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16. Abstract

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CONGRESS AVENUE REGIONAL ARTERIAL STUDY: GRADE SEPARATIONS

by

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Research Report SWUTC 95/60019-3

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EXECUTIVE SUMMARY

This report is the third of four which document work performed as part of the Southwest Region University Transportation Center (SWRUTC) study "Demonstration of Enhanced Arterial Street Traffic Flow, Reduced Fuel Consumption and User Costs Through Application of Super Street Technology". This study constitutes an effort to demonstrate user benefits through development and application of state-of-the-art traffic engineering technology. Specifically, it is an effort to produce an improvement program for Congress Avenue in Austin, Texas which will upgrade its functional class from "major arterial" street to "regional arterial status" and quantify associated user benefits. One extremely important study component is development of new technology which can solve basic problems encountered during improvement plan preparation.

This report presents an analysis of costs versus user benefits of grade separation structures as part of a long range super street development program. Grade separations on non-controlled access arterial streets are not a totally new concept, but have been rarely implemented. Analyses and resulting guidelines presented here a based upon user benefits stemming from reduced fuel consumption, vehicle delay, and mobile source emissions. User fuel and time savings are converted to dollars and compared to conceptual construction costs.

The analysis of candidate grade separation sites along the Congress Avenue corridor is presented as a series of four case studies. Guidelines developed in earlier sections are applied and tested. Economic analyses considering potential user time and fuel savings are presented. Detailed computer simulation of before and after grade separation implementation cases is used as an impact quantification tool.

ABSTRACT

This study describes a simple analysis for determining whether or not grade separation is warranted for intersections along urban arterial streets (a case study of four major intersections along Congress Avenue in Austin, Texas is provided). This determination required the evaluation of user benefits attributable to operational and design improvements made to arterial intersections. Comparisons are made between grade-separated interchanges (GSI) and at-grade intersections (AGI) in terms of the delay, user travel-time costs, and vehicle operating costs. Overall, the justification for grade separation is dependent on the user benefits offsetting the interchange construction cost over an assumed design life. A discussion is provided on generalized warrants for grade separation and on methodologies, used by others, to justify such structures. Also, this study discusses numerous geometric design considerations for grade-separated interchanges and other roadway facilities.

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CHAPTER 1 INTRODUCTION

Major cities throughout the nation are experiencing considerable surges of population growth and consequently, the traffic demands on each city's transportation infrastructure also increases. With these increased traffic demands comes the overwhelming problem of decreased mobility. Sources of public transportation help alleviate congestion to some degree, but the majority of mobility is still handled by highway systems. Therefore, the need to improve these systems has become a matter of utmost importance. Unfortunately, many highways are approaching the end of their design lives, and for a multitude of reasons highway planning, construction, and improvements have not kept up with the demand for improvement [29]. Also, given constraints such as access control, limited right-of-way, environmental and esthetic issues, and especially the limitations of time and funding, it is generally difficult, if not impossible, to accommodate increasing traffic demands on overloaded segments of highway. Much of the burden of urban mobility has inevitably fallen onto arterial streets. In many cases these arterial streets already have limited capacity.

Since traffic congestion on urban arterial streets is becoming an ever increasing problem and frustration to motorists, the need for arterial modifications which minimize vehicle delay and ideally increase capacity becomes the concern of all who use the roadway. Since at-grade intersections represent primary capacity constraints, grade separation provides one of the best means of handling capacity problems. However, due to various design constraints, not to mention cost constraints, there exists an inherent order in which improvements can be provided to an arterial street. In general, surface treatments such as signal optimization, channelization, and pavement re-striping represent the most cost and time effective means of increasing mobility. However, when all relevant at-grade solutions have been exhausted and mobility problems continue to plague an arterial street, the introduction of grade-separated interchanges becomes the next, if only, design alternative [6].

BACKGROUND

Because of construction expenses, time requirements, land acquisition problems, and traffic flow disruptions, grade separation has conventionally been associated with freeway design. However, with the advent of strategic arterials or "high-flow arterials" this association has diminished considerably [24]. Today many cities with congested arterial streets are developing plans to alleviate their traffic congestion through the strategic placement of grade-separated interchanges at overly saturated intersections. Los Angeles, Chicago, and Houston are but a few such cities with projects completed, or in the works. Since the available right-of-way adjacent to surface intersections is generally limited, the implementation of grade separation structures which demand minimal right-of-way are essential to the project cost-effectiveness. Flyovers and various other diamond-type interchanges are structures which have minimal space requirements and therefore, can generally be designed to fit, that is "retrofitted", within constricted spaces.

Flyovers are grade-separated structures ideally suited for the retrofit of surface intersections. They can be designed to fit within an existing right-of-way of 100 feet, even after the surface intersection has been fully developed. Flyovers are defined as structures which allow arterial through traffic to go over a crossing arterial or collector without slowing down or stopping for an at-grade signal. They have also been described as being "prefabricated" structures, narrower than typical diamond interchanges, with limited storage length under the structure [7]. Flyovers not only accommodate through traffic, but in special cases they are used to allow turning lanes to bypass congested intersections [24]. Because of the flyovers' characteristic of being narrow, the at-grade portion of the bypass should generally be signalized as a wide intersection rather than a typical diamond interchange. Grade-separated interchanges that operate as a single intersection, as opposed to the dual intersection operation of most diamond-type forms, are typically categorized as "urban interchanges" [30], or single-point diamonds.

The versatile nature of diamond-type interchanges has allowed them to experience a history of great success when utilized in urban locations [20]. This success is due in part to its overall compactness. Diamond-type interchanges which employ the use of retaining walls, thereby limiting the amount of right-of-way necessary for construction, include the compressed diamond and the single-point diamond. The most significant geometric and operational difference between these two structures is in their intersection orientation and phasing requirements respectively. Compressed diamonds have dual intersections and typically utilize a four-phase (overlap) signal phasing pattern, on the other hand the single-point diamond is comprised of a single intersection and can utilize a three-phased pattern. Despite the fact that single-point diamonds can accommodate simultaneous U-turns, and simultaneous multiple-lane left and right turns, they still require longer clearance intervals than other diamond-type interchanges [13]. Therefore, any possible operational advantage the three-phase signal pattern has over four-phased patterns quickly diminishes. Also, in situations where frontage roads are present the three-phased patterns are not possible [21]. Overall, the greater variability of traffic patterns accommodated by compressed diamonds makes them more efficient than single-point diamonds.

SCOPE AND LIMITATIONS OF REPORT

It is apparent from the literature that there exists a general lack of studies which specifically address the issue of when or whether a given design alternative, namely grade separation, is warranted for an arterial intersection. Therefore, this study provides a methodology for evaluating the necessity of grade separations along regional arterials. This methodology examines both the operational improvements provided by grade separations and any benefits/costs associated with these improvements. Operational improvements were measured by the delay savings and the increased capacity of grade-separated interchanges when compared to that of surface intersections. Costs to motorists were measured in excess user travel time and vehicle operating costs (i.e. fuel consumption costs and non-fuel costs such as oil consumption, tire wear, maintenance/repairs, and vehicle depreciation). Benefits to motorists were measured by taking the difference of driver costs for the existing conditions and for the conditions after the improvement. Through these measures guidelines could be developed for warranting grade separations.

This study evaluates common four-leg at-grade intersections where the intended improvement is a diamond-type interchange with grade-separated through lanes on the major approach and signalization on the surface cross street (i.e. minor approach). In general this represents the most common configuration encountered in an urban arterial environment. At-grade intersections that would require exotic or atypical design layouts would likely require a more specialized and detailed evaluation. Therefore, such intersections are beyond the scope of this report. However, the guidelines proposed in this study should remain an applicable means of aiding decision-makers and/or planners in justifying grade separations for most arterial intersections.

Since it is not cost-effective to purchase large quantities of right-of-way, especially when the adjacent land has already been developed, this study assumes that any introduction of grade-separated interchanges to problematic intersections would be retrofitted within the existing right-of-way. Generally it is the case where regional arterials are highly developed corridors with limited amounts of available adjacent right-of-way. Therefore, in this study land-hungry configurations such as cloverleafs, multi-level interchanges, and other similar configurations will not be considered as relevant design alternatives.

LITERATURE REVIEW

The following review provides a conceptual overview of the use of grade separations along arterial streets. The review material used in this study placed little emphasis on concepts dealing exclusively with design. Instead, material was chosen if it offered conceptual ideas that could be used as a basis for formulating guidelines oriented towards the improvement of arterial streets. These proposed guidelines could be used for justifying grade separations at surface intersections. In general the reference material is meant to address the question, "When are grade separations needed or justified for an arterial intersection?" The review is organized according to issues related to this study and includes the following topics: (1) flyovers; (2) diamond-type interchanges; (3) strategic arterial concepts; (4) geometric design considerations; and (5) other general information.

Flyovers

Flyovers are widely used to alleviate traffic congestion in many European cities, but according to Pleasants [23], design standards imposed by the highway establishments in the United States have severely limited the construction of such interchanges. He emphasizes that American grade separation structures are built with heavy-weight, high-speed trucks in mind, while on the other hand European flyovers are typically meant to alleviate automobile traffic exclusively. With this in mind and the fact that flyovers; use minimal right-of-way, require very little installation time, and have the potential to reduce energy consumption and car emissions [12], Pleasants suggests that flyovers are a viable solution for American arterials.

In the same article Bagon [4] outlines many design aspects of his flyover bridge built in Brussels, Belgium (i.e. the AB-1 bridge completed in February of 1975.) Although this article is oriented specifically towards bridge design, it does point out that flyovers can be constructed quickly, thereby reducing the potential interference with traffic operations. Also, since the bridge deck is "prefabricated", it has the potential to be dismantled and replaced with a larger deck. Bridges of this nature have been described by Koger as being "fast-assembly bridges" [18]. His report, like Bagons', describes many design features of the bridge and sites an example that spans the Aegidientorplatz in Hannover.

Bonilla [7] examines the following design considerations for flyovers: The minimum cross section for a given right-of-way, the at-grade treatments, the traffic capacity, the structures length, the intersection geometrics, the cost-effectiveness of construction, and general warrants for flyover construction. Overall it is pointed out that the implementation of flyovers becomes cost effective when less expensive at-grade solutions have been exhausted [6, 7].

In a similar report from the Texas Transportation Institute Bonilla and Urbanik [6] demonstrate that the capacity of congested arterials can be increased in a cost effective manner through the use of grade separation. This was shown by relating flyover benefits to average approach volumes of the current plus 20 year forecast. Flyover benefits were shown to be dependent on the amount of traffic diverted to the flyover and the ability of the improved intersection to process the remaining at-grade traffic. The report also identifies operational considerations, proposes warranting conditions, and suggests implementation guidelines for the flyover development .

Haefner [12] illustrates the traffic engineering efficiency of a flyover by comparing the capacity of an at-grade signalized intersection to that of the same intersection with a flyover installed.

The positive and negative aspects of using flyovers to solve traffic problems is discussed by Byington [8]. The following are issues included in his study: The design of flyovers as either temporary or permanent structures, traffic capacity, safety considerations, esthetic and environmental issues, right-of-way design characteristics, cost elements, construction impacts, and the cost-effectiveness of flyovers in relation to other traffic control measures. The intent of this study was to provide the decision-makers with both the advantages and disadvantages of flyover construction, thereby aiding in the decision process needed in selecting an appropriate traffic control scheme.

Diamond-Type Interchanges

As engineers gained more experience with the operational characteristics of the interstate system during the 1960s and early 70s, it was recognized that improved interchange geometrics and signalization offered substantial increases in both safety and capacity. Therefore, Leisch [20] points out that engineers began refining existing diamond interchanges and, through their acquired observations, designed new diamond-type forms. As a result, three new types of diamond interchanges have evolved: the single-point diamond or "urban interchange", the three-point diamond, and the stacked diamond. The report provides an informal description of each interchange and discusses appropriate uses for each in an urban environment.

In a similar, but more detailed, report Leisch et. al. [21] provide a comparison between a compressed diamond and a single-point diamond. The study compares a variety of general characteristics for both forms. Characteristics such as geometrics, signal operations, bridge design, right-of-way, construction costs, and future adaptability, are included in this study. Additionally the relative efficiency, capacity, and level-of-service for each form were compared. In general it was determined that the compressed diamond was less costly, had similar right-of-way requirements, and was more efficient than the single-point diamond. Overall the study provided a comprehensive understanding of the characteristics and applications for both forms.

In a report prepared by Hernandez-Echavez [14], the operational characteristics of various interchange forms were evaluated, current geometric design guidelines were reviewed, and minimum and desirable geometric standards were provided. The study focused on diamond-type forms for urban arterial streets with restricted right-of-way. Specifically, the study evaluated at-grade intersections, compressed diamonds, and single-point diamonds through the use of the TEXAS Model (i.e. a microscopic simulation model for intersection traffic).

The use of urban interchanges as a means of alleviating congestion at bottleneck intersections is discussed in a report by Hawkes [13]. When compared to a conventional diamond interchange, he notes that an urban interchange accommodates greater traffic volumes, provides more safety, and demands less right-of-way. Unfortunately, his discussion is limited exclusively to the advantages of urban interchanges and, unlike Leisch et. al. [21], he never points out the drawbacks.

