6	53		36	62	
7	56		37	54	
8	58		38	51	
9	54		39	46	
10	63		40	51	
11	56		41	52	
12	54		42	48	Y
13	53		43	52	
14	60		44	53	
15	55		45	55	
16	58		46	53	
17	41		47	58	
18	65		48	57	
19	55		49	61	
20	49		50	56	
21	54		51	53	Y
22	60		52	53	
23	53		53	57	
24	51		54	49	
25	44		55	59	Y
26	48		56	54	
27	47		57	53	
28	48		58	61	
29	56		59	60	
30	45		60	56	Y

Mean Speed: 54.02 P.C. Mean Speed: 54.02
Std. Dev. 5.09 Std. Dev. 5.15
85th Percentile Speed: 60

Note: 1 mph = 1.6 km/h

County: Smith Highway: SH 64
Date: 7-1-93 Direction of Travel: WB
Limits: Adam Henry Rd. to NuWay Chevron 237.7 m (780 ft)
Start Time: 2:27 P.M. End Time: 2:45 P.M.
Posted Speed: 55 SMI2SPPM.WQ1

Vehicle	Speed (mph)		Vehicle	Speed (mph)	
Number	PC	HV	Number	PC	HV
1	56		31	54	
2	64		32	56	Y
3	50		33	52	
4	48		34	54	
5	55		35	60	
6	63		36	56	
7	55		37	57	
8	49		38	54	
9	55		39	56	
10	63		40	52	
11	62		41	55	
12	60		42	54	
13	58		43	48	
14	56		44	55	
15	60		45	49	
16	59		46	51	
17	55		47	52	
18	49		48	54	
19	50		49	52	
20	58		50	50	Y
21	59		51	54	
22	51		52	56	
23	57		53	59	
24	54		54	55	
25	49		55	50	
26	64		56	57	
27	62		57	54	
28	54		58	53	
29	50		59	52	
30	49	Y	60	61	

Mean Speed: 54.93 P.C. Mean Speed: 55.57
Std. Dev. 4.27 Std. Dev. 4.25
85th Percentile Speed: 60

Note: 1 mph = 1.6 km/h

County: Victoria Highway: S.H. 185
Date: 8-31-93 Direction of Travel: S.B.
Limits: From Dudley Street to Canales Fina 161.5 m (530 ft)
Start Time: 10:03 A.M. End Time: 10:31 A.M.
Posted Speed 55 VICTSPAM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	45		31	43	
2	49		32	48	Y
3	46	Y	33	45	
4	50		34	41	
5	41		35	44	
6	51		36	53	
7	48		37	54	
8	44		38	51	
9	45		39	57	
10	47		40	54	
11	47	Y	41	54	
12	55		42	47	
13	43		43	47	
14	46	Y	44	51	
15	53		45	49	
16	54		46	44	
17	50	Y	47	46	Y
18	49		48	49	
19	45		49	44	
20	57		50	47	
21	40		51	49	
22	49		52	50	
23	47		53	52	Y
24	46		54	50	
25	43		55	48	
26	52		56	47	
27	45		57	59	
28	45		58	55	Y
29	47		59	56	
30	39		60	45	

Mean Speed: 48.28 P.C. Mean Speed: 48.21
Std. Dev. 4.48 Std. Dev. 4.65
85th Percentile Speed: 53

Note: 1 mph = 1.6 km/h

County:	Victoria	Highway:	S.H. 185
Date:	8-31-93	Direction of Travel:	S.B.
Limits:	From Dudley Street to Canales Fina 161.5 m (530 ft)		
Start Time:	2:12 P.M.	End Time:	2:36 P.M.
Posted Speed	55	VICTSPPM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	51		31	40	
2	42		32	51	
3	52		33	51	
4	50		34	56	
5	46		35	42	
6	53		36	59	Y
7	43	Y	37	51	
8	42		38	53	
9	47	Y	39	43	
10	53		40	44	
11	55		41	41	
12	59		42	40	
13	47		43	53	
14	48		44	49	
15	50		45	62	
16	48		46	53	
17	58		47	53	
18	52		48	48	
19	55	Y	49	50	
20	40		50	53	
21	46		51	67	
22	52		52	43	
23	48		53	50	
24	48		54	46	
25	48		55	41	
26	38		56	45	
27	55		57	48	Y
28	44		58	54	
29	45		59	47	
30	53		60	56	

Mean Speed:	49.28	P.C. Mean Speed:	49.18
Std. Dev.	5.89	Std. Dev.	5.89
85th Percentile Speed:	55		

Note: 1 mph = 1.6 km/h

County: Williamson Highway: US 79
Date: 6-15-93 Direction of Travel: S.W.
Limits: Beginning of School zone on NE to Main St.
304.8 m (1000 ft.)
Start Time: 10:05 A.M. End Time: 10:36 A.M.
Posted Speed 50 WILLSPAM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	51		31	39	
2	42		32	41	
3	44		33	49	
4	45		34	32	
5	55		35	45	
6	56		36	50	
7	58		37	45	
8	48		38	44	
9	50		39	51	
10	40		40	52	
11	45	Y	41	46	Y
12	54		42	55	
13	54		43	45	
14	53		44	46	
15	48		45	49	
16	45		46	47	
17	34		47	46	
18	49		48	48	
19	48	Y	49	46	
20	55		50	49	
21	48	Y	51	45	
22	35		52	51	
23	51		53	47	
24	47		54	46	
25	49		55	45	
26	38		56	48	
27	42	Y	57	41	
28	44	Y	58	44	
29	50		59	45	
30	40		60	45	

Mean Speed: 46.67 P.C. Mean Speed: 46.80
Std. Dev. 5.28 Std. Dev. 5.51
85th Percentile Speed: 51

Note: 1 mph = 1.6 km/h

County:	Williamson	Highway:	US 79
Date:	6-15-93	Direction of Travel:	S.W.
Limits:	Beginning of School zone on NE to Main St. 304.8 m (1000 ft.)		
Start Time:	1:25 P.M.	End Time:	1:55 P.M.
Posted Speed	50	WILLSPPM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	49		31	50	
2	45		32	41	Y
3	47		33	47	
4	51		34	48	Y
5	37		35	48	Y
6	45		36	50	
7	40		37	45	
8	44		38	43	
9	40		39	40	Y
10	48	Y	40	48	
11	48		41	43	
12	47		42	46	
13	46		43	38	Y
14	43		44	37	
15	46		45	41	
16	39		46	49	
17	44	Y	47	42	
18	42		48	45	
19	43	Y	49	45	
20	47		50	47	
21	50		51	43	
22	40		52	55	
23	47		53	43	Y
24	50		54	42	
25	46		55	35	
26	47		56	43	
27	39		57	48	
28	63		58	45	
29	50	Y	59	50	
30	46		60	50	

Mean Speed:	45.27	P.C. Mean Speed:	45.46
Std. Dev.	4.67	Std. Dev.	4.80
85th Percentile Speed:	50		

Note: 1 mph = 1.6 km/h

County: Comal Highway: S.H. 46
Date: 6-17-93 Direction of Travel: E.B.
Limits: 121.9 m (400 ft) West of Oelkers St. to 91.4 m (300ft) East of Oelkers Street
Start Time: 10:05 A.M. End Time: 10:29 A.M.
Posted Speed 55 COMASPAM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	55		31	49	
2	51		32	50	Y
3	54		33	51	
4	54		34	49	
5	52	Y	35	60	
6	51		36	51	
7	45	Y	37	50	
8	50		38	57	
9	52		39	45	Y
10	53		40	52	
11	52		41	49	
12	46		42	45	
13	53		43	46	
14	53		44	49	Y
15	60		45	62	
16	48		46	50	
17	40		47	44	
18	48		48	49	
19	55		49	48	
20	45		50	52	
21	45		51	54	
22	42		52	51	
23	50		53	48	
24	55		54	46	
25	52		55	52	
26	48		56	48	
27	52		57	50	
28	51		58	47	
29	44	Y	59	47	Y
30	54		60	51	