The potential benefits provided by the implementation of an urban interchange were evaluated in a study by Witkowski [30]. In this study a procedure for calculating user benefits of highway improvement (i.e. reduction in vehicle operating cost, travel time, and accidents) were demonstrated for use in sketch planning. The specific urban interchange evaluated in the study conservatively provided significant economic benefits as a replacement to the existing at-grade intersection. Overall this evaluation was intended to guide decision-making and planning for projects similar to the case study.

Two warrants for grade separation, taken from AASHTO's policy manual [3], were examined, in a report by Agent [1]. These warrants dealt with the elimination of hazards and road-user benefits. Overall it was found that the only solution for intersections with a considerable number of traffic accidents was through the provision of access control and interchanges. Also, for intersections with relatively high traffic volumes it is economically justifiable to build an interchange based on a benefit-to-cost analysis.

Rymer and Urbanik [25] propose a methodology meant to assist decision-makers in determining when grade separation is warranted. This methodology was based on benefit-cost analysis and on delay savings. They developed a single delay equation that estimates the vehicle delay incurred on signalized, at-grade portions of intersections and diamond interchanges. This equation was used in an economic analysis to determine if the reduced delay benefits to drivers would offset the grade separation structure construction cost.

In a similar report, Sargious and Tam [26] developed an equation for calculating the reduction in delay caused by the introduction of a diamond interchange to a standard at-grade intersection. Only the approaching volumes and through lane counts were required for calculating the delay savings. The report points out that by comparing the potential benefit from delay savings with the cost of constructing an interchange, an estimate can be made of the traffic volume above which grade separation is warranted.

Strategic Arterial Concepts

The TRANSYT-7F computer simulation model was used by Fitzpatrick [10] to evaluate improvements to an existing arterial street in Houston, Texas. High speeds and reduced delay can be obtained by judicious use of tum prohibitions, signal spacing, and grade separation. It was noted that since each arterial is unique, different combinations of improvements must be evaluated to determine what combination provides the optimal service.

A modified version of the TRANSYT-7F computer simulation model was also used by Recker [24] to analyze an arterial in the Greater Los Angeles area. As a result it was discovered that the use of flyovers, in conjunction with the optimization of signals, can substantially reduce vehicle stops and travel delays along heavily congested traffic corridors.

Ward [29] proposes a conceptual system of improved arterial streets for Harris County, Texas. Based on computer simulation it was shown that such a conceptual system would divert a significant amount of traffic from both freeways and other arterials. Factors such as, median separated roadways, the elimination of left-turns, grade separations, partial access control, 40 to 50 mph design speeds, and the priority treatment of strategic arterials at infrequently spaced signalized intersections were all proposed to be included in the conceptual geometric design and operational scheme of strategic arterials. The prioritization of route selection in anticipation of future urban growth and the acquisition of adequate amounts of right-of-way are critical factors in establishing such arterials.

In a study performed by McShane and Pignataro [22], a variety of guidelines for the treatment of traffic congestion and saturation on arterial street networks were examined. The study outlines each guideline, recommends a method for approaching the problem, and provides a framework for addressing the problem. The framework focuses on indentifying the problem in terms of its preliminary cause and it categorizes the available treatments. Recommended treatments such as optimizing signalization and the provision for added space (i.e. bays, lanes, and the like) tend to have the greatest impact. The guidelines produced in this study provide the traffic engineer with both a tutorial and an illustrated reference for what techniques to consider and how to systematically consider them.

The concept of strategic arterials is defined and evaluated in the doctoral dissertation prepared by Kruger [19]. In this study the current needs and trends in urban mobility are discussed and guidelines were proposed for the development of strategic arterials. A majority of the study concentrates on defining the classification, the operational characteristics, the appropriate design standards, and the planning aspects of strategic arterials. The remaining portion of the study examines the required standards for geometric design and outlines guidelines for the treatment of access control, curb-side activity, pedestrian activity, and implementation issues (i.e. analyzing operational characteristics of candidate streets and establishing an hierarchy of needed improvements.)

Geometric Design Considerations

The American Association of State Highway and Transportation Officials (AASHTO) provides a comprehensive policy on the geometric design of highway facilities in their 1990 edition of the "Green Book" [3]. Their text provides guidance to the design engineer by offering a recommended range of values for the dimensioning of various geometric features. Guidelines are provided for freeways, arterials, collectors, and local roads, in both rural and urban environments. The intent of these guidelines is to provide the motorist a consistent and familiar roadway with operational efficiency, comfort, safety, convenience, and a consideration for environmental impacts and esthetic consistency. These policy guidelines are referenced in considerable detail in both Chapters 2 and 3 of this report.

A multitude of general transportation issues are provided in the text by Garber and Hoel [11]. Topics such as traffic operations, traffic planning, geometric design, and pavement design are included within this text. Many of the same topics and fundamental principles are presented in a manual by Homburger and Kell [15]. The combination of these two references provided a sufficient amount of background information on the fundamentals of Transportation Engineering. Generally both sources were used to provide a means of referencing concepts or terms. Likewise, the *Transportation and Traffic Engineering Handbook* [16] provided a substantial amount of information on various transportation issues and design concepts.

General Information

The *Highway Capacity Manual* [28] discusses a variety of fundamental operational concepts pertaining to the flow of traffic for a variety of roadway facilities. Therefore, this provided a basic understanding of traffic flow and how physical changes to the traffic stream could either improve or hinder the overall flow.

A variety of travel demand forecasts were performed by CRSS [9] to develop a transportation plan for the Austin Metropolitan Area. Short-term, mid-term, and long-term growth expectations for both population and employment were investigated. These growth projections formed a basis for the forecasting and analysis of travel demand. The information contained in this report was used in this study to help predict the future volumes for a select group of arterial intersections in Austin.

The benefit analysis of Chapter 5 required the use of a variety of sources. An invaluable source was provided by AASHTO in their *Manual on User Benefit Analysis of Highway and Bus Transit Improvements* [2]. This manual provided cost factors, nomographs, and guidelines for estimating the economic effects on roadway and transit users by highway and bus-transit improvement projects.

The reports by Ismart [17], Rymer [25], Witkowski [30], and Zaniewski et. al. [31] were the primary sources used for the energy analysis of Chapter 5. This analyis estimated the excess fuel consumption for each intersection based on delay reductions.

OVERVIEW OF REPORT

In this chapter it was suggested that grade separation may be the only viable solution to mobility problems along arterial streets once all surface treatments have been exhausted. However, this raises the question of whether an improvement such as grade separation is warranted for a given intersection. Therefore, the following chapters provide a discussion of concepts related to the design and use of grade separations

A variety of fundamental geometrical design concepts are presented in Chapter 2. The purpose of this chapter is to provide a basic understanding of geometric design and fundamental considerations for design controls and criteria. A significant portion of this material was taken from AASHTO's geometric design policy manual [3], and the *Transportation and Traffic Engineering Handbook* [16].

An overview of concepts related to the geometric design of grade-separated interchanges, specifically diamond-type forms, are presented in Chapter 3. Also, the operation of such interchanges are discussed in some detail. The intent of this chapter is to provide a basic understanding of the general considerations necessary for the design of grade separations in an urban environment.

Generalized warrants that are currently used as a basis for justifying grade separations are discussed in some detail in Chapter 4. Also, four methodologies that could be used for justifying grade separations are presented for the readers consideration. Elements of these methodologies were used in the benefit analysis of Chapter 5.

A case study analysis of several intersections along an existing arterial street is provided in Chapter 5. The purpose of this analysis is to determine whether or not grade-separated interchanges are warranted for a given surface intersection based on an evaluation of benefits and costs.

Finally, Chapter 6 provides various concluding remarks and recommendations, based primarily on the results obtained in the benefit analysis that is described in Chapter 5.

CHAPTER 2
GEOMETRIC DESIGN CONSIDERATIONS

INTRODUCTION

This chapter provides a discussion of fundamental aspects of geometric design that act as criteria for the construction of roadway facilities. These characteristics include considerations for vehicles, pedestrians, and traffic operations. Topics covered include: design controls and criteria, horizontal and vertical alignment, and a variety of cross-sectional elements of the roadway. These topics are outlined in considerable detail in AASHTO's policy manual [3].

DESIGN CONTROLS AND CRITERIA

This section discusses basic controls and criteria that are employed in the design of roadway facilities. The characteristics of vehicles and traffic are the primary criteria used to ensure that a given facility will (1) accommodate the expected traffic demands; and (2) encourage consistency and uniformity in traffic operations. For the most part, these controls and criteria can be applied to all highway and street functional classes.

Access Control

Access control is the condition in which a public authority regulates or controls public access rights to and from properties adjacent to a highway or roadway system. These regulations can be described as either providing fully controlled access, or partially controlled access.

Fully-controlled access is the condition in which preference is provided for the through moving lanes of traffic on a given roadway. This is accomplished by limiting the number of access connections to public roads. This control limitation is provided through the prohibition of all at-grade crossings and the prohibition of direct access to private driveway connections.

Partially-controlled access is the condition in which preference is provided for the through moving lanes of traffic to a degree that, in addition to allowing limited access with select public roads, some at-grade and private driveway connections may be present along the roadway.

Design Vehicles

Elements essential for the geometric design of various highway facilities are the physical and operational characteristics of motor vehicles. Given the wide variety of vehicle types, it is necessary to group vehicles into general classes of comparable design. Vehicles can be classified into three general classes, namely, passenger cars, trucks, and buses or recreational vehicles [3]. Within each of these classes, representatively sized vehicles are established for design purposes. In the construction of a specific highway facility the largest of these vehicles, anticipated to use the facility, would govern design.

Tables 2-1 and 2-2 provide vehicle dimensions and turning radii respectively for a select group of commonly used design vehicles. These physical characteristics are used to design elements such as, lane and shoulder widths, parking bays and garages, sight distances, vertical curves, gradients, and a variety of other geometric features. Also, the design values for turning radii are typically used in the geometric design of outside pavement edges. Note that these design values are only for turns made at less than 10 mph. Higher speeds require radii greater than the indicated minimum.

TABLE 2-1. DESIGN VEHICLE DIMENSIONS

Design Vehicle		Dimensions (ft)						
		Overall			Overhang		Wheelbase	
Type	Symbol	Height	Width	Length	Front	Rear	WB1	WB2
Passenger Car	P	4.25	7.0	19	3	5	11	-
Single-Unit Truck	SU	13.50	8.5	30	4	6	20	-
Single-Unit Bus	BUS	13.50	8.5	40	7	8	25	-
Intermediate Semi	WB-40	13.50	8.5	50	4	6	13	27
Large Semitrailer	WB-50	13.50	8.5	55	3	2	20	30

Source: Ref. 3, Table II-1.

TABLE 2-2. MINIMUM TURNING RADII OF DESIGN VEHICLES

Design Vehicle		Design Values (ft)	
Type	Symbol	Minimum Turning Radius	Minimum Inside Radius
Passenger Car	P	24	13.8
Single-Unit Truck	SU	42	27.8
Single-Unit Bus	BUS	42	24.4
Intermediate Semitrailer	WB-40	40	18.9
Large Semitrailer	WB-50	45	19.2

Source: Ref. 3, Table II-2.

Design Speed

The design speed represents the maximum safe speed a vehicle can maintain over a given segment of highway when conditions are so ideally suited for the roadway that the surrounding design elements govern [3]. Design elements which are directly related to, and vary considerably with design speed, include superelevation, curvature, sight distances, and gradients. Other elements that are not directly related to design speed, but nonetheless influence the speed of the vehicle, include lane and shoulder widths, and general clearances to obstructions. Therefore, nearly all geometric features of roadway facilities are affected by the design speed.

The selection of design speed is influenced by the type of terrain, by the density and type of land use, by the type and purpose of the roadway, by the expected traffic volumes, and by economic and environmental considerations. Table 2-3 provides typical minimum design speeds for various types of highways.

Design Volume

The design volume is defined as a volume of traffic estimated for design purposes, and it represents the amount of traffic expected to use a given roadway facility during the design year

TABLE 2-3. TYPICAL MINIMUM DESIGN SPEEDS FOR VARIOUS TYPES OF HIGHWAYS (MPH)

Freeways

Terrain	Rural	Urban
Level	70	50
Rolling	60	50
Mountainous	50	50

	Local Roads and Streets (Rural)				Arterial Highways (Rural)	
Terrain	Current ADT < 50	Current ADT < 250	Current ADT 250-400	Current ADT ADT > 400 DHV > 100	Current ADT 50-750 DHV < 200	DHV ≥ 200
Level	30	30	40	50	50	70
Rolling	20	30	30	40	40	60
Mountainous	20	20	20	30	30	40

Arterial Highways

Urban	Suburban
30-40 mph for all types of terrain and for all traffic volumes.	40-50 mph for all types of terrain and for all traffic volumes.

Local Roads and Streets (Urban)

Collector Streets	Local Streets
30-40 mph for all types of terrain and for all traffic volumes.	20-30 mph for all types of terrain and for all traffic volumes.

Source: Ref. 16, Table 19-1.

(typically 10 to 20 years ahead) [3]. The design volume is used to determine various geometric design elements such as roadway widths and to a large extent, it determines the type of facility. Using the current or estimated average daily traffic (ADT), design volumes can be determined. In general this is done by projecting the ADT to some future year beyond the estimated time of construction, generally 5 to 20 years. Then the design hourly volume (DHV) and the directional design hourly volume (DDHV) can be established. The DHV represents a percentage of the expected ADT, where the 30th highest hourly volume is usually selected for design purposes. The DDHV is found by multiplying the DHV by the directional distribution of the traffic at the design hour. Design volumes are an invaluable resource in capacity analysis and in evaluating the performance of design alternatives such as grade separations.