Mean Speed: 50.20 P.C. Mean Speed: 50.57
Std. Dev. 4.22 Std. Dev. 4.24
85th Percentile Speed: 54

Note: 1 mph = 1.6 km/h

County: Comal Highway: S.H. 46
Date: 6-17-93 Direction of Travel: E.B.
Limits: 121.9 m (400 ft) West of Oelkers St. to 91.4 m (300 ft) East of Oelkers Street
Start Time: 2:05 P.M. End Time: 2:31 P.M.
Posted Speed 55 COMASPPM.WQ1

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	54		31	53	
2	50		32	50	
3	46		33	52	
4	54		34	61	
5	57		35	45	
6	52		36	53	
7	50		37	51	
8	52		38	41	
9	64		39	55	
10	59		40	59	
11	47		41	45	Y
12	51		42	42	
13	53		43	59	
14	53		44	50	
15	52		45	58	
16	55		46	52	
17	56		47	49	
18	52		48	48	
19	50		49	55	
20	58		50	54	
21	46		51	48	
22	52		52	49	
23	54		53	57	
24	45	Y	54	52	
25	63		55	57	
26	53		56	59	
27	48		57	54	
28	36	Y	58	47	
29	40		59	62	
30	46		60	58	

Mean Speed: 52.05 P.C. Mean Speed: 52.58
Std. Dev. 5.69 Std. Dev. 5.23
85th Percentile Speed: 58

Note: 1 mph = 1.6 km/h

County:	Gregg	Highway:	Loop 281
Date:	8-25-93	Direction of Travel:	N.B.
Limits:	From McDonald's to Gilmour Street 256 m (840 ft)		
Start Time:	10:07 A.M.	End Time:	10:31 A.M.
Posted Speed	50	GREGSPAM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	51		31	42	Y
2	53		32	45	
3	38	Y	33	51	
4	51		34	46	
5	47		35	45	
6	59		36	47	
7	44		37	48	
8	52		38	41	Y
9	46		39	48	
10	50		40	47	
11	52		41	45	
12	40		42	53	
13	52		43	46	
14	46		44	54	
15	48		45	49	
16	41		46	45	
17	52		47	47	
18	44		48	49	
19	47		49	50	
20	50		50	43	
21	44		51	44	
22	43		52	42	
23	45	Y	53	56	
24	44		54	53	
25	33	Y	55	49	
26	55		56	38	
27	43		57	45	
28	50		58	49	
29	43		59	45	Y
30	50		60	54	

Mean Speed:	47.15	P.C. Mean Speed:	47.87
Std. Dev.	4.83	Std. Dev.	4.32
85th Percentile Speed:	52		

Note: 1 mph = 1.6 km/h

County: Gregg Highway: Loop 281
Date: 8-25-93 Direction of Travel: N.B.
Limits: From McDonald's to Gilmour Street 256 m (840 ft)
Start Time: 2:20 P.M. End Time: 2:43 P.M.
Posted Speed 50 GREGSPPM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	43		31	60	
2	49		32	43	
3	44		33	48	
4	60		34	42	
5	54		35	42	Y
6	44	Y	36	45	
7	39	Y	37	52	
8	42		38	45	
9	48		39	52	
10	45		40	57	
11	48		41	56	
12	56		42	54	
13	46		43	48	
14	45		44	42	
15	52		45	51	Y
16	44		46	49	
17	49		47	45	
18	47		48	42	
19	48		49	52	
20	38		50	43	Y
21	57		51	45	
22	42		52	53	
23	45		53	50	
24	46		54	40	
25	41		55	45	
26	45		56	56	
27	52		57	40	Y
28	46		58	46	
29	42		59	58	
30	55		60	48	

Mean Speed: 47.68 P.C. Mean Speed: 48.19
Std. Dev. 5.54 Std. Dev. 5.46
85th Percentile Speed: 54

Note: 1 mph = 1.6 km/h

County:	Henderson	Highway:	S.H. 31
Date:	8-24-93	Direction of Travel:	E.B.
Limits:	From Kidd Jones Shamrock to Rippy's Citgo		
	213.4 m (700 ft)		
Start Time:	10:05 A.M.	End Time:	10:21 A.M.
Posted Speed	55	HENDSPAM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	45		31	49	
2	48		32	53	
3	43		33	49	Y
4	67		34	50	
5	43		35	63	
6	49		36	47	
7	57		37	38	
8	54		38	48	
9	40		39	55	
10	45		40	56	
11	49		41	53	
12	46		42	44	
13	54		43	56	
14	59		44	66	
15	54		45	50	
16	51	Y	46	53	
17	55		47	51	
18	49		48	40	
19	57		49	57	
20	54		50	40	
21	51		51	60	
22	53		52	48	
23	46		53	55	
24	50		54	54	
25	55		55	57	
26	58		56	50	
27	49		57	55	
28	49		58	53	
29	53		59	51	
30	50		60	53	

Mean Speed:	51.45	P.C. Mean Speed:	51.50
Std. Dev.	5.91	Std. Dev.	6.00
85th Percentile Speed:	57		

Note: 1 mph = 1.6 km/h

County:	Henderson	Highway:	S.H. 31
Date:	8-24-93	Direction of Travel:	E.B.
Limits:	From Kidd Jones Shamrock to Rippy's Citgo		
	213.4 m (700 ft)		
Start Time:	2:28 P.M.	End Time:	2:47 P.M.
Posted Speed	55	HENDSPPM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	44	Y	31	50	
2	43		32	50	
3	44		33	54	
4	51		34	50	
5	56		35	50	
6	44		36	53	
7	56		37	51	
8	57		38	44	Y
9	48		39	49	
10	52		40	47	
11	54		41	56	
12	44		42	49	
13	55		43	54	
14	52		44	49	
15	51	Y	45	51	Y
16	56		46	44	
17	40		47	44	
18	48		48	54	
19	64	Y	49	51	
20	58		50	52	
21	45		51	49	
22	51		52	50	Y
23	54		53	53	
24	48	Y	54	58	
25	53		55	52	
26	55		56	51	
27	43		57	47	
28	48		58	50	Y
29	49		59	53	
30	56		60	52	

Mean Speed:	50.60	P.C. Mean Speed:	50.81
Std. Dev.	4.58	Std. Dev.	4.35
85th Percentile Speed:	55		

Note: 1 mph = 1.6 km/h

County:	Lamb	Highway:	U.S. 84
Date:	8-16-93	Direction of Travel:	E.B.
Limits:	76.2 m (250 ft) E. of Wilson St. to Austin St.		
	243.8 m (800 ft)		
Start Time:	10:06 A.M.	End Time:	10:32 A.M.
Posted Speed	50	LAM1SPAM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	48		31	48	
2	47		32	47	
3	51	Y	33	38	
4	43	Y	34	45	
5	51		35	46	
6	47		36	50	
7	51		37	54	
8	51		38	47	
9	47		39	54	
10	50		40	45	
11	49		41	47	
12	54		42	43	
13	50		43	47	
14	46		44	53	
15	48		45	57	
16	47		46	52	Y
17	39		47	46	
18	46		48	46	
19	49		49	49	Y
20	52		50	41	
21	50		51	53	
22	46		52	43	
23	47		53	36	
24	48		54	46	
25	54		55	42	
26	47		56	51	
27	53		57	41	
28	50		58	54	
29	44	Y	59	44	
30	48		60	51	

Mean Speed:	47.82	P.C. Mean Speed:	47.82
Std. Dev.	4.25	Std. Dev.	4.30
85th Percentile Speed:	52		

Note: 1 mph = 1.6 km/h

County:	Lamb	Highway:	U.S. 84
Date:	8-16-93	Direction of Travel:	E.B.
Limits:	76.2 m (250 ft) E. of Wilson St. to Austin St.		
	243.8 m (800 ft)		
Start Time:	2:07 P.M.	End Time:	2:32 P.M.
Posted Speed	50	LAM1SPPM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	47		31	48	
2	55		32	45	
3	49		33	49	
4	47		34	39	
5	42		35	52	
6	46		36	41	Y
7	50		37	40	
8	44		38	47	
9	48		39	46	Y
10	49		40	46	
11	52		41	50	Y
12	46	Y	42	49	
13	42		43	40	
14	48		44	54	
15	49	Y	45	55	
16	49		46	55	
17	50		47	42	Y
18	51		48	50	
19	45		49	46	
20	49		50	53	
21	41		51	42	
22	51		52	53	
23	51	Y	53	58	
24	53		54	53	Y
25	56		55	49	
26	49		56	40	Y
27	44		57	52	
28	46		58	46	
29	53		59	41	
30	49		60	48	