HORIZONTAL ALIGNMENT

Horizontal alignment is comprised of tangents and horizontal curves, where horizontal curves are circular curves with a constant radius, and the tangents to these curves are typically connected with transitions (i.e. compound, single circular, or spiral curves). As mentioned in Section 2.2.3, design speed is a critical factor in the design of highway curves. The design of such curves requires an understanding of the relationships between speed and curvature, specifically relations with superelevation and side friction. The foundation for these relations come from the laws of mechanics however, the actual values used in design are dependent on practical limits and factors determined empirically over a range of given variables. The following is the basic formula used for designing horizontal circular curves:

$$e + f = \frac{V^2}{15R} \qquad \text{(Eq. 2.1)}$$

where
- e = rate of roadway superelevation, (ft/ft),
- f = side friction factor,
- V = vehicle speed, (mph),
- R = radius of curve, (ft).

Superelevation

Horizontal curves on roadway facilities are generally banked (i.e. inclined laterally upwards from the inside edge) in order to countervail centrifugal forces, and thus steering effort is minimized and driver comfort and safety is maintained. Maximum superelevation rates are governed by several factors, each of which may vary considerably. The frequency and amount of snow and ice, the type of area (i.e. rural or urban), and the frequency of slow moving vehicles are all factors which limit the rate of superelevation. Maximum superelevation rates vary upwards to 0.10, sometimes 0.12, for highways in rural areas with no snow or ice. In areas where the prevailing conditions include snow and ice, maximum superelevation rates of 0.08 to 0.10 are common. In general 0.08 is used in most rural highway design. Superelevation is generally not used in the design of urban streets except where speeds and terrain warrant their use.

TABLE 2-4. MAXIMUM DEGREE OF CURVE AND MINIMUM RADIUS FOR DESIGN

	Design Speed								
mph	20	30	40	50	60	65	70	75	80
km/h	(35)	(50)	(65)	(80)	(95)	(105)	(110)	(120)	(130)
Max. f	0.17	0.16	0.15	0.14	0.12	0.11	0.10	0.09	0.08
Max e	Minimum Radius (ft)								
0.04	127	302	573	955	1528	-	-	-	-
0.06	116	273	509	849	1348	1637	2083	2546	3274
0.08	107	252	468	764	1206	1528	1910	2292	2865
0.10	99	231	432	694	1091	1348	1637	2083	2546
	Maximum Degree of Curve								
0.04	45.0	19.0	10.0	6.0	3.75	-	-	-	-
0.06	49.25	21.0	11.25	6.75	4.25	3.5	2.75	2.25	1.75
0.08	53.5	22.75	12.25	7.5	4.75	3.75	3.0	2.5	2.0
0.10	58.0	24.75	13.25	8.25	5.25	4.25	3.5	2.75	2.25

Source: Ref. 16, Table 19-11.

Side Friction Factor

The other factor that influences horizontal curves is side friction. The upper limit of this factor is that at which the tire is skidding or at a point where a skid is inevitable. However, because there is a margin of safety and comfort used in the design of roadway facilities, the side friction factor is substantially less than that of the impending skid. Table 2-4 provides the maximum degree of curvature and the minimum radii associated with design speeds, maximum superelevation rates, and maximum side friction factors.

In addition to these controls, horizontal curvature can also be influenced by sight obstructions. In cases where a sight obstruction cannot be easily removed from the roadway, the provision for sight distance may become a controlling factor.

Provisions for Sight Distance

The provision of adequate visibility along a roadway is critical in the design of roadway facilities. Sight distance is the minimum length of roadway visible to the driver which allows all but the few fastest drivers to safely stop before reaching, or colliding with, a given object. Stopping sight distances are comprised of two distances, namely the distance traveled during the perception-reaction time of the driver and the distance traveled during braking. Along arterial streets where speeds are typically between 30 and 50 mph, safe stopping sight distances used for design range between 200 and 475 feet respectively. The following equation is used to determine stopping sight distances (SSD):

$$SSD = 1.47Vt + \frac{V^2}{30(f \pm g)} \qquad \text{(Eq. 2.2)}$$

where
- V = speed from which stop is made, (mph),
- t = perception-reaction time in sec, (2.5 sec is typical for rural design),
- f = coefficient of friction, (wet pavement, locked wheel used for design),
- g = percent of grade divided by 100, (where upgrades are added and, downgrades are subtracted).

SSD requirements not only can determine horizontal curvature but, they can also dictate the minimum lengths of vertical curves.

VERTICAL ALIGNMENT

The vertical alignment of a roadway is comprised of gradual changes in the grade of tangent lines of the approaches to a curve, and of lengths of vertical curves. In general vertical curves are parabolic in design and therefore, the analysis of such curves are based on basic mathematical properties of parabolas. In an urban environment vertical curves are commonly used in the grade separation of conflicting roadways. Two types of vertical curves are used in highway design, namely crest vertical curves and sag vertical curves. Figure 2-1 shows the general layout of both types.

Crest Vertical Curves

Two cases need to be considered when designing the minimum length of vertical curves. The first case is where the sight distance is less than the length of the vertical curve, and the second case is where the sight distance is greater than the vertical curve length. Case 1 represents a preferred design alternative and should be incorporated into design whenever possible. Both cases are provided by Equations 2.3a and 2.3b respectively. These equations are used to find the lengths of parabolic crest vertical curves in terms of sight distance and the algebraic difference in grade.

$$\text{Case 1: } (S < L); \qquad L_C = \frac{AS^2}{200\left(\sqrt{h_e} + \sqrt{h_o}\right)^2} \qquad \text{(Eq. 2.3a)}$$

$$\text{Case 2: } (S > L); \qquad L_C = 2S - \frac{200\left(\sqrt{h_e} + \sqrt{h_o}\right)^2}{A} \qquad \text{(Eq. 2.3b)}$$

where S = sight distance, (ft),

$L_{C,S}$ = length of crest or sag vertical curve, (ft),

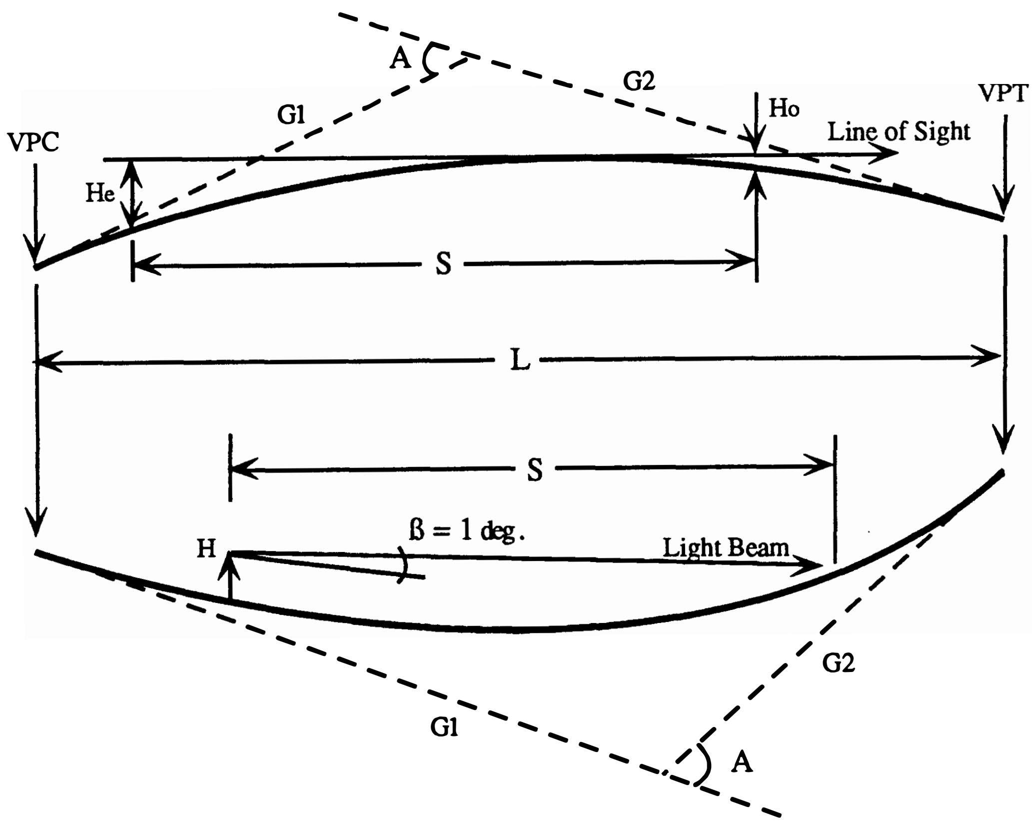
A = algebraic difference in grades, (%),

h_e = height of driver's eye in ft, (3.5 ft. is typical for design),

h_o = height of object in roadway in ft, (6 in. is typical for design).

(Note: The variables in Equation 2.3 are applicable for Equations 2.3 through 2.7)

Through the use of a factor which characterizes the rate of change of vertical curvature, the length of vertical curves and the difference in tangent grades can be related to the sight



L	=	length of vertical curve, (ft).	VPC	=	vertical point of curvature.
S	=	sight distance, (ft).	VPT	=	vertical point of tangency.
H	=	height of vehicle headlights, (ft).	G1	=	slope of 1st tangent, (%).
H_e	=	height of driver's eye, (ft).	G2	=	slope of 2nd tangent, (%).
H_o	=	height of object in roadway, (ft).	A	=	(G1 - G2)
ß	=	upward divergence of light beam, (deg).			

Figure 2-1. Sight Distances on Crest and Sag Vertical Curves.

Source: Adapted from Ref. 11, Figs. 3.8 and 3.9.

distance requirements. This K-factor represents the horizontal distance to effect a one percent change in grade, and is computed by the following formula:

$$K = \frac{L}{A} \qquad \text{(Eq. 2.4)}$$

Smaller values of K are indicative of sharper curves and consequently, limited visibility (i.e. poorer sight distances). In the design of new roadway facilities, sight distances should be compatible with the facility design speed. Appropriate K-values for crest and sag vertical curves, based on stopping sight distances, are provided in Table 2-5.

Sag Vertical Curves

The primary criteria used to determine the minimum lengths of sag vertical curves include: (a) the sight distance provided by vehicle headlights; (b) driver comfort; (c) underpass clearance; (d) drainage control; and (e) the overall appearance, or aesthetics.

a) Headlight Sight Distance. The headlight sight distance is based on a vehicle traveling on a sag vertical curve at night. The position and direction of the headlight beam determines the stretch of roadway that is visible (i.e. the distance that can be seen by the driver). The following two equations use the relationship of sight distance and headlight beams to determine sag vertical curve lengths. Generally a one degree upward divergence of the headlight beam, (β), and a two foot headlight height, (H), are used. Refer to Figure 2-1 for further reference.

Case 1: $(S < L)$;

$$L_S = \frac{S^2 A}{200(H + S\tan\beta)} \qquad \text{(Eq. 2.5a)}$$

Case 2: $(S > L)$;

$$L_S = 2S - \frac{200(H + S\tan\beta)}{A} \qquad \text{(Eq. 2.5b)}$$

b) Driver Comfort. In the design of sag vertical curves driver comfort is of greater concern than that for crest vertical curves. This is because the gravitational and centrifugal forces act in combination rather than as opposing forces. A sag vertical curve is said to be comfortable when the centrifugal acceleration does not exceed 1 ft/sec^2. The general expression for this criterion follows, where V is the design speed in mph [3]:

$$L_S = \frac{AV^2}{46.5} \qquad \text{(Eq. 2.6)}$$

TABLE 2-5.

DESIGN CONTROLS FOR CREST AND SAG VERTICAL CURVES BASED ON STOPPING SIGHT DISTANCE (SSD)

Design Speed (mph)	Assumed Speed for Condition (mph)	Coefficient of Friction f	SSD* Rounded for Design (ft)	Rate of Vertical Curvature, K [length (ft) per percent of A]			
				Crest Vertical Curves		Sag Vertical Curves	
				Computed**	Rounded for Design	Computed**	Rounded for Design
20	20 - 20	0.40	125 - 125	8.6 - 8.6	10 - 10	14.7 - 14.7	20 - 20
25	24 - 25	0.38	150 - 150	14.4 - 16.1	20 - 20	21.7 - 23.5	30 - 30
30	28 - 30	0.35	200 - 200	23.7 - 28.8	30 - 30	30.8 - 35.3	40 - 40
35	32 - 35	0.34	225 - 250	35.7 - 46.4	40 - 50	40.8 - 48.6	50 - 50
40	36 - 40	0.32	275 - 325	53.6 - 73.9	60 - 80	53.4 - 65.6	60 - 70
45	40 - 45	0.31	325 - 400	76.4 - 110.2	80 - 120	67.0 - 84.2	70 - 90
50	44 - 50	0.30	400 - 475	106.6 - 160.0	110 - 160	82.5 - 105.6	90 - 110
55	48 - 55	0.30	450 - 550	140.4 - 217.6	150 - 220	97.6 - 126.7	100 - 130
60	52 - 60	0.29	525 - 650	189.2 - 302.2	190 - 310	116.7 - 153.4	120 - 160
65	55 - 65	0.29	550 - 725	227.1 - 394.3	230 - 400	129.9 - 178.6	130 - 180
70	58 - 70	0.28	625 - 850	282.8 - 530.9	290 - 540	147.7 - 211.3	150 - 220

Source: Ref. 3, Table III-40 and III-42.
* Based on a 2.5 second perception-reaction time.
** Using computed values of stopping sight distance (SSD).

c) Underpass Clearance. In the common situation where a sag vertical curve is used in the design of an underpass, the clearances between a design vehicle and the upper bridge deck must be considered. This is because the upper bridge deck is considered a sight obstruction and thus, limits sight distance. The following two equations provide sag vertical curve lengths for underpasses, where C equals the vertical clearance in feet:

Case 1: $(S < L)$; $$L_S = \frac{S^2A}{800[C - 0.5(h_e + h_o)]} \quad \text{(Eq. 2.7a)}$$

Case 2: $(S > L)$; $$L_S = 2S - \frac{800[C - 0.5(h_e + h_o)]}{A} \quad \text{(Eq. 2.7b)}$$

d) Drainage Control. Drainage of curbed pavements on sag vertical curves, which are flatter than normal, requires careful profile design. The criteria used to avoid drainage difficulties for both crest and sag vertical curves is through the provision of a minimum grade of 0.30 percent within 50 feet of the high or low points respectively. This criteria relates to a maximum K-value of 167 [3].

e) Aesthetics. A general rule-of-thumb is employed when the appearance of the vertical curve is in question. The minimum value of L, recommended by AASHTO [3], is 100A. This represents a generalized expression for small or intermediate values of, A, the algebraic difference in grades.

Vertical Gradients

The selection of maximum grades for a roadway is dependent on the design speed and the design vehicle. In general grades of 4 to 5 percent have an insignificant effect on passenger cars, except for those with a high weight-to-horsepower ratio. When grades exceed 5 percent, however, the speed of passenger cars decrease on upgrades and increase on downgrades. Table 2-6 provides the maximum recommended grades for urban arterials.

The impact of grades on heavy vehicles such as semi-trucks is much more significant than that for passenger cars. Depending on the percent and length of grade, truck speeds can increase up to 5 percent on downgrades and can be reduced by as much as 7 percent on upgrades [11]. Since steep grades have the potential to affect vehicle speeds, it follows that the overall capacity can be decreased. Also, during adverse weather conditions vehicles can experience undesirable operational problems, especially at intersections. Therefore, it is desirable to provide the flattest possible grades practicable, while still allowing for proper drainage. In other words, maximum grades for any roadway should be selected judiciously.

TABLE 2-6. MAXIMUM GRADES FOR URBAN ARTERIALS

	Maximum Gradient (%)		
Design Speed (mph)	Level Terrain	Rolling Terrain	Mountainous Terrain
30	8	9	11
40	7	8	10
50	6	7	9
60	5	6	8

Source: Ref. 3, Table VII-4.

Minimum grades are dependent on the roadway drainage conditions. On uncurbed pavements with cross slopes that adequately drain surface water laterally, zero percent grades may be used. However, a longitudinal grade should be provided when pavements are curbed. This will facilitate the longitudinal surface flow of water. In such cases a minimum grade of 0.5 percent is ordinarily used. Although, for high-type pavements constructed on firm ground with a suitable crown, this grade may be reduced to 0.3 percent [11]. Cross slopes are discussed further in Section 2.5.1.

CROSS SECTIONAL ELEMENTS

The principal right-of-way of a given roadway is, in general, made up of the following: (1) traveled ways; (2) auxiliary lanes; (3) shoulders; (4) medians; (5) curbs; and (6) the bordering areas, or miscellaneous roadside elements. Table 2-7 provides minimum design standards for cross-sectional elements of urban streets. Further discussion of these design elements are provided in the following sections.

TABLE 2-7. CROSS-SECTION ELEMENTS DESIGN STANDARDS FOR STREETS

Cross-Sectional Design Element	Major Arterial	Collector LDR*	Collector Other	Local Road LDR*	Local Road Other
Number of Traffic Lanes	4-6	2	4	2	2-4
Width of Traffic Lanes, (ft)	10-12	10-11	11	9-11	11
Width of Turn Lanes, (ft)	11	-	9-10	-	9-10
Width of Parking Lanes, (ft)	10	7-8	10	7-8	10
Width of Bordering Areas, (ft)	12	10	8	5-10	8
Width of Median, (ft)	14-20	-	14-20	-	-
Width of Right-of-Way, (ft)	80-130	60	80	50-60	60-70

Source: Ref. 15, Table 19-3.
* LDR - Low-Density Residential areas.

Traveled Ways

The section of roadway that is designated for the movement of vehicles, exclusive of shoulders and auxiliary lanes, is considered the traveled way. Lane widths for freeways, expressways, and other highway systems are commonly at least 12 feet. However, in some cases it is necessary to use 11 ft traffic lanes in conjunction with reduced shoulder widths in order to accommodate an additional traffic lane. The minimum desirable width for local roads and streets is 11 feet, although for low traffic volumes with a minimal amount of trucks 9 and 10 ft traffic lanes are adequate [16].

In order to provide proper drainage on urban arterials, adequate cross slopes must be provided. Driver safety is considerably decreased when surface water does not drain properly. This reduced safety is because of the problems associated with splashing and hydroplaning, especially for heavy traffic volumes at intermediate to high speeds. Therefore, roads should be designed with cross slopes that range from 1.5 to 3 percent [3]. Center lanes typically have lower cross slopes than the outer lanes (i.e. cross slopes increase about 1 percent for each additional lane over which water must drain until a maximum cross slope of 3 percent is reached). However, the overall appearance of the cross section should be smoothly rounded and without any sharp breaks. As mentioned in Section 2.4.3, curbed arterials should have provisions for both longitudinal and cross slope drainage.

Auxiliary Lanes

Any section of roadway, adjoining the traveled way, that is used for a function additional to the through traffic movements is considered an auxiliary lane. The following are typical functions, or components, of auxiliary lanes: (a) tapers; (b) parking; (c) weaving sections; (d) speed changing lanes; (e) storage for turning vehicles; and (f) climbing lanes for steep grades.

a) Tapers. Tapered sections are an integral part of auxiliary lanes or of roadway segments that require the redirected alignment of a lane (i.e. exit/entrance ramps for grade-separated interchanges). The primary types of tapers include approach tapers, departure tapers, turning bay tapers, and lane-drop tapers.

b) Parking. In regions along the roadway where parking is permitted, an additional 10 or 12 ft should be provided. Parking spaces are typically marked 8 feet from the curb regardless of the available pavement width. The extra pavement width ensures the proper operation of the adjacent traffic lane. Parking lanes that are 10 feet or wider can be converted during the peak hours of roadway operation into a storage lane, a turning lane, a bus or high occupancy vehicle (HOV) lane, or an additional through traffic lane.

c) Weaving Sections. A weaving section is any segment of a roadway where vehicles entering or leaving at contiguous points of access results in vehicle paths that merge, diverge, or cross each other. Such sections are located within interchanges, between ramp terminals, and along sections of overlapping roadways.

d) Speed Changing Lanes. Any lane that is provided to allow vehicles entering or exiting the through traffic lanes to accelerate or decelerate respectively, is considered an auxiliary lane. Such lanes reduce potential interference in the through traffic movements.

e) Storage for Turning Vehicles. Any lane or lane group that is provided for the storage of turning vehicles (left or right) are considered auxiliary lanes. Such lanes, or bays, are typically the same width as through moving lanes and the determination of their length is primarily dependent upon vehicle speeds, turning percentages, and traffic volumes.

f) Climbing Lanes. In regions where there are steep sustained grades the operating speed of heavy vehicles can be significantly below that of passenger cars (See Section 2.4.3 and Table 2-6). Consequently, roadway capacities and driver safety may be significantly reduced, unless there is a provision for climbing lanes. However, the use of such lanes on urban arterials is rare.

Shoulders

In general, a shoulder is the section of roadway contiguous with the traveled way for lateral support of subbase, base, and surface courses. Shoulders are also used for improved sight distance, for driver comfort, for emergency use, for stopped vehicles, and for other special purposes (i.e. paths designated for pedestrians and cyclists [16]).

The width of shoulders varies considerably, from only 2 feet on minor rural roads to as much as 12 feet on major roads. However, shoulders cannot be provided in every urban region because of right-of-way limitations and the necessity of using available space for additional traffic lanes. When shoulders are provided however, they must be sloped sufficiently to drain surface water, but not to the extent that vehicular use would be hazardous. For materials common to an urban arterial (i.e. bituminous and concrete surfaces), shoulders should be sloped 2 to 6 percent.

Medians

A median is the region between the through-lane edges on two-way roads, including left shoulder edges, that separate the opposing traffic movements. In general, medians or central reserves can be categorized as follows [16, 19]:

1) Narrow, curbed, or raised sections; (4 to 6 feet wide; raised sections may consist of longitudinal, physical barriers such as concrete median barriers, and metal beam guard fences).
2) Painted separations; (2 to 4 feet wide).
3) Painted or curbed sections that provide space for left turn lanes/bays; (10 to 18 feet wide for a single lane, or 22 to 28 feet wide for double lanes).
4) Traversable or curbed sections that provide a protected, shielded space for vehicles crossing an intersection, and/or a space reserved for parkway landscape features; (20 to 40 feet wide).

Curbs

Curbs can be classified as either barrier curbs, or mountable curbs. Each classification has numerous types and design details. Along urban streets curbs can be provided for drainage control, for protection of pedestrians, for delineation, or to permit greater use of available right-of-way. Generally curbs are 4 to 9 inches in height, depending on drainage requirements, traffic control, and safety considerations.

Bordering Areas

The area between the roadway edge, or the frontage road if one exists, and the right-of-way line is considered the bordering area. This area does not include shoulders and should be wide enough to accommodate any elements necessary to the roadway such as cut/fill slopes, ditches, walls, bicycle paths/sidewalks, and any landscaped buffers. Along city streets this area should include space for the placement of utilities. For urban streets a bordering width of 4 to 8 feet in addition to the sidewalk width is desirable.

SUMMARY

This chapter provided a variety of geometric design elements relating to the design of arterial streets and highways. These elements included design controls and criteria, horizontal and vertical alignment, and various cross-sectional elements of the roadway. These basic design considerations can be used in the design of new roadway facilities, or in the evaluation and upgrade of existing ones.

The following chapter examines a variety of geometric elements and operational aspects used in the design of grade separations.

CHAPTER 3. DESIGN AND OPERATIONAL CONSIDERATIONS FOR GRADE-SEPARATED INTERCHANGES

INTRODUCTION

This chapter provides a variety of fundamental concepts related to the geometric design of grade-separated interchanges. Also, a section is provided for the discussion of basic interchange operations.

GEOMETRIC DESIGN CONSIDERATIONS

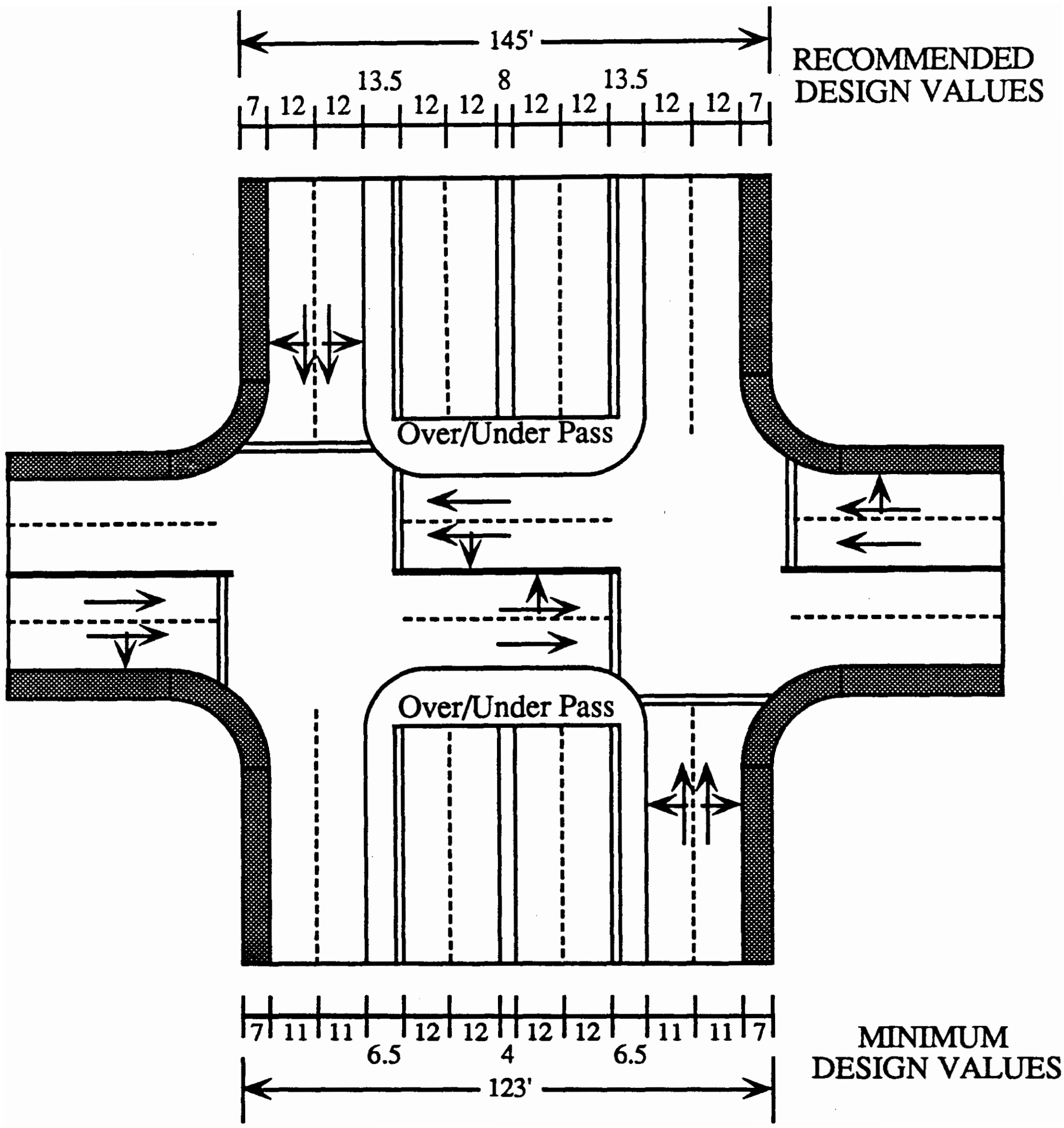
At-grade intersections that experience considerable problems with traffic congestion and/or accident frequency display a need for improvements that can accommodate high traffic demands. Grade separations can provide this need for increased capacity, safety, and efficiency [3]. Intersections that are grade-separated are commonly called interchanges. An interchange is any system of intersecting roadways in combination with one or more grade separations, where the provision for movement between two or more roadways is on different levels [3]. This section discusses basic considerations in their use and design.