Mean Speed:	48.00	P.C. Mean Speed:	48.27
Std. Dev.	4.54	Std. Dev.	4.51
85th Percentile Speed:	53		

Note: 1 mph = 1.6 km/h

County:	Nueces	Highway:	F.M. 2444
Date:	8-30-93	Direction of Travel:	W.B.
Limits:	Potter's Mill Apts. First Three Drives from the East 170.7 m (560 ft)		
Start Time:	10:05 A.M.	End Time:	10:25 A.M.
Posted Speed	50	NUECSPAM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	45		31	46	
2	51		32	56	
3	43		33	56	
4	50		34	54	
5	62		35	41	
6	42		36	37	
7	48		37	50	
8	43		38	36	
9	49		39	44	
10	47		40	42	
11	43	Y	41	43	
12	47		42	50	
13	41		43	47	
14	46		44	43	
15	46		45	50	
16	47		46	49	
17	53		47	42	
18	52		48	53	
19	56		49	49	
20	43		50	36	
21	52		51	40	
22	43		52	49	
23	46		53	45	
24	62		54	47	
25	46		55	46	
26	46		56	36	
27	53		57	46	
28	44		58	50	
29	46		59	43	
30	47		60	47	

Mean Speed:	46.87	P.C. Mean Speed:	46.93
Std. Dev.	5.52	Std. Dev.	5.55
85th Percentile Speed:	52		

Note: 1 mph = 1.6 km/h

County:	Nueces	Highway:	F.M. 2444
Date:	8-30-93	Direction of Travel:	W.B.
Limits:	Potter's Mill Apts. First Three Drives from the East 170.7 m (560 ft)		
Start Time:	2:37 P.M.	End Time:	2:55 P.M.
Posted Speed	50	NUECSPPM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	44		31	42	
2	45		32	45	
3	45		33	43	
4	54		34	40	
5	48		35	43	
6	43		36	55	
7	51		37	44	
8	55		38	50	
9	42		39	48	
10	51		40	47	
11	56		41	45	
12	57		42	45	Y
13	38		43	42	
14	42		44	47	
15	44		45	47	
16	42		46	42	
17	47		47	45	
18	46		48	46	
19	39		49	52	
20	42		50	42	
21	49		51	42	
22	45		52	52	
23	47		53	42	
24	52		54	55	
25	44		55	52	
26	47		56	55	
27	50		57	48	
28	51		58	42	
29	52		59	53	
30	47		60	50	

Mean Speed:	46.93	P.C. Mean Speed:	46.97
Std. Dev.	4.72	Std. Dev.	4.75
85th Percentile Speed:	52		

Note: 1 mph = 1.6 km/h

County:	Rusk	Highway:	U.S. 79
Date:	8-26-93	Direction of Travel:	S.B.
Limits:	From St. Paul Street to People's State Bank 243.8 m (800 ft)		
Start Time:	10:17 A.M.	End Time:	10:40 A.M.
Posted Speed	50	RUSKSPAM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	53		31	37	
2	43		32	51	
3	53		33	48	
4	46		34	52	
5	40		35	45	
6	45		36	47	
7	40		37	43	
8	45		38	54	
9	46		39	47	
10	48	Y	40	40	
11	49	Y	41	48	
12	41		42	55	
13	45		43	46	
14	44		44	42	
15	49		45	43	
16	39		46	38	Y
17	48		47	48	
18	48		48	49	
19	47		49	65	
20	41		50	56	
21	41		51	54	
22	44		52	45	
23	47	Y	53	45	
24	46		54	40	Y
25	44		55	50	
26	51		56	49	
27	49		57	49	
28	39		58	49	
29	45		59	50	
30	38		60	43	

Mean Speed:	46.37	P.C. Mean Speed:	46.55
Std. Dev.	5.15	Std. Dev.	5.17
85th Percentile Speed:	51		

Note: 1 mph = 1.6 km/h

County: Rusk   Highway: U.S. 79
Date: 8-26-93   Direction of Travel: S.B.
Limits: From St. Paul Street to People's State Bank
243.8 m (800 ft)
Start Time: 2:18 P.M.   End Time: 2:39 P.M.
Posted Speed 50   RUSKSPPM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	41		31	48	
2	45		32	54	
3	41		33	47	
4	42		34	51	
5	54	Y	35	47	
6	49		36	49	
7	42		37	45	
8	52		38	48	Y
9	44		39	45	
10	47		40	45	
11	42		41	40	
12	50		42	48	
13	48		43	44	
14	49		44	47	Y
15	44		45	52	
16	44		46	51	
17	52		47	50	
18	54		48	43	
19	43		49	41	
20	48		50	45	
21	50		51	40	
22	49		52	37	
23	45	Y	53	44	
24	49		54	43	
25	50		55	47	
26	40		56	42	
27	47		57	45	
28	48		58	60	
29	41		59	51	
30	44		60	52	

Mean Speed: 46.58   P.C. Mean Speed: 46.45
Std. Dev. 4.39   Std. Dev. 4.42
85th Percentile Speed: 51

Note: 1 mph = 1.6 km/h

County: San Patricio Highway: SH 35
Date: 6-23-93 Direction of Travel: NB
Limits: Douglas Machine Works to Coastal Bend Bowling Lanes 225.6 m (740 ft)
Start Time: 10:10 A.M. End Time: 10:33 A.M.
Posted Speed 55 SANPSPAM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	47		31	47	
2	43		32	46	
3	51		33	40	Y
4	45		34	45	Y
5	41		35	39	
6	41		36	42	
7	49		37	41	
8	45		38	54	
9	42		39	41	
10	51		40	47	
11	44		41	47	
12	51		42	52	
13	46		43	44	
14	39	Y	44	45	
15	50		45	55	
16	45		46	49	
17	47		47	46	
18	48		48	56	
19	44	Y	49	54	
20	50		50	50	
21	49		51	45	
22	47		52	43	
23	41		53	50	Y
24	43		54	51	
25	45		55	49	
26	48		56	43	
27	47		57	42	
28	41		58	43	
29	47		59	44	
30	47		60	45	

Mean Speed: 46.15 P.C. Mean Speed: 46.38
Std. Dev. 4.04 Std. Dev. 3.97
85th Percentile Speed: 50

Note: 1 mph = 1.6 km/h

County: San Patricio Highway: SH 35
Date: 6-23-93 Direction of Travel: NB
Limits: Douglas Machine Works to Coastal Bend Bowling Lanes 225.6 m (740 ft)
Start Time: 2:35 P.M. End Time: 2:54 P.M.
Posted Speed 55 SANPSPPM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	43		31	64	
2	41		32	51	
3	43		33	49	
4	44		34	54	Y
5	43		35	54	
6	52		36	58	
7	52		37	36	
8	49		38	44	
9	45		39	49	
10	52		40	37	
11	44		41	47	
12	43		42	39	
13	46		43	39	
14	54		44	46	
15	49		45	51	
16	52	Y	46	49	
17	49		47	43	
18	47		48	46	
19	43		49	43	
20	51		50	43	
21	47		51	46	
22	50		52	50	
23	55		53	48	
24	56		54	46	
25	40		55	48	
26	42		56	46	
27	42		57	47	
28	49		58	38	
29	51		59	35	
30	44		60	46	

Mean Speed: 46.83 P.C. Mean Speed: 46.62
Std. Dev. 5.52 Std. Dev. 5.49
85th Percentile Speed: 52