General Configurations

Some of the most common interchanges include the trumpet, the cloverleaf, the directional, the diamond, and numerous combination-interchanges that incorporate more than one form. Because of the numerous varieties of interchange forms, it is inconceivable to discuss design elements for each interchange. Therefore, this report is limited to the discussion of simple diamond-type forms. The diamond form was chosen because of its adaptability along developed arterial streets where right-of-way is generally limited. Two diamond-type interchanges are commonly found along arterial streets, namely the compressed diamond and the single-point diamond (or urban interchange).

Compressed diamonds are very similar to conventional diamonds except that they require less right-of-way. In general, retaining walls are responsible for this minimal use of space. Consequently, the compressed form has little or no provision for the storage of vehicles between ramps. Figure 3-1 shows a generalized form of the compressed diamond.

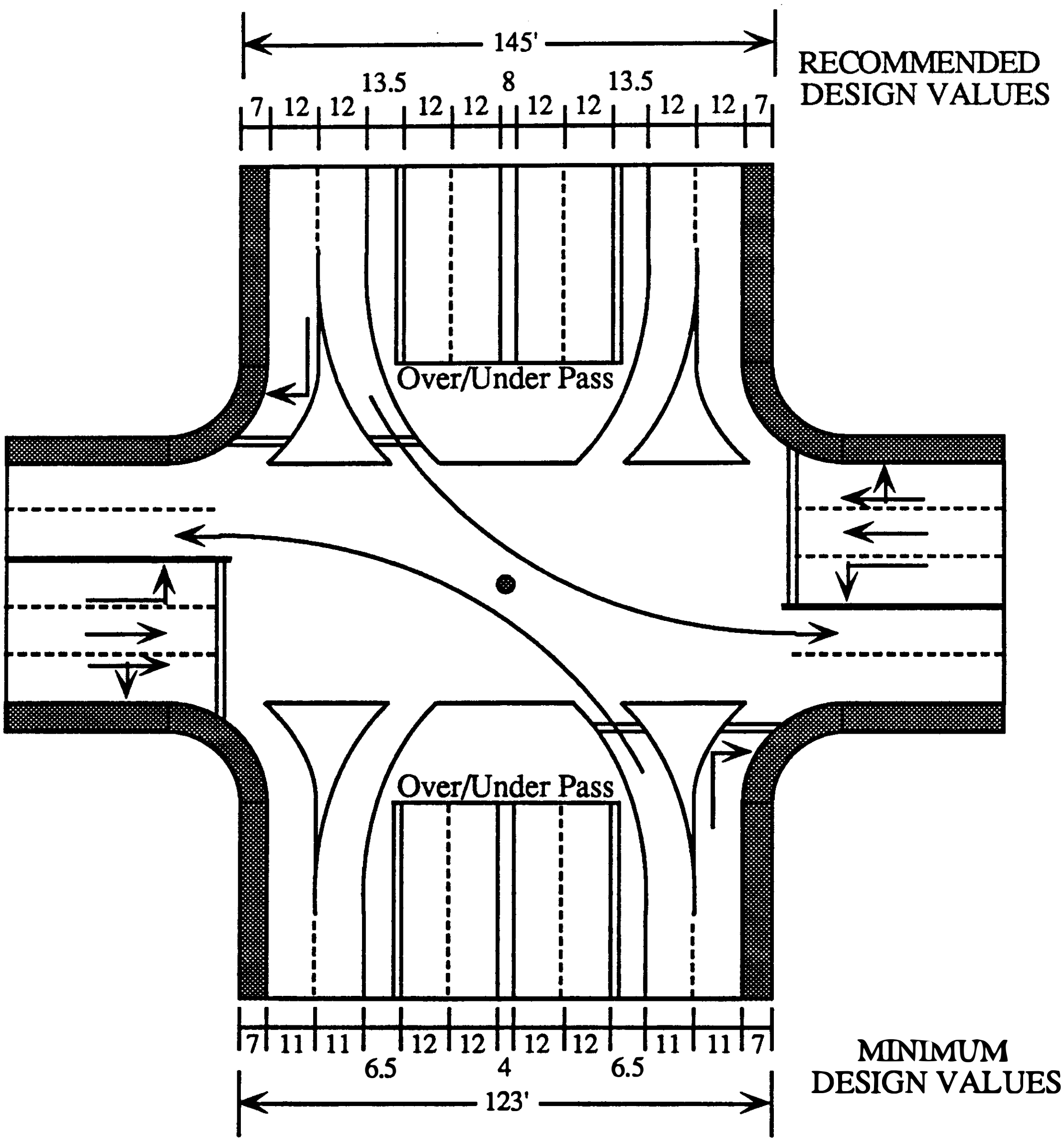
Single-point diamonds allow the simultaneous operation of left turning vehicles from each ramp [13]. This unique characteristic allows the single-point diamond to employ a three-phased signal instead of a four-phased signal found on conventional diamonds. However, three-phased



Note: Recommended and minimum design values taken from Table 3-1.

Figure 3-1. General Configuration and Right-of-Way Requirements for a Compressed Diamond Interchange.
Source: Adapted from Ref. 14, Fig. 2-8 and Fig. 3-2.

operations are only possible when there are no through lane movements on the ramps [21]. These phasing requirements will be discussed in more detail later. Figure 3-2 shows a generalized form of the single-point diamond.



Note: Recommended and minimum design values taken from Table 3-1.

Figure 3-2. General Configuration and Right-of-Way Requirements for a Single-Point Diamond Interchange. Source: Ref. 14, Figs. 2-8, and 3-4.

The following list describes geometric features of compressed and single-point diamonds and should further distinguish their differences. The list represents a summary of items discussed by Leisch et. al. [21].

1) Open pavement area requirements, (under similar conditions of high traffic volumes):
    a) Compressed diamonds; (125 by 100 feet),
    b) Single-point diamonds; (250 by 100 feet),
    c) Assumption: the two conflicting roadways intersect at or near right angles, (if this angle is much less than 90 degrees, then the distance between stop lines of a single-point diamond and the area of its open pavement increases considerably).
2) Left-turn lanes for the compressed form cannot be located opposite each other, whereas for the single-point diamond they can. (Potentially this can save either one or two lanes of width for the single-point design depending on the number of left-turn lanes).
3) Turning movement considerations:
    a) Compressed diamonds; left turns from the ramp stop bars are 80 to 90 degrees and are accommodated on 50 to 75 feet turning radii,
    b) Single-point diamonds; left turns from the ramp stop bars are 45 to 60 degrees, 90 degrees for left turns off the cross street, and all turns are accommodated on 50 to 75 feet turning radii,
    c) Note: to gain maximum lane utilization, adequate lateral clearances must be provided between opposing left turns. Also, values may vary as to conditions and specific geometrics.

Lateral Clearances

Clearance to roadside obstructions, or design elements, can influence capacity, vehicle running speeds, safety, and driver comfort. Features that are considered to be obstructions include curbs, median barriers, retaining walls, bridge piers, light poles, road signs, and any other element that may restrict lateral clearance. The overall importance in providing adequate clearance to obstructions is more apparent from the values contained in Table 3-1. This table shows the effect different lane widths and lateral clearances have on the capacity of a traffic lane. In the design of roadway facilities as much clearance as possible should be provided along the traveled ways. The following section provides grade separation right-of-way requirements and cross-sectional details.

TABLE 3-1. COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY

Usable Shoulder Width or Clearance to Obstruction (ft)	Capacity of Narrow Lanes with Restricted Lateral Clearance (Percent of Capacity of 12 ft Lane)*		
	12 ft Lanes	11 ft Lanes	10 ft Lanes
6	100	93	84
4	92	85	77
2	81	75	68
0	70	65	58
	Four-Lane Undivided (One Direction Travel - Obstruction One Side)		
6	100	95	89
4	98	94	88
2	95	92	86
0	88	85	80

Source: Ref. 3, Table IV-2.
* Assumptions: Uninterrupted flow, Level of Service B, High type pavement.

Right-of-Way Considerations

Because it is generally not cost-effective to purchase large quantities of land adjacent to an existing roadway, it is beneficial to find interchange configurations that are not land hungry. Typically diamond-type forms require less right-of-way than any other type of configuration, however this right-of-way will vary for each location depending on site conditions and traffic demands. This section provides recommended right-of-way requirements for diamond interchanges. Figures 3-1 and 3-2 show typical design values for the cross-section of a compressed diamond and a single-point diamond respectively. In most situations there is little difference in right-of-way requirements. Rarely would the additional right-of-way for the compressed versus the single-point exceed half an acre [21]. Table 3-2 provides recommended minimum design values for diamond interchange right-of-way.

TABLE 3-2. MINIMUM GRADE SEPARATION RIGHT-OF-WAY

	Minimum Right-of-Way, (ft)			
Ramp Terminals	Desirable Design		Minimum Design	
(number of lanes)	Four-Lane	Six-Lane	Four-Lane	Six-Lane
Two-Lane	145	169	123	147
Three-Lane	169	193	145	169

Source: Ref. 14, Table 2-14.

Since many urban arterial intersections do not have adequate space to provide the desirable clearances of grade-separated interchanges, Bonilla [7] suggests that trade-offs must be made in the lateral clearances of the grade-separated interchange. The resulting intersection, because of these trade-offs, can be described as either having marginal, low type, or high type clearances (as shown in Figure 3-3). Also, Table 3-3 provides the recommended minimum right-of-way for each type of interchange operating with two, four, and six lanes. Note the difference between design values in Tables 3-2 and 3-3.

TABLE 3-3. MINIMUM RIGHT-OF-WAY* FOR URBAN ARTERIAL FLYOVERS

Type	Minimum Right-of-Way*, (ft) (by number of grade separated lanes) Two-Lanes	Four-Lanes	Six-Lanes
Marginal	76	98	-
Low Type	100	120	140
High Type	120	144	168

Source: Adapted from Ref. 7, Table 1.
* Based on reductions made in lateral clearances (less than min. design standards of Table 3-1).

Grade separations with marginal right-of-way configurations (see Figure 3-3) have cross-sectional dimensions that approach an absolute minimum recommended width for urban arterials. These design widths are not recommended because the clearances do not meet lateral safety standards. However, marginal structures could be used for the following: (1) in extraordinary cases where its use is considered a temporary measure; (2) in cases where other measures would not satisfy specific needs; or (3) in cases where other measures would not be economically feasible [7].