Note: 1 mph = 1.6 km/h

County:	Smith	Highway:	SH 155
Date:	6-30-93	Direction of Travel:	SB
Limits:	146.3 m (480 ft) N. of Westway Street to 167.6 m (550 ft) S. of Westway 314 m (1030 ft)		
Start Time:	10:06 A.M.	End Time:	10:25 A.M.
Posted Speed:	55	SMI1SPAM.WQ1	

Vehicle Number	Speed (mph) PC	HV	Vehicle Number	Speed (mph) PC	HV
1	54		31	45	Y
2	56		32	47	
3	50		33	52	Y
4	53		34	56	
5	60		35	51	
6	58		36	52	
7	49		37	51	
8	56		38	49	
9	58		39	47	
10	48		40	57	
11	53		41	65	
12	60		42	43	
13	55	Y	43	54	
14	52		44	42	
15	48		45	53	
16	49		46	50	
17	60		47	56	
18	46		48	62	
19	47	Y	49	60	
20	53	Y	50	57	
21	56		51	54	
22	56		52	54	
23	57		53	53	
24	58		54	53	
25	54		55	55	
26	57		56	55	
27	56		57	64	
28	51		58	46	
29	53		59	54	
30	55		60	58	

Mean Speed:	53.55	P.C. Mean Speed:	53.84
Std. Dev.	4.86	Std. Dev.	4.85
85th Percentile Speed:	58		

Note: 1 mph = 1.6 km/h

County:	Smith	Highway:	SH 155
Date:	6-30-93	Direction of Travel:	SB
Limits:	146.3 m (480 ft) N. of Westway Street to 167.6 m (550 ft) S. of Westway 313.7 m (1030 ft)		
Start Time:	2:30 P.M.	End Time:	2:50 P.M.
Posted Speed:	55	SMI1SPPM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	55		31	64	
2	53		32	51	
3	50	Y	33	54	
4	59		34	50	
5	51		35	48	
6	53		36	48	
7	58		37	64	
8	59		38	48	
9	51		39	55	
10	53		40	50	
11	53		41	56	
12	54	Y	42	63	
13	55		43	56	
14	52		44	53	
15	52		45	56	
16	61		46	46	
17	64		47	56	
18	51	Y	48	54	
19	54		49	61	
20	47	Y	50	56	
21	59		51	57	
22	56		52	59	
23	56		53	52	Y
24	49		54	55	
25	51		55	50	
26	47		56	50	
27	45	Y	57	48	
28	60		58	54	
29	51		59	53	
30	59		60	52	

Mean Speed:	53.95	P.C. Mean Speed:	54.41
Std. Dev.	4.63	Std. Dev.	4.55
85th Percentile Speed:	59		

Note: 1 mph = 1.6 km/h

County: Tom Green Highway: R.M. 584
Date: 8-19-93 Direction of Travel: S.B.
Limits: 30 m (100 ft) No. of Red Bluff Rd. to 168 m (550 ft) So. of Red Bluff Rd. 198 m (650 ft)
Start Time: 10:05 A.M. End Time: 10:40 A.M.
Posted Speed 55 TOMGSPAM.WQ1

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	40		31	49	
2	52		32	46	
3	54		33	46	
4	58		34	46	
5	46		35	49	
6	64		36	61	
7	47		37	52	
8	44		38	51	
9	46		39	44	
10	56		40	47	
11	36		41	50	
12	58		42	48	
13	52		43	50	
14	48		44	40	
15	46		45	44	
16	47		46	57	
17	54		47	56	
18	59		48	54	
19	52		49	43	
20	52		50	59	
21	49		51	55	
22	53		52	53	
23	46		53	52	
24	55		54	56	
25	49		55	47	
26	57		56	52	
27	50		57	55	
28	52		58	56	
29	51		59	46	
30	60		60	48	

Mean Speed: 50.75 P.C. Mean Speed: 50.75
Std. Dev. 5.56 Std. Dev. 5.56
85th Percentile Speed: 56

Note: 1 mph = 1.6 km/h

County:	Tom Green	Highway:	R.M. 584
Date:	8-19-93	Direction of Travel:	S.B.
Limits:	30 m (100 ft) No. of Red Bluff Rd. to 168 m (550 ft) So. of Red Bluff Rd. 198 m (650 ft)		
Start Time:	2:20 P.M.	End Time:	2:40 P.M.
Posted Speed	55	TOMGSPPM.WQ1	

Vehicle Number	Speed (mph)		Vehicle Number	Speed (mph)	
	PC	HV		PC	HV
1	53		31	58	
2	67		32	56	
3	53		33	58	
4	52		34	47	
5	56		35	54	
6	47		36	53	
7	48		37	48	
8	49		38	49	
9	50		39	59	
10	58		40	56	
11	58		41	58	
12	51		42	51	
13	53		43	46	
14	51		44	55	
15	58		45	50	
16	56		46	65	
17	47		47	52	
18	53		48	48	
19	49		49	50	
20	50		50	49	
21	53		51	49	
22	49		52	48	
23	55		53	52	
24	58		54	50	
25	54		55	52	
26	49		56	56	
27	54		57	40	
28	48		58	44	
29	54		59	51	
30	61		60	61	

Mean Speed:	52.65	P.C. Mean Speed:	52.65
Std. Dev.	4.94	Std. Dev.	4.94
85th Percentile Speed:	58		

Note: 1 mph = 1.6 km/h

APPENDIX D

SUPPORTING DATA FOR TWO-WAY LEFT-TURN LANE STUDY

CRITICAL HEADWAY CALCULATIONS

Leading, Left-Turning Vehicle

Assumptions:

- Decelerates from 80-89 km/h (50-55 mph) to 0 km/h (0 mph) to make left turn
- Speed Limit = 80-89 km/h (50-55 mph)

Deceleration Distance: 120 m (1994 AASHTO Green Book, p. 40)
400 ft (1990 AASHTO Green Book, p. 40)

Deceleration Rate:

METRIC: $$v^2 = 2dx;\ d = \frac{v^2}{2x};\ d = \frac{[(85)(0.28)\ m/s]^2}{2(120\ m)};\ d = 2.4\ m/s^2$$

ENGLISH: $$v^2 = 2dx;\ d = \frac{v^2}{2x};\ d = \frac{[(53)(1.47)\ ft/s]^2}{2(400\ ft)};\ d = 7.6\ ft/s^2$$

Deceleration Time:

METRIC: $$v = v_0 + a(t);\ t = \frac{v - v_0}{a};\ t_1 = \frac{0 - 85(0.28)\ m/s}{-2.4\ m/s^2};\ t_1 = 10\ s$$

ENGLISH: $$v = v_0 + a(t);\ t = \frac{v - v_0}{a};\ t_1 = \frac{0 - 53(1.47)\ ft/s}{-7.6\ ft/s^2};\ t_1 = 10\ s$$

Following, Through Vehicle

Assumptions:

- Following, through vehicle does not react to leading, left-turning vehicle until headway equals stopping sight distance
- Speed Limit = 80-89 km/h (50-55 mph)

At 80-89 km/h (50-55 mph), SSD = 160 m (1994 AASHTO Green Book, p.120)
515 ft (1990 AASHTO Green Book, p. 120)

METRIC:

$$x = v(t);\ t = \frac{x}{v};\ t_2 = \frac{160\ m}{85(0.28)\ m/\text{sec}};\ t_2 = 7\ s$$

ENGLISH:

$$x = v(t);\ t = \frac{x}{v};\ t_2 = \frac{515\ ft}{53(1.47)\ ft/\text{sec}};\ t_2 = 7\ s$$

$$Critical\ Headway = t_1 + t_2 = 10\ s + 7\ s = \underline{17\ s}$$

POISSON DISTRIBUTION

$$P(x) = \frac{m^x e^{-m}}{x!}$$

where

P(x)	=	probability of exactly x vehicles arriving in time interval t
m	=	average number of vehicles arriving in time interval t
x	=	number of vehicles arriving in time interval being investigated
t	=	selected time interval

Prob(Hdwy < 17.0 sec.)