Grade Separation Length

By using appropriate values for the required sight distance and the allowable maximum grade (i.e. values corresponding to an applicable design speed), the required horizontal length for an overpass or underpass can be calculated. Appropriate values for sight distance and maximum gradients were provided previously in Tables 2-5 and 2-6. Assuming that the layout of vertical curves are symmetrical (i.e. the approaches to the interchange are horizontal, gradients on both sides of the interchange are equal, and the vertical curve is oriented directly in the center of the interchange), the horizontal length of the elevated or depressed section can be calculated by Equation 3.1 [19]. This length is shown in Figure 3-4.

$$L = 2[G(K_S + K_C) + T] \qquad \text{(Eq. 3.1)}$$

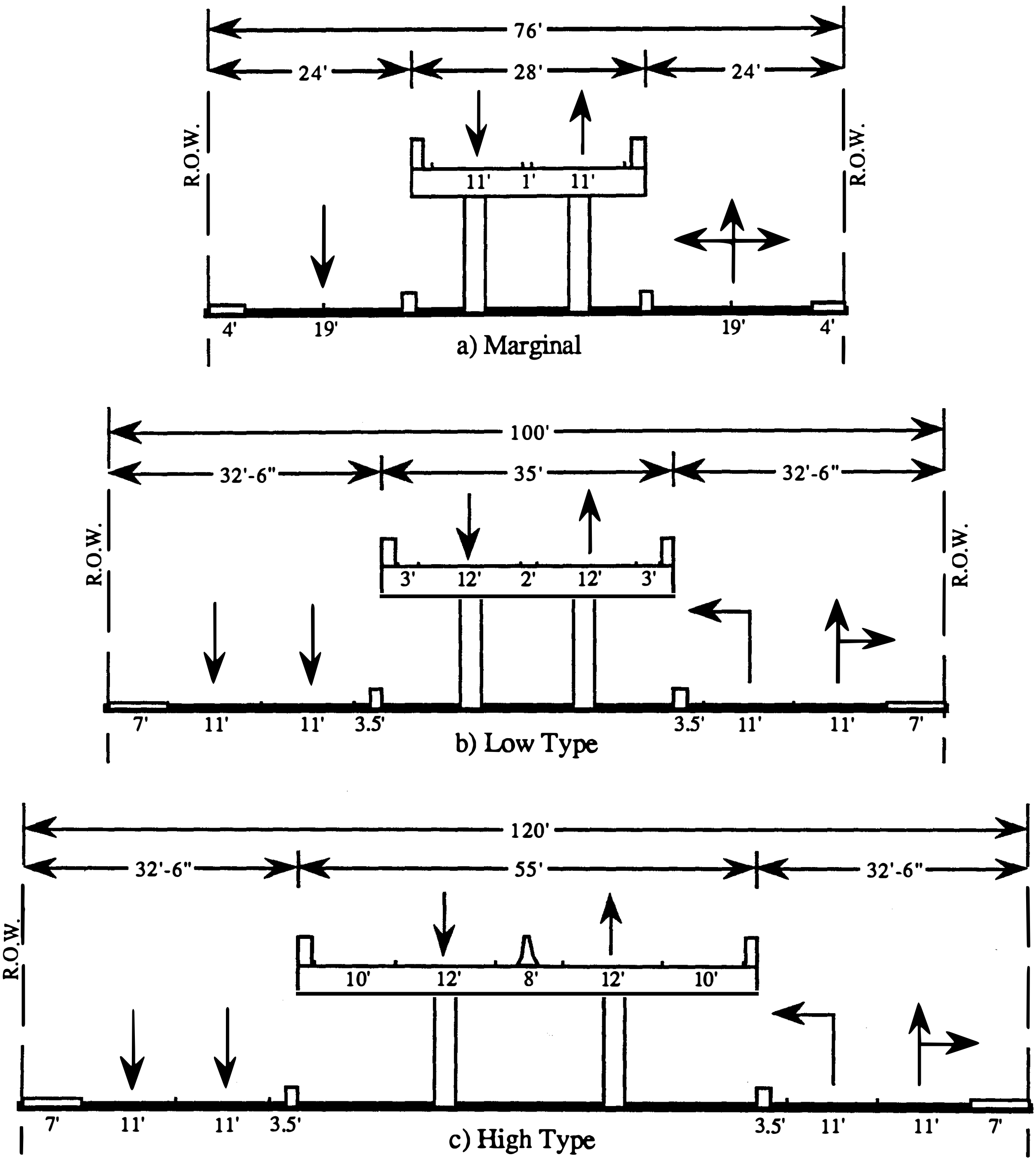
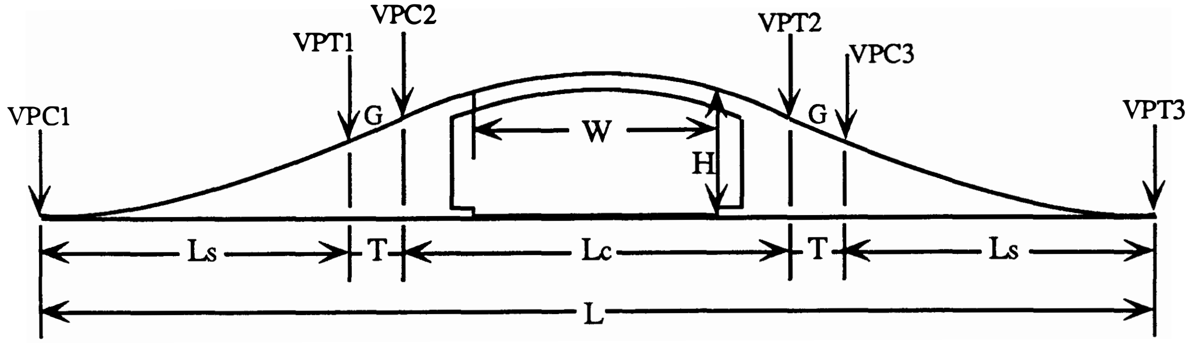


Figure 3-3. Minimum Cross Section and Right-of-Way for Two- Lane Flyovers.
Source: Ref. 7, Fig. 2.



L_S = length of sag vertical curve.
L_C = length of crest vertical curve.
G = gradient of approaches.
T = tangent distance between vertical curves, ($T \geq 0$).

VPC = vertical point of curvature.
VPT = vertical point of tangency.
H = minimum height.
W = width between minimum elevations.

Figure 3-4. General Layout of Vertical Curves at a Grade Separation. Source: Ref. 19, Fig. 5.21.

Note that T is the tangential distance between sequential vertical curves,

$$T = \frac{100H}{9} - \frac{G(K_S + K_C)}{2} + \frac{W^2}{8GK_i} \geq 0 \qquad \text{(Eq. 3.2)}$$

where $K_{C,S}$ = required rate of curvature for a crest or sag vertical curve,
K_i = K_C for an overpass, and K_S for an underpass,
G = gradient of approaches, (%),
W = width of required minimum elevation H, (ft),
H = required elevation above or below the level section at the location of the roadway clearance opening, (ft).

Tables 3-4 and Table 3-5 provide required overpass and underpass lengths for both lighted and unlighted facilities. The lengths in each table were derived using Equation 3.1. Note that consecutive vertical curves cannot overlap and therefore, there must be a tangential distance, T, of a length greater than or equal to zero. Also, the following recommendations, concerning the design of grade-separated interchanges and their associated vertical curves, are made by AASHTO [3]:

1) Vertical clearances:
   a) Absolute minimum; (14.5 feet),
   b) Desirable minimum; (16.5 feet, this includes a 6 inch allowance for future road resurfacing),
   c) Typical minimum clearances; (1.3 feet plus the maximum vehicle height allowed by state law).
2) Maximum gradients and design speed:
   a) 6 percent for a design speed of 40 mph,
   b) 5 percent for 50 mph,
   c) 4 percent for 60 mph.

Ramp Terminals

Ramp terminals are segments of roadway adjacent to the traveled way that provide a transition for through moving traffic. This transition is intended for merging, diverging, or turning maneuvers. Ramp terminals primarily consist of speed change lanes, tapers, and islands. Basic types of terminals include: (1) at-grade types, such as the cross street terminal of a diamond interchange; and (2) free-flow types, where ramp traffic either merges into or diverges from high speed through moving traffic at flat angles [3]. Terminals can be further classified as follows: (1) single or multilane, which refers to the number of lanes on the ramp; and (2) tapered or parallel, which refers to the configuration of the speed change lane.

Besides providing traffic with a transition zone, ramp terminals function as acceleration and deceleration lanes. Therefore, the length of exit ramps are governed by deceleration rates, and average running speeds for both the highway and the exit curve. Similarly, the length of entrance ramps are governed by acceleration rates, highway speeds, and initial speeds at entrance curves. Tables 3-6 and 3-7 provide minimum deceleration and acceleration lengths for exit and entrance terminals respectively.

TABLE 3-4. LENGTH REQUIRED FOR UNLIGHTED OVERPASSES AND UNDERPASSES FROM A FLAT GRADE

Min. Elev. (ft)	Design Speed (mph)	Max. Gradient (%)*	K_s	K_c	Suitable Gradient (%)*	Required Overpass Length, (ft) Width of Min. Elev., (ft) 50	100	150	200	Required Underpass Length, (ft) Width of Min. Elev., (ft) 50	100	150	200
15	40	6.0	70	80	4.0	1352	1358	1368	1381	1352	1359	1370	1386
	45	5.5	90	120	3.5	1594	1598	1606	1616	1594	1600	1610	1624
	50	5.0	110	160	3.0	1811	1815	1822	1831	1812	1818	1827	1840
	55	4.5	130	220	2.5	2076	2080	2085	2093	2077	2083	2092	2106
	60	4.0	160	310	2.5	2376	2378	2382	2388	2377	2381	2389	2400
20	40	6.0	70	80	5.0	1552	1556	1564	1575	1566	1572	1582	1596
	45	5.5	90	120	4.0	1841	1845	1852	1861	1842	1847	1856	1868
	50	5.0	110	160	3.5	2089	2092	2098	2106	2089	2094	2102	2114
	55	4.5	130	220	3.0	2384	2387	2392	2398	2385	2390	2398	2409
	60	4.0	160	310	2.5	2776	2778	2782	2788	2777	2781	2789	2800
25	40	6.0	70	80	5.0	1752	1756	1764	1775	1736	1741	1749	1760
	45	5.5	90	120	4.5	2057	2061	2067	2075	2058	2062	2070	2081
	50	5.0	110	160	4.0	2331	2334	2339	2346	2331	2336	2343	2353
	55	4.5	130	220	3.5	2654	2657	2661	2667	2655	2659	2666	2676
	60	4.0	160	310	3.0	3077	3079	3083	3087	3078	3082	3088	3098

Source: Ref. 19, Tables 5.5 and 5.6.
Note: K-Values set for 2.5 second brake reaction time, and stopping sight distances at sags are determined by headlight sight distance.
* Max. Gradients are recommended by AASHTO; Suitable Gradients are the grades required by vertical curve geometry to avoid overlapping.

TABLE 3-5. LENGTH REQUIRED FOR LIGHTED OVERPASSES AND UNDERPASSES FROM A FLAT GRADE

Min. Elev. (ft)	Design Speed (mph)	Max. Gradient (%)*	K_S	K_C	Suitable Gradient (%)*	Required Overpass Length, (ft) Width of Min. Elev., (ft) 50	100	150	200	Required Underpass Length, (ft) Width of Min. Elev., (ft) 50	100	150	200
15	40	6.0	40	50	5.5	1043	1050	1061	1077	1043	1052	1066	1086
	45	5.5	50	80	4.5	1253	1259	1267	1279	1254	1263	1277	1296
	50	5.0	60	120	4.0	1471	1475	1482	1491	1473	1480	1493	1512
	55	4.5	70	160	3.5	1663	1667	1672	1680	1665	1672	1685	1703
	60	4.0	80	230	3.0	1931	1934	1938	1944	1933	1940	1953	1972
20	40	6.0	40	50	6.0	1209	1215	1225	1240	1209	1217	1230	1248
	45	5.5	50	80	5.5	1444	1448	1455	1465	1445	1451	1463	1479
	50	5.0	60	120	4.5	1700	1704	1709	1717	1701	1708	1720	1736
	55	4.5	70	160	4.0	1921	1924	1929	1936	1922	1929	1940	1956
	60	4.0	80	230	3.5	2229	2231	2235	2240	2230	2237	2248	2264
25	40	6.0	40	50	6.0	1375	1382	1392	1407	1376	1384	1397	1415
	45	5.5	50	80	5.5	1626	1630	1637	1647	1626	1633	1645	1660
	50	5.0	60	120	5.0	1901	1904	1909	1917	1902	1908	1919	1933
	55	4.5	70	160	4.5	2147	2150	2154	2160	2148	2154	2164	2178
	60	4.0	80	230	4.0	2491	2493	2496	2501	2492	2498	2508	2521

Source: Ref. 19, Tables 5.7 and 5.8.
Note: K-Values set for 1.5 second brake reaction time, and stopping sight distances at sags are determined by comfort criterium.
* Max. Gradients are recommended by AASHTO; Suitable Gradients are the grades required by vertical curve geometry to avoid overlapping.

TABLE 3-6. MINIMUM DECELERATION LENGTHS FOR EXIT TERMINALS WITH FLAT GRADES OF 2 PERCENT OR LESS

		Deceleration Length, (ft) For Design Speed of Exit Curve, (mph)								
Highway Design Speed (mph)	Average Running Speed (mph)	Stop	15	20	25	30	35	40	45	50
		For Average Running Speed on Exit Curve, (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	185	160	140	-	-	-	-	-
40	36	315	295	265	235	185	155	-	-	-
50	44	435	405	385	355	315	285	225	175	-

Source: Ref. 3, Table X-6.

TABLE 3-7. MINIMUM ACCELERATION LENGTHS FOR ENTRANCE TERMINALS WITH FLAT GRADES OF 2 PERCENT OR LESS

		Acceleration Length, (ft) For Entrance Curve Design Speed, (mph)								
Highway Design Speed (mph)	Highway Speed Reached (mph)	Stop	15	20	25	30	35	40	45	50
		and Initial Speed, (mph)								
		0	14	18	22	26	30	36	40	44
30	23	190	-	-	-	-	-	-	-	-
40	31	380	320	250	220	140	-	-	-	-
50	39	760	700	630	580	500	380	160	-	-

Source: Ref. 3, Table X-4.

OPERATIONAL CONSIDERATIONS

Operational elements that should be considered for diamond-type interchanges may include signal phasing requirements, clearance intervals, saturation flow rates, turning movements, and signal timing. The following sections provide a discussion of these operational considerations.

Signal Phasing

The phasing operation for the double intersection of a compressed diamond typically consists of a four-phase (overlap) timing scheme. Essentially these two intersections operate as a single one. This scheme is shown in Figure 3-5.

The phasing operation for the single intersection of a single-point diamond typically consists of a three-phased plan (provided that there are no through movements from an exit terminal to an entrance terminal). The single-point diamond is an impractical design alternative when such movements are necessary. Therefore, single-point diamonds are generally not used when frontage roads are present. The three-phased operation is shown in Figure 3-6. Although the movement is not shown on Figure 3-6, single-point diamonds can be modified to accommodate independent U-turns [13].

Clearance Intervals

Given the distinct geometric differences between these two diamond-type forms, there is a difference in the required yellow clearance interval (generally 2 seconds per phase) [21]. Single-point diamonds require longer clearance intervals because of their larger than normal pavement area. Also, it is not uncommon to incorporate an additional all-red clearance interval in order to provided a margin-of-safety. Table 3-8 shows minimum clearance time requirements for given intersection widths and approach speeds. The values in this table can be derived from the following expression [11]:

$$Y = t + \frac{V}{2a} + \frac{W + L}{V} \qquad \text{(Eq. 3.3)}$$

where
- Y = clearance interval, (sec),
- t = driver's perception-reaction time, (sec),
- V = approach speed, (ft/sec),
- a = deceleration rate, (ft/sec^2),
- W = width of intersection, (ft),
- L = length of design vehicle, (ft).

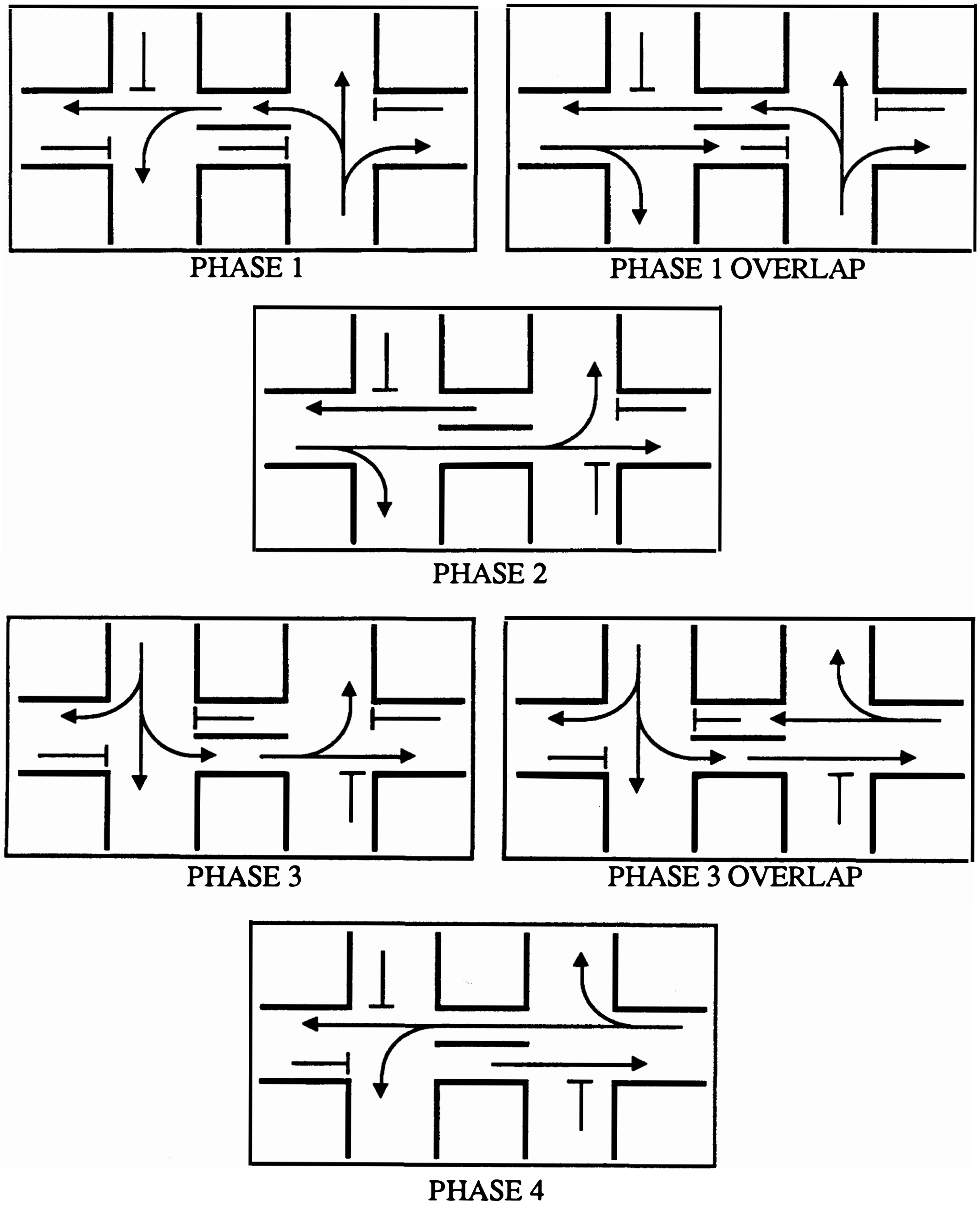


Figure 3-5. Signal Phasing for a Compressed Diamond Interchange.
Source: Adapted from Ref. 21, Fig. 5.

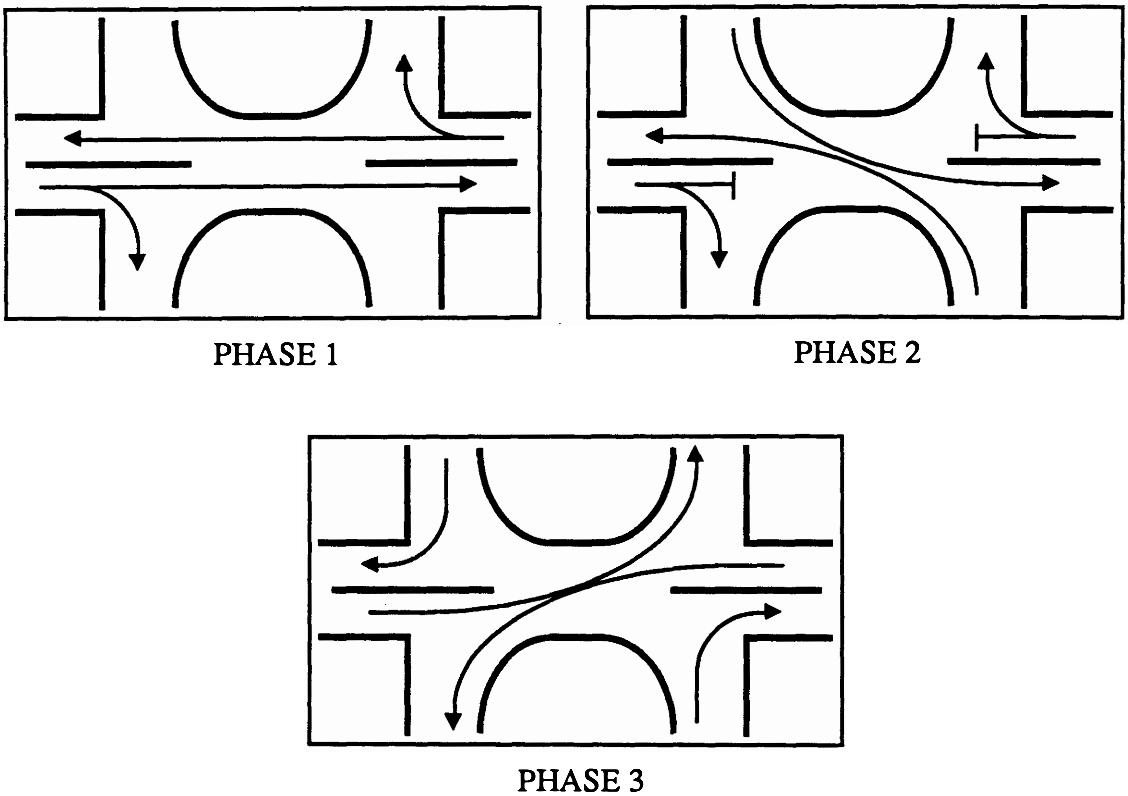


Figure 3-6. Signal Phasing for a Single-Point Diamond Interchange.
Source: Adapted from Ref. 21, Fig. 4.

TABLE 3-8. MINIMUM CLEARANCE INTERVAL REQUIREMENTS

Intersection Width (ft)	Clearance Interval, (seconds) Approach Speed, (mph)					
	20	25	30	35	40	45
50	4.9	4.7	4.8	4.9	5.1	5.4
75	5.7	5.4	5.4	5.4	5.5	5.7
100	6.6	6.1	5.9	5.9	6.0	6.1
150	8.3	7.5	7.1	6.9	6.8	6.9
175	9.2	8.2	7.6	7.4	7.3	7.3
200	10.0	8.9	8.2	7.9	7.7	7.6
225	10.9	9.5	8.8	8.4	8.1	8.0
250	11.8	10.2	9.3	8.8	8.6	8.4

Source: Ref 21, Table 1.

Clearance intervals in excess of 8 seconds are generally not desirable, however large intervals are often necessary for single-point operations [21]. Therefore, any operational advantage a single-point diamond has over a compressed form may be reduced considerably as the clearance intervals for single-points are increased.

Saturation Flow Rates

The saturation flow rate represents the maximum number of vehicles per hour per lane that can conceivably pass through an intersection when the green signal is available for an entire hour. This assumes that the average headway of every vehicle entering the intersection equals the saturation headway (i.e. the average minimum headway for a continuous and stable moving queue of vehicles, typically 2 sec/veh) [28]. The following equation can be used to calculate saturation flows [28]:

$$S = (S_o)(N)(f_w)(f_{HV})(f_g)(f_p)(f_{bb})(f_a)(f_{RT})(f_{LT}) \qquad \text{(Eq. 3.4)}$$

where S = saturation flow for a selected lane group, expressed as a total for all lanes in the lane group under prevailing conditions, (vphg),

S_o = ideal saturation flow rate per lane, generally taken as 1800 (vphgpl),

N = number of lanes in selected lane group,

f_w = adjustment factor for lane width,

f_{HV} = adjustment factor for heavy vehicles,

f_g = adjustment factor for approach grade,

f_p = adjustment factor for parking,

f_{bb} = adjustment factor for the blocking effect of local buses that stop within the intersection area,

f_a = adjustment factor for the area type,

f_{RT} = adjustment factor for right turns in the lane group,

f_{LT} = adjustment factor for left turns in the lane group.

Since the through movements of compressed and single-point diamonds essentially operate identically, the saturation flow rates for this movement can be assumed to be equal for both interchanges. However, the left turn maneuvers of each form show operational differences that influence saturation flow rates. Because of the obtuse nature of left turn maneuvers on single-point diamonds (i.e. turns which require greater than normal turning radii), drivers can negotiate turns at speeds higher than that for conventional diamonds. Therefore, saturation flow rates that are nearly the same as through moving flow rates can be produced [21]. The following observations of saturation flow rates were made by Leisch et. al. [21] from their comparison of compressed and single-point diamonds:

1) Through movements:
   a) Compressed and single-point diamonds essentially have identical saturation flow rates, (typically 1800 pcphpl).
2) Left-turn maneuvers:
   a) Compressed diamonds; saturation flow rates may be 8 to 20 percent less than through movements,
   b) Single-point diamonds; saturation flow rates may be 5 to 15 percent less than through movements.

The overall importance of saturation flow rates is that in conjunction with critical-lane volumes, signal cycle lengths and green time splits can be estimated. This is briefly discussed in the following section.

Cycle Length and Green Time Splits

Although there are numerous methods available to establish signal cycle length, Webster's method [11,16] is typically used since it approximates a cycle length that minimizes the total intersection delay. Cycle length is obtained by the following equation:

$$C_o = \frac{1.5L + 5}{1 - \sum_{i=1}^{n} Y_i} \qquad \text{(Eq. 3.5)}$$

where C_o = optimum cycle length, (sec),

L = total lost time per cycle, (sec),

Y_i = maximum value of the ratios of approach volumes to saturation flows for phase i,

n = number of phases.

Once the cycle length has been determined, the available green time per cycle must be distributed among the signal phases in proportion to their critical-lane volumes. Equation 3.6 estimates the green time per phase in seconds [11].

$$(G_i + t_i) = \frac{Y_i(C - L)}{\sum_{i=1}^{n} Y_i} + l_i \qquad \text{(Eq. 3.6)}$$

where G_i = green time for each phase i, sec,

t_i = clearance interval, or amber time for phase i, sec,

C = actual cycle length used, (usually obtained by rounding off C_o to the nearest 5 sec),

l_i = lost time per phase, sec.

SUMMARY

This chapter provided a variety of geometric design elements related to the design of compressed and single-point diamonds. Items discussed in this chapter included general interchange configurations, lateral clearances, right-of-way considerations, grade separation length, and ramp terminals. Also, a section was devoted to basic operational elements of diamond-type interchanges. Once again, the items discussed in this chapter and the previous one can be used in the design of new roadway facilities, or in the evaluation and upgrade of existing ones.

The next chapter discusses basic situations, or conditions that help justify the implementation of grade separations. The following chapter also presents a variety of methods, found in the literature, that can be used for evaluating the need for grade separations.