Volume (vphpl)	m	p(x > 1) p(H < 17.0 sec)
0	0.000	0.000
100	0.478	0.084
200	0.956	0.248
300	1.433	0.420
400	1.911	0.569
500	2.389	0.689
600	2.867	0.780
700	3.344	0.847
800	3.822	0.894
900	4.300	0.928
1000	4.778	0.951

RESULTS FROM LEFT-TURN FIELD STUDY

Location	Length of Site (m) (ft)	Number of Drives	Drives per Kilometer (per Mile)	Total Volume	Number of Left Turns	Percent Left-Turn Vehicles
Gregg Loop 281	274.3 (900)	7	25.5 (41)	A.M. 1858 P.M. 2406	176 256	9.47 10.64
Henderson SH 31	289.6 (950)	4	13.7 (22)	A.M. 2325 P.M. 1020	126 63	5.42 6.18
Lamb US 84	243.8 (800)	2	8.07 (13)	A.M. 429 P.M. 578	31 21	4.90 2.60
Mills US 84	213.4 (700)	3	14.3 (23)	A.M. 672 P.M. 475	40 18	5.95 3.79
San Patricio SH 35	225.6 (740)	5	22.4 (36)	A.M. 887 P.M. 927	76 29	8.57 9.39
Smith SH 64	237.7 (780)	2	8.7 (14)	A.M. 1748 P.M. 741	79 33	4.52 4.45
Victoria SH 185	161.5 (530)	2	5.6 (9)	A.M. 1185 P.M. 820	40 24	3.38 2.93

PROBABILITY OF LEFT-TURN, SAME DIRECTION CONFLICT

Volume (vph)	Driveway Density drives /kilometer (drives/mile)			
	6 (10)	12 (20)	18 (30)	24 (40)
0	0.000	0.000	0.000	0.000
100	0.069	0.117	0.165	0.213
200	0.246	0.411	0.577	0.742
300	0.496	0.816	1.137	1.458
400	0.791	1.284	1.776	2.269
500	1.112	1.779	2.447	3.114
600	1.444	2.280	3.116	3.953
700	1.776	2.770	3.764	4.758
800	2.101	3.239	4.377	5.514
900	2.415	3.682	4.948	6.214
1000	2.715	4.095	5.475	6.855

APPENDIX E

TYPICAL BENEFIT/COST MODEL INPUT

The following is a typical input file for the benefit/cost model used in the clear zone study as described in Chapter 5. This particular data file is for a baseline clear zone of 3.05 m (10 ft), a high roadside hazard rating, and a R.O.W. cost of \$21.5/m^2 (\$2/ft^2). The alternatives or options correspond to incremental increases in the clear zone. In addition to the R.O.W. purchase cost, the cost of each alternative includes a pole relocation cost of \$1,500 per pole and a clearing cost of \$1,000 per 4047 m^2 (1 acre). In the analysis, the ADT is varied until a B/C ratio of 1.0 is obtained.

```
5.0   3.0
1.0   4.0   4.0   100.0 0.025 55.
10.0  10.0  10.0  10.0  10.0  0.04
0.0   1375. 3135. 10295. 25350. 56535. 116555. 186150. 281720. 395500.
500000. 0.0  0.0  0.0  0.0  0.0  0.0
5.83  15.83 3000. 1.0  0.0  6.0  18.0  4000. 0.0  0.0
8.5   40.   12812. 0.0  0.0  8.5  55.  55121. 0.0  0.0
1.0   0.0   0.0   0.0  0.0  0.0  0.0  0.0   0.0  0.0
OPTION 1 - clear zone width = 10 ft (3.0 m), 500 ft (152.4 m) layout
2 utility poles spaced 250 ft (76.2 m) apart, high hazard rating (10 trees spaced 50 ft (15.2 m) apart)
12.0  0.0   0.0   0.0
0.0   0.0   1.0              first utility pole
125.0 10.0  0.833 0.833 0.0  0.0  0.0  0.0  6.5  280000.
280000. 280000. 280000.
0.0107 59277.
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0929 6.5  0.0  6.5  0.0
0.0   0.0   0.0
```

```
0.0    0.0    1.0                   second utility pole
375.0  10.0   0.833  0.833  0.0   0.0   0.0   0.0   6.5   280000.
280000. 280000. 280000.
0.0107  59277.
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0929 6.5    0.0    6.5   0.0
0.0    0.0    0.0
0.0    0.0    1.0                   first tree
25.0   15.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
0.0    0.1229 10.0   0.0    10.0  0.0
```

```
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0     1.0                  second tree
75.0   15.0    0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0     1.0                  third tree
125.0  15.0    0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
```

```
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.0    0.0
0.0   0.0    1.0                fourth tree
175.0  15.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0   0.0    0.0
0.0   0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.0    0.0
0.0   0.0    1.0                fifth tree
225.0  15.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0   0.0    0.0
0.0   0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
```

```
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.0     0.0
0.0     0.0     1.0                 sixth tree
275.0   15.0    0.833   0.833   0.0     0.0     0.0     0.0     10.0    0.0
0.0     0.0     0.0
0.0     0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.0     0.0
0.0     0.0     1.0                 seventh tree
325.0   15.0    0.833   0.833   0.0     0.0     0.0     0.0     10.0    0.0
0.0     0.0     0.0
0.0     0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
0.0     0.1229  10.0    0.0     10.0    0.0
```

```
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.0    0.0
0.0    0.0    1.0                eighth tree
375.0  15.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.1229 10.0  0.0   10.0  0.0
0.0    0.0    0.0
0.0    0.0    1.0                ninth tree
425.0  15.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0  0.0   10.0  0.0
```

```
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.0    0.0
0.0    0.0    1.0                  tenth tree
475.0  15.0   0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.0    0.0
```

OPTION 2 - clear zone width = 15 ft (4.6 m), 500-ft (152.4-m) layout, pole relocation cost=2*$1,500

2 utility poles spaced 250 ft (76.2 m) apart, high hazard rating (10 trees spaced 50 ft (15.2 m) apart)

```
12.0   8057.4 0.0    0.0
0.0    0.0    1.0                 first utility pole
125.0  15.0   0.833  0.833  0.0    0.0    0.0    0.0    6.5    280000.
280000. 280000. 280000.
0.0107  59277.
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0    0.0
0.0    0.0    1.0                 second utility pole
375.0  15.0   0.833  0.833  0.0    0.0    0.0    0.0    6.5    280000.
280000. 280000. 280000.
0.0107  59277.
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
0.0    0.0929 6.5    0.0    6.5    0.0
```

```
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0929 6.5   0.0   6.5   0.0
0.0   0.0    0.0
0.0   0.0    1.0               first tree
25.0  20.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0   0.0
0.0   0.0    0.0
0.0   0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.0    0.0
0.0   0.0    1.0               second tree
75.0  20.0   0.833  0.833  0.0   0.0   0.0   0.0   10.0   0.0
0.0   0.0    0.0
0.0   0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
0.0   0.1229 10.0  0.0   10.0  0.0
```

```
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.0   0.0
0.0   0.0   1.0               third tree
125.0  20.0  0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0   0.0   0.0
0.0   0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.0   0.0
0.0   0.0   1.0               fourth tree
175.0  20.0  0.833  0.833  0.0   0.0   0.0   0.0   10.0  0.0
0.0   0.0   0.0
0.0   0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
0.0   0.1229  10.0  0.0   10.0  0.0
```

```
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0     1.0                  fifth tree
225.0  20.0    0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0     1.0                  sixth tree
275.0  20.0    0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0
```

```
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0     1.0                  seventh tree
325.0  20.0    0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0     0.0
0.0    0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
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0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.1229  10.0   0.0    10.0   0.0
0.0    0.0     0.0
```

```
0.0    0.0    1.0                eighth tree
375.0  20.0   0.833  0.833  0.0  0.0  0.0  0.0  10.0  0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.0    0.0
0.0    0.0    1.0                ninth tree
425.0  20.0   0.833  0.833  0.0  0.0  0.0  0.0  10.0  0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.0    0.0
0.0    0.0    1.0                  tenth tree
475.0  20.0   0.833  0.833  0.0    0.0    0.0    0.0    10.0   0.0
0.0    0.0    0.0
0.0    0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
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0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.1229 10.0   0.0    10.0   0.0
0.0    0.0    0.0
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APPENDIX F

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN HIGHWAYS, ENGLISH UNITS

SUBURBAN HIGHWAYS (4-300 A)

4-301 A GENERAL

The term "suburban highway" as used in this publication refers to high-speed (50 to 55 mph) roadways which serve as transitions between low speed (45 mph and below) urban streets and high speed rural highways (i.e., suburban highways connect urban streets to rural highways). These facilities are typically 1 to 3 miles in length and have light to moderate driveway densities (approximately 10 to 30 driveways per mile). Because of their location, suburban highways have both rural and urban characteristics. For example, these sections typically maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban characteristic). Consequently, guidelines for suburban highways typically fall between those for rural highways and urban streets.

4-302 A BASIC DESIGN FEATURES

Figure 4-26 A shows tabulated basic geometric design criteria for suburban highways. The basic design criteria shown in Figure 4-26 A reflect minimum and desired values which are applicable to projects on new location or major improvement projects.

A. Access Control

A major concern for suburban highways is the large number of access points introduced due to commercial development. This creates conflicts between exiting/entering traffic and through traffic. In addition, the potential for severe accidents is increased due to the high-speed differentials. Driver expectancy is also violated because through traffic traveling at high speeds does not expect to have to slow down or stop. Research has shown that reducing the number of access points and increasing the amount of access control will reduce the potential for accidents. In addition, accident experience can be reduced by separating conflicting traffic movements with the use of turn bays and/or turn lanes.

Access driveways shall be installed in accordance with the departmental publication, Regulations For Access Driveways To State Highways.

B. Medians

Medians are desirable for suburban highways with four or more lanes primarily to provide storage space for left-turning vehicles. Medians may be curbed or flush.

1. Curbed Medians

Raised medians with curbing are used on suburban arterials where it is desirable to control left-turn movements. These medians should be delineated with curbs of the mountable type. Curbed medians are applicable on high-volume roadways with high demand for left turns. Curbed medians should be minimally 12 feet (10-foot storage lane plus 2-foot

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN HIGHWAYS

Item	Functional Class	Desired	Minimum
Design Speed (mph)	All	60	50
Maximum Horiz. Curvature (degrees)	All	See Figure 4-4	
Maximum Gradient (%)	All	See Figure 4-14	
Stopping Sight Distance (ft)	All	--	200
Width of Travel Lanes (ft)	Arterial	12	11[1]
	Collector	12	10[2]
Curb Parking Lane Width (ft)	All	None	
Shoulder Width[3] (ft)	All	10	4
Width of Refuge Lanes[4] (ft)	All	11 - 12	10
Offset to Face of Curb (ft)	All	4[5]	2
Median Width (ft)	All	See Sec. 4-302A(B)1&2	
Border Width (ft)	Arterial	12	8[6]
	Collector	11	8[6]
Right-of-Way Width (ft)	All	Determined by Local Conditions	
Sidewalk Width (ft)	All	6 - 8[7]	4
Superelevation	All	Yes	None
Clear Zone Widths (ft)	All	See Sec. 4-302A(G)	
Vertical Clearance for New Strs. (ft)	All	16.5	16.5[8]
Turning Radii	All	See Sec. 4-710(D)	
Structure Widths (ft)	All	Curb face-to-curb face plus sidewalk(s)	

1 In highly restricted locations, 10 ft permissible.
2 In industrial areas 12 ft usual, and 11 ft minimum for restricted R.O.W. conditions. In non-industrial areas, 10 ft minimum.
3 For ADT > 5000 shoulders provide significant benefit. For ADT < 3000 shoulders provide no significant benefit.
4 Applicable when right or left-turn lanes are provided.
5 Applicable for areas with concentrated bicycle traffic.
6 Depends on clear zone requirements.
7 Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
8 Exceptional cases near as practical to 16.5 ft but never less than 14.5 ft. Existing structures that provide at least 14 ft may be retained.

Figure 4-26 A. Refers to Paragraph 4-302 A

divider at restricted locations) and desirably up to 18 feet (12-foot storage lane plus 6-foot divider) in width.

2. Flush Medians

Flush medians may include pavement markings delineating directional turning bays, or they may be used where appropriately marked as continuous two-way left-turn lanes (TWLTL). The TWLTL design allows use of the flush median area for left turns by traffic from either direction. The TWLTL is applicable on suburban highways with moderate traffic volumes and low to moderate demands for left turns. For suburban highways, TWLTL facilities should minimally be 14 feet and desirably 16 feet in width.

The usual value of 16 feet width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The "minimum" value of 14 feet width is appropriate for restrictive right-of-way projects and improvement projects where attaining "usual" median lane width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

To warrant the use of a continuous two-way left-turn lane on a suburban highway, the following three criteria should be met:

1. ADT volume of 3000 or more;
2. Side road plus driveway density of 10 or more entrances per mile; and
3. Length of three lane section of 1.5 miles or less.

C. Borders and Sidewalks

The border, which accommodates sidewalks, utilities, etc., and separates traffic from privately-owned areas, is the area between the roadway and right-of-way line. Every effort should be made to provide wide borders to serve functional needs, reduce traffic nuisances to adjacent development, and for aesthetic reasons. Sidewalks should be a minimum of 4 feet in width with increased widths applicable near schools, commercial areas, or other areas with high pedestrian volumes. A 2-foot separation should be provided between the backside of curb and the edge of sidewalk. Border widths minimally are 8 feet and desirably 12 feet or more.

D. Bicycle Facilities

Generally, on high-speed roadways minimum shoulder and curb-offset widths are adequate to accommodate expert riders. If high bicycle volumes are anticipated, or volumes with less experienced riders, separate facilities should be considered. Additional guidance is presented in AASHTO's *Guide for the Development of Bicycle Facilities*.

E. Grade Separations and Interchanges

Although grade separations and interchanges are infrequently provided on suburban highways, they may be the only means available for providing sufficient capacity at critical intersections.

Normally, a grade separation is part of an interchange (except grade separations with railroads); it is usually of the diamond type with four legs. Locations considered include high-volume intersections and intersections where terrain conditions favor separation of grades.

F. Right-of-Way Width

The width of right-of-way for suburban highways is influenced by traffic volume requirements, land use, cost, extent of ultimate expansion, and availability. Width is the summation of the various cross-sectional elements, including widths of travel lanes, shoulders, median, sidewalks, and borders.

G. Clear Zone

Guidelines for clear zone widths for suburban highways are developed based on the benefit/cost approach. The basic concept behind this approach is that funds should only be invested in projects where the expected benefits would equal or exceed the expected direct costs of the project. Figure 4-27A presents the general clear zone guidelines for suburban highways. The information is intended to provide general guidance to the highway engineer in the selection of appropriate clear zone widths for high-speed curb and gutter sections.

The recommendations contained in this figure are rather straightforward. For each of the four different ADT ranges, the minimum and desirable clear zone widths are provided. For example, for roadways with ADT between 8,000 and 12,000, the recommended minimum and desirable clear zone widths are 10 ft and 20 ft, respectively. Due to the probabilistic nature of the benefit/cost analyses and the assumptions inherent therein, some flexibility in the application of this information is considered acceptable and a certain degree of judgment should be exercised.

Figure 4-27 A. General Clear Zone Guidelines

ADT	Recommended Clear Zone Distance[1], ft	
	Minimum	Desirable
< 8,000	10	10
8,000 - 12,000	10	20
12,000 - 16,000	10	25
> 16,000	20	30

[1]Note. Purchase of 5 ft or less of additional right-of-way strictly for satisfying clear-zone provisions is not cost-beneficial and thus not required.

H. Intersections

The number, design, and spacing of intersections influence the capacity, speed, and safety on suburban highways. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Dimensional layout or geometric design considerations are closely influenced by traffic volumes and operational characteristics and the type of traffic control measures used.

Due to high operating speeds (50 mph or greater) on suburban highways, curve radii for turning movements should equal that of rural highway intersections (see Section 4-710); however, space restrictions due to right-of-way limitations in suburban areas may necessitate reduction in the values given for rural highways. Where heavy volumes of trucks or buses are present, increased curb radii of 30 or 50 feet expedite turns to and from through lanes. Where combination tractor-trailer unit are anticipated in significant volume, reference should be made to the material in Section 4-710.

In general, intersection design should be rather simple, free of complicated channelization, to minimize driver confusion. Sight distance is an important consideration even in the design of signalized intersections since, during the low-volume hours, flashing operation may be used.

I. Speed Change Lanes

Depending on available cross-section and due to high operating speeds on suburban highways, speed change lanes may be provided as space for deceleration and acceleration from intersecting side streets with significant volumes.

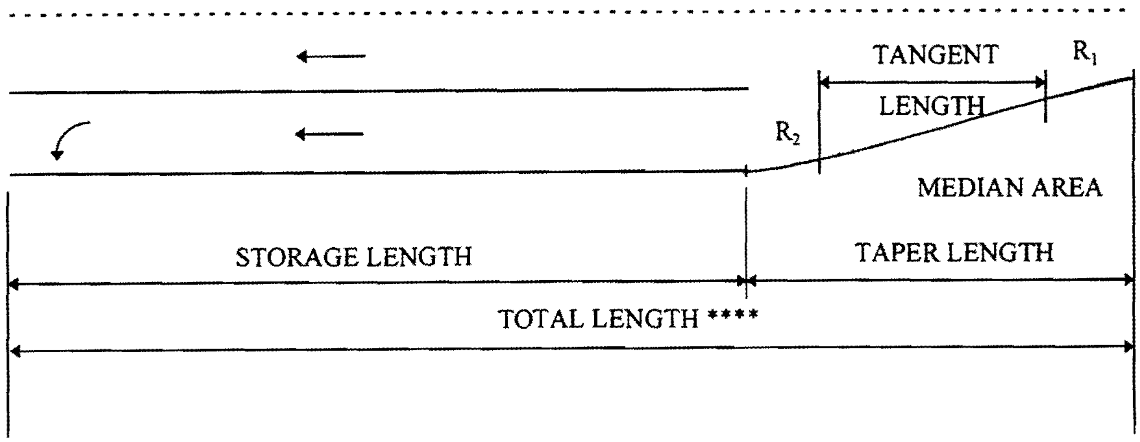
Figure 4-28 A shows taper and storage lengths for left turn lanes on suburban highways. A short curve is desirable on each end of the taper, but may be omitted for construction ease. Where reverse curves are used, the intervening tangent should be one-third to one-half of the total taper length, and the turnoff curve should be about twice the radius of the second curve.

J. Parking

Desirably, parking adjacent to the curb on suburban highways should not be allowed.

LENGTH OF LEFT TURN LANES - SUBURBAN HIGHWAY *

DESIGN SPEED (mph)	MINIMUM TAPER LENGTH (ft)	STORAGE LENGTH (ft) **			
		SIGNALIZED		NON-SIGNALIZED	
		MIN.	DES.	MIN.	DES.
50	180	***	320	100	320



$R_1 = 2R_2$ (Approx.)
TANGENT LENGTH = (1/3 TO 1/2) (TAPER LENGTH)
NOTE: TAPER LENGTH AND STORAGE LENGTH FROM TABLE

* APPLICABLE TO SPEED CHANGE LANES TO ACCOMMODATE LEFT OR U-TURNS AT MEDIAN OPENING OR INTERSECTIONS; APPLIES ALSO TO SPEED CHANGE LANES FOR RIGHT TURNS WHERE DESIRED.

** BLOCK SPACING MAY DICTATE LESSER VALUES.

*** BASED ON DESIGN HOUR VOLUME; STORAGE LENGTH = 0.19 TO 0.25 MULTIPLIED BY LEFT TURN PEAK HOUR VOLUME.

**** TOTAL LENGTH OF LEFT TURN LANE = STORAGE LENGTH + TAPER LENGTH.

Figure 4-28 A. Refers to Paragraph 4-302 A (I)

APPENDIX G

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN HIGHWAYS, METRIC UNITS

SUBURBAN HIGHWAYS (4-300 A)

4-301 A GENERAL

The term "suburban highway" as used in this publication refers to high-speed (80 to 88 km/h) roadways which serve as transitions between low speed (72 km/h and below) urban streets and high speed rural highways (i.e., suburban highways connect urban streets to rural highways). These facilities are typically 1.6 to 4.8 kilometers in length and have light to moderate driveway densities (approximately 5 to 20 driveways per kilometer). Because of their location, suburban highways have both rural and urban characteristics. For example, these sections typically maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban characteristic). Consequently, guidelines for suburban highways typically fall between those for rural highways and urban streets.

4-302 A BASIC DESIGN FEATURES

Figure 4-26 A shows tabulated basic geometric design criteria for suburban highways. The basic design criteria shown in Figure 4-26 A reflect minimum and desired values which are applicable to projects on new location or major improvement projects.

A. Access Control

A major concern for suburban highways is the large number of access points introduced due to commercial development. This creates conflicts between exiting/entering traffic and through traffic. In addition, the potential for severe accidents is increased due to the high-speed differentials. Driver expectancy is also violated because through traffic traveling at high speeds does not expect to have to slow down or stop. Research has shown that reducing the number of access points and increasing the amount of access control will reduce the potential for accidents. In addition, accident experience can be reduced by separating conflicting traffic movements with the use of turn bays and/or turn lanes.

Access driveways shall be installed in accordance with the departmental publication, *Regulations For Access Driveways To State Highways*.

B. Medians

Medians are desirable for suburban highways with four or more lanes primarily to provide storage space for left-turning vehicles. Medians may be curbed or flush.

1. Curbed Medians

Raised medians with curbing are used on suburban arterials where it is desirable to control left-turn movements. These medians should be delineated with curbs of the mountable type. Curbed medians are applicable on high-volume roadways with high demand for left turns. Curbed medians should be minimally 3.6 m (3.0 m storage lane plus 0.6 m divider

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN HIGHWAYS

Item	Functional Class	Desired	Minimum
Design Speed (km/h)	All	95	80
Maximum Horiz. Curvature (degrees)	All	See Figure 4-4	
Maximum Gradient (%)	All	See Figure 4-14	
Stopping Sight Distance (m)	All	--	60
Width of Travel Lanes (m)	Arterial	3.6	3.3[1]
	Collector	3.6	3.0[2]
Curb Parking Lane Width (m)	All	None	
Shoulder Width[3] (m)	All	3.0	1.2
Width of Refuge Lanes[4] (m)	All	3.3-3.6	3.0
Offset to Face of Curb (m)	All	1.2[5]	0.6
Median Width (m)	All	See Sec. 4-302A(B)1&2	
Border Width (m)	Arterial	3.6	2.4[6]
	Collector	3.3	2.4[6]
Right-of-Way Width (m)	All	Determined by Local Conditions	
Sidewalk Width (m)	All	1.8-2.4[7]	1.2
Superelevation	All	Yes	None
Clear Zone Widths (m)	All	See Sec. 4-302A(G)	
Vertical Clearance for New Strs. (m)	All	5.0	5.0[8]
Turning Radii	All	See Sec. 4-710(D)	
Structure Widths (m)	All	Curb face-to-curb face plus sidewalk(s)	

1 In highly restricted locations, 3.0 m permissible.
2 In industrial areas 3.6 m usual, and 3.3 m minimum for restricted R.O.W. conditions. In non-industrial areas, 3.0 m minimum.
3 For ADT > 5000 shoulders provide significant benefit. For ADT < 3000 shoulders provide no significant benefit.
4 Applicable when right or left-turn lanes are provided.
5 Applicable for areas with concentrated bicycle traffic.
6 Depends on clear zone requirements.
7 Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
8 Exceptional cases near as practical to 5.0 m but never less than 4.4 m. Existing structures that provide at least 4.3 m may be retained.

Figure 4-26 A. Refers to Paragraph 4-302 A

at restricted locations) and desirably up to 5.4 m (3.6 m storage lane plus 1.8 m divider) in width.

2. Flush Medians

Flush medians may include pavement markings delineating directional turning bays, or they may be used where appropriately marked as continuous two-way left-turn lanes (TWLTL). The TWLTL design allows use of the flush median area for left turns by traffic from either direction. The TWLTL is applicable on suburban highways with moderate traffic volumes and low to moderate demands for left turns. For suburban highways, TWLTL facilities should minimally be 4.2 m and desirably 4.8 m in width.

The usual value of 4.8 m width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The "minimum" value of 4.2 m width is appropriate for restrictive right-of-way projects and improvement projects where attaining "usual" median lane width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

To warrant the use of a continuous two-way left-turn lane on a suburban highway, the following three criteria should be met:

1. ADT volume of 3000 or more;
2. Side road plus driveway density of 6 or more entrances per kilometer; and
3. Length of three lane section of 2.4 kilometers or less.

C. Borders and Sidewalks

The border, which accommodates sidewalks, utilities, etc., and separates traffic from privately-owned areas, is the area between the roadway and right-of-way line. Every effort should be made to provide wide borders to serve functional needs, reduce traffic nuisances to adjacent development, and for aesthetic reasons. Sidewalks should be a minimum of 1.2 m in width with increased widths applicable near schools, commercial areas, or other areas with high pedestrian volumes. A 0.6 m separation should be provided between the backside of curb and the edge of sidewalk. Border widths minimally are 2.4 m and desirably 3.6 m or more.

D. Bicycle Facilities

Generally, on high-speed roadways minimum shoulder and curb-offset widths are adequate to accommodate expert riders. If high bicycle volumes are anticipated, or volumes with less experienced riders, separate facilities should be considered. Additional guidance is presented in AASHTO's *Guide for the Development of Bicycle Facilities.*

E. Grade Separations and Interchanges

Although grade separations and interchanges are infrequently provided on suburban highways, they may be the only means available for providing sufficient capacity at critical intersections.

Normally, a grade separation is part of an interchange (except grade separations with railroads); it is usually of the diamond type with four legs. Locations considered include high-volume intersections and intersections where terrain conditions favor separation of grades.

F. Right-of-Way Width

The width of right-of-way for suburban highways is influenced by traffic volume requirements, land use, cost, extent of ultimate expansion, and availability. Width is the summation of the various cross-sectional elements, including widths of travel lanes, shoulders, median, sidewalks, and borders.

G. Clear Zone

Guidelines for clear zone widths for suburban highways are developed based on the benefit/cost approach. The basic concept behind this approach is that funds should only be invested in projects where the expected benefits would equal or exceed the expected direct costs of the project. Figure 4-27A presents the general clear zone guidelines for suburban highways. The information is intended to provide general guidance to the highway engineer in the selection of appropriate clear zone widths for high-speed curb and gutter sections.

The recommendations contained in this figure are rather straightforward. For each of the four different ADT ranges, the minimum and desirable clear zone widths are provided. For example, for roadways with ADT between 8,000 and 12,000, the recommended minimum and desirable clear zone widths are 3.0 m and 6.1 m, respectively. Due to the probabilistic nature of the benefit/cost analyses and the assumptions inherent therein, some flexibility in the application of this information is considered acceptable and a certain degree of judgment should be exercised.

Figure 4-27 A. General Clear Zone Guidelines

ADT	Recommended Clear Zone Distance[1], m	
	Minimum	Desirable
< 8,000	3.0	3.0
8,000 - 12,000	3.0	6.1
12,000 - 16,000	3.0	7.6
> 16,000	6.1	9.1

[1]Note. Purchase of 1.5 m or less of additional right-of-way strictly for satisfying clear-zone provisions is not cost-beneficial and, thus, not required.

H. Intersections

The number, design, and spacing of intersections influence the capacity, speed, and safety on suburban highways. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Dimensional layout or geometric design considerations are closely influenced by traffic volumes and operational characteristics and the type of traffic control measures used.

Due to high operating speeds (80 km/h or greater) on suburban highways, curve radii for turning movements should equal that of rural highway intersections (see Section 4-710); however, space restrictions due to right-of-way limitations in suburban areas may necessitate reduction in the values given for rural highways. Where heavy volumes of trucks or buses are present, increased curb radii of 9.1 m or 15.2 m expedite turns to and from through lanes. Where combination tractor-trailer unit are anticipated in significant volume, reference should be made to the material in Section 4-710.

In general, intersection design should be rather simple, free of complicated channelization, to minimize driver confusion. Sight distance is an important consideration even in the design of signalized intersections since, during the low-volume hours, flashing operation may be used.

I. Speed Change Lanes

Depending on available cross-section and due to high operating speeds on suburban highways, speed change lanes may be provided as space for deceleration and acceleration from intersecting side streets with significant volumes.

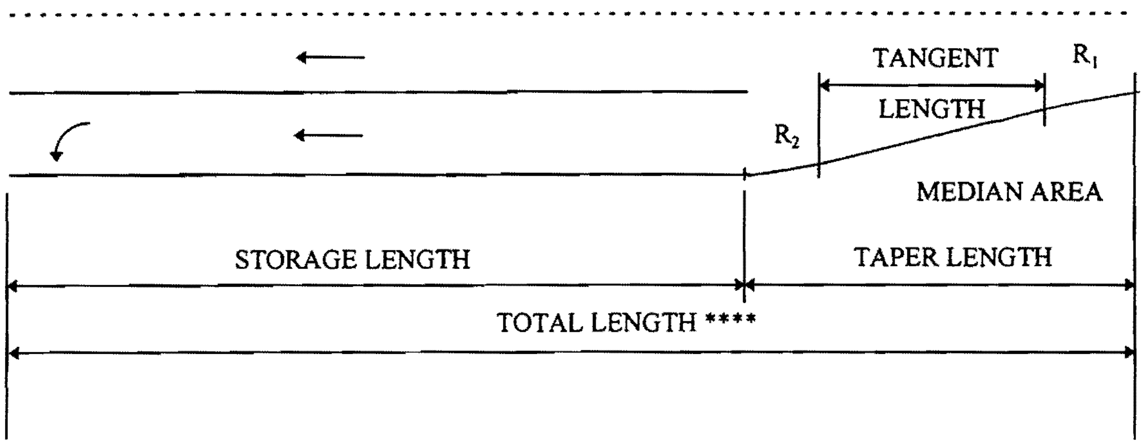
Figure 4-28 A shows taper and storage lengths for left turn lanes on suburban highways. A short curve is desirable on each end of the taper, but may be omitted for construction ease. Where reverse curves are used, the intervening tangent should be one-third to one-half of the total taper length, and the turnoff curve should be about twice the radius of the second curve.

J. Parking

Desirably, parking adjacent to the curb on suburban highways should not be allowed.

LENGTH OF LEFT TURN LANES - SUBURBAN HIGHWAY *

DESIGN SPEED (km/h)	MINIMUM TAPER LENGTH (m)	STORAGE LENGTH (m) **			
		SIGNALIZED		NON-SIGNALIZED	
		MIN.	DES.	MIN.	DES.
80	55	***	100	30	100



$R_1 = 2R_2$ (Approx.)
TANGENT LENGTH = (1/3 TO 1/2) (TAPER LENGTH)
NOTE: TAPER LENGTH AND STORAGE LENGTH FROM TABLE

* APPLICABLE TO SPEED CHANGE LANES TO ACCOMMODATE LEFT OR U-TURNS AT MEDIAN OPENING OR INTERSECTIONS; APPLIES ALSO TO SPEED CHANGE LANES FOR RIGHT TURNS WHERE DESIRED.

** BLOCK SPACING MAY DICTATE LESSER VALUES.

*** BASED ON DESIGN HOUR VOLUME; STORAGE LENGTH = 0.058 TO 0.076 MULTIPLIED BY LEFT TURN PEAK HOUR VOLUME.

**** TOTAL LENGTH OF LEFT TURN LANE = STORAGE LENGTH + TAPER LENGTH.

Figure 4-28 A. Refers to Paragraph 4-302 A (I)