CHAPTER 4
WARRANTING GRADE SEPARATIONS

INTRODUCTION

Grade-separated interchanges provide an effective means of processing traffic. Their versatile nature allows them to be adapted to a wide variety of intersections. This is evident from the numerous geometric configurations found on roadway systems. However, there are significant costs associated with the construction of any grade-separated interchange. Therefore, the implementation of grade separation is limited to cases where the required expenditure can be justified [3]. Unfortunately it is extremely difficult to develop a specified list of conditions or warrants which justify the construction of an interchange, not to mention the complexities involved in selecting an appropriate candidate site. This is because of the wide variety of traffic characteristics, site conditions, intersection geometrics, and roadway types. Therefore, warrants that may justify grade separation for one location could be different for another. This chapter provides a summary of warrants and models, found within the literature, that may be used by planners to help them formulate recommendations for the implementation of grade separations.

GENERAL WARRANTS

This section describes warrants proposed primarily by AASHTO [3], however other sources were also employed to compile a representative list. Warrants described in this section include: (1) design designations; (2) elimination of bottlenecks (traffic volume warrant); (3) accident reduction; (4) driver benefits; (5) site topography; and (6) miscellaneous warrants.

Design Designations

The design designation of a roadway is one method used to help decision-makers decide whether or not grade-separated interchanges are justified. If a given route has been designated as a freeway corridor, then all conflicting approaches to the freeway must be evaluated individually. In other words, it must be determined whether a given crossroad should be terminated, rerouted, or provided with grade separation. This determination is based on the importance of the crossroad. A minor or local road would typically be terminated, or rerouted, due to their nature of having low traffic volumes. On the other hand, a primary arterial or collector with significant traffic volumes may be a good candidate for grade separation. Since this determination is based, in part, on traffic volumes, a method for estimating the amount of traffic that justifies grade separation needs to be evaluated in more detail. Such a methodology (for arterial streets) is provided in Chapter 5. Section 4.2.2 provides a brief discussion on traffic volume warrants.

The justification for implementing grade separations based solely on the design designation of a roadway is generally applicable in the case of freeway corridors only. Because the arterial street in this study has not been designated as a freeway corridor and does not conflict with any existing freeways, this warrant would not be applicable. Of course this does not imply that the designation of the roadway cannot be changed in the future. In fact the alteration of an arterial street to a high-flow arterial may merely be a stepping stone in the design evolution from a simple arterial street to a freeway.

Elimination of Bottlenecks (Traffic Volume Warrant)

Intersections that cannot provide sufficient capacity for roadways with significantly large volumes will inevitably experience excessive levels of congestion on one or more of its approaches. Such intersections are typically classified as bottlenecks [3]. A single congested intersection of an arterial network can easily affect nearby intersections or driveways if long queues of vehicles are allowed to spillback. Therefore, it is essential to minimize the delay that results from heavy congestion through the use of at-grade treatments. Surface treatments may include signal optimization, channelization, and pavement re-striping. If a bottleneck cannot be eliminated through simple and cost-effective means, then grade separation may be justified. Chapter 5 examines intersection delay and relates it to the overall cost-effectiveness of grade separation.

Accident Reduction

For intersections where the accident rate is significantly high, grade separation may be justified. This is because more crossing or turning conflicts are encountered at surface intersections, and grade separations remove a significant portion of these vehicle conflicts. Therefore, the likelihood of traffic accidents can be significantly decreased. Since the elimination or reduction of driver accidents can be considered as a driver benefit, a discussion is provided in the following section.

Driver Benefits

There are a variety of significant driver costs associated with the delays experienced at congested surface intersections. These costs are typically in the form of fuel consumption, oil use, tire wear, waiting time, accidents, and excess mechanical wear (i.e. engine, transmission, or break wear) [3,30]. Overall these costs are generated because of the necessity for speed-changes, stops, and waiting (idling) at intersections with interrupted operations. Therefore, the operational expense associated with such intersections is well in excess of intersections with continuous or uninterrupted operation, namely grade separations. Given this relationship between driver benefits and the cost of improvement, there is an economic indication that may warrant an improvement such as grade separation. This relation is most often expressed as a ratio (i.e. the annual benefit divided by the annual capital cost for the improvement). The annual benefit represents the difference in driver costs for the existing conditions and for the conditions after the improvement. The annual capital cost for the improvement represents the sum of the interest, and the payment due on the principal at the time of each periodic interest payment. In order to justify the economic expenditure of constructing a grade-separated interchange, an absolute minimum benefit-to-cost ratio of one would be required. Anything in excess of one makes this justification even stronger. The major emphasis of the case study presented in Chapter 5 is driver, or user benefits.

Site Topography

In regions where the surrounding terrain experiences significant changes in grade, it may be economically and/or physically impossible to implement any type of intersection design other than that of a grade-separated one. This is due primarily by the design constraints associated with vertical alignment (see Section 2.4). However, the arterial intersections evaluated in Chapter 5 of this report experiences no significant changes in grade. This is typical for arterial intersections in most urban environments. Therefore, the application of this warrant is somewhat rare in urban regions and will not be considered further.

Miscellaneous Warrants

The following list describes miscellaneous circumstances that might warrant the consideration of grade separation [3]. Note however that these warrants are, in general, not relevant to the case study presented in Chapter 5, nor are they applicable to all urban arterials. Nonetheless, this list is provided to help in decision-making, and for general interest. The warrants include:

1) Locations where the termination of local roads and streets is not feasible because of limitations in freeway right-of-way.
2) Areas that are not accessible by means of frontage roads or other sources of access.

3) Rail (transit) corridors.
4) Locations with unusual concentrations of pedestrian and/or bicycle traffic (for example, a golf courses, a city park, or any other recreational area that may be developed on both sides of a major roadway).
5) Other areas with routine pedestrian and/or bike traffic, especially in school zones.
6) Mass transit stations that require access because of their location within the confines of a major arterial.
7) Free-flow characteristics of certain ramp configurations and completing the geometry of an interchange.

METHODS FOR EVALUATING GRADE-SEPARATED INTERCHANGES

This section provides a discussion of various methods, that have been used by others, to evaluate the benefits associated with upgrading surface intersections to simple grade-separated interchanges. These methods may be employed to help planners justify improvements to arterial streets. Four methods were found in the literature, they include work from the following individuals and their respective institutions: M.A. Sargious and T. Tam from The University of Calgary, Canada [26]; B. Rymer and T. Urbanik (TTI) from Texas A&M University [25]; J.M. Witkowski from The University of Arizona at Tucson [30]; and T. Kruger [19] from The University of Texas at Austin.

The Sarglous and Tam Method

It is suggested by Sargious and Tam [26] that in order to justify the upgrading of a surface intersection to that of a simple diamond-type interchange, it is necessary to evaluate and quantify the savings in time, or delay savings. Their report discusses a methodology for this evaluation.

In their analysis of diamond interchanges various generalized assumptions and/or stipulations were made, they include:

1) An ordinary four-phase (overlap) signal timing scheme (see Figure 3-5) is the only plan considered in the analysis however, the method can be extended to include other schemes.
2) Webster's delay equation is used to estimate vehicular delay when the intersection is undersaturated, whereas for oversaturated intersections a queuing model developed by Gazis is used to estimate vehicular delay (see report [26] for further reference).

3) For Webster's delay equation use a saturation flow rate of 1700 pcphpl and a lost time of 3.5 seconds. For the queuing model use 1600 pcphpl (lost time is included in the capacity of the intersection).
4) The estimated vehicular delay is equal to the sum of delays for the six external approaches to the interchange.
5) The through traffic lanes on the freeway (i.e. the grade-separated lanes) can pass through the intersection without delay.
6) The delay for right turning traffic is approximately the same for at-grade intersections as it is for diamond interchanges.

Once a method was developed for finding vehicular delay at diamond interchanges, an expression was developed for the purpose of estimating delay savings. This expression was developed using a regression analysis, and it represented the best fit equation. The data base for this analysis included: (1) number of approach lanes; (2) percentage of left turning vehicles; and (3) 540 different combinations of traffic volumes (ranging between 70 and 55,000 veh-hr/day). Since the input volumes were in the form of average daily traffic (ADT), an assumed daily distribution of traffic was used to convert the ADT into hourly volumes. The following is the equation developed:

$$\ln(RD) = \left[(0.0141) \cdot V_m^2 \cdot \ln(A_m)\right] + \left[(0.0319) \cdot V_n^2 \cdot \ln(A_n)\right] + \left[(0.0923) \cdot L_{tm}^2\right] + \left[(0.1232) \cdot L_{tn}^2\right] + (4.1997) \qquad \text{(Eq. 4.1)}$$

where RD = reduction in delay when a diamond interchange is constructed instead of an at-grade intersection, (veh-hr/day),

$V_{m,n}$ = through lane volumes in one direction on the freeway and arterial street respectively, (1000 veh/day),

$L_{tm,tn}$ = total number of left turning vehicles in one direction on the freeway and arterial respectively, (1000 veh/day),

$A_{m,n}$ = number of through lanes in one direction on the freeway and arterial respectively,

ln = natural logarithm.

Once an estimate is made for the reduction in delay, due to the upgrading of an at-grade intersection to a diamond interchange, the potential benefit of this reduction can be compared with the costs of the facility construction and maintenance. Through this comparison, a rough estimate can be obtained for the traffic volume above which a diamond interchange is warranted.

The Rymer and Urbanik (TTI) Method

In a similar report by The Texas Transportation Institute [25], a procedure was established for evaluating grade separation projects based on quantifying vehicle delay improvements. At-grade intersections, conventional diamond interchanges, and three-level diamond interchanges were evaluated. The tool employed in this evaluation was the TRANSYT-7F computer program (a macroscopic deterministic traffic model). Overall, the purpose of using this computer model was to find estimates for the total system delay (i.e. stopped delay plus approach delay) of a given facility. The delay was calculated on the basis of hourly volumes, turning movement percentages, and other assumptions. Some of these assumptions included the following:

1) All at-grade intersections had separate left and right turn bays.
2) Saturation flow rates for left turns were estimated to be 1700 vph, and 1750 vph for through and right turning traffic.
3) General signal phasing and timing assumptions:
    a) High-type intersections; four phases with leading left turns, and a minimum cycle length of 40 seconds with a 3 second clearance interval,
    b) Diamond interchange; three phases with appropriate offsets between the two intersections, and the same timing plan as the previous intersection,
    c) Three-level diamond; coordinated two phase scheme, and a minimum cycle length of 30 seconds with a 3 second clearance interval.
4) The crossroad directional volume split was 50/50.
5) Two turning movement scenarios were provided for each configuration:
    a) Heavy turning movements; 20 percent for each left and right turn movement on each approach,
    b) Light turning movements; 10 percent movements on each approach.
6) Delay is neglected on the free moving through lanes (i.e. the grade-separated lanes).

Through the use of the TRANSYT simulation, a graphical relationship between total system delay and hourly volume was developed. The resulting curves were obtained by initially using low traffic volumes and then incrementally increasing the volumes in each succeeding simulation until oversaturation occurred. In order to provide a reliable operational comparison of intersections and grade separations, the total system delay represented a summation of all intersection(s) delay within the system. In all cases, an asymptotic relationship between delay and volume was evident. Given the similarities among the delay and volume characteristics of each configuration, two delay equations were developed for estimating vehicular delay incurred on the signalized, at-grade section of a diamond interchange. Also, the similarities of curve shapes among the at-grade intersection, the diamond, and the three-level diamond allowed direct comparisons to be made among them. The equations are as follows:

$$Delay_{(4x4)} = (1.1778)\, e^{V(0.00072452)} \qquad \text{(Eq. 4.2a)}$$

$$Delay_{(6x6)} = (1.2662)\, e^{V(0.00056726)} \qquad \text{(Eq. 4.2b)}$$

where $Delay_{(4x4)}$ = delay at a 4x4 high-type intersection (i.e. four through lanes by four through lanes), (veh-hr/hr),

$Delay_{(6x6)}$ = delay at a 6x6 high-type intersection, (veh-hr/hr),

V = total volume entering at-grade intersection, (veh/hr).

Overall, these equations can be used in an hour-by-hour, day-by-day, or year-by-year economic planning analysis for evaluating a grade separation with the assumed geometrics. In other words, this economic analysis determines if the benefits to the drivers of reduced delay will offset the cost of a grade-separated interchange.

The Witkowski Method

Another method for evaluating the user benefits from grade separation was outlined by J.M. Witkowski [30]. This evaluation was demonstrated through a hypothetical case study that compared operations of an urban grade-separated interchange (i.e. a single-point diamond) to operations of an at-grade intersection. This comparison, using several traffic demand levels, was made in terms of vehicle operating costs, accidents, vehicle emissions, and vehicular delay. The procedure in this study represented a synthesis of various techniques from related reports and traffic manuals. Like the other methods discussed in this chapter, Witkowski's procedure incorporated a variety of basic assumptions and stipulations (too numerous to list). Suffice it to say that many of these assumptions resembled those of other investigations. Also, because of the complexities involved in the iterative process, a detailed explanation of the procedure will not be provided. The following sections will highlight key points only.

a) Delay Estimation

A significant portion of the report dealt with the estimation of delay. Operational analysis procedures for signalized intersections, as outlined in the HCM [28], were used to estimate stopped time delay. Approach volume-weighted average stopped delay per vehicle was calculated for each intersection approach for both the peak and off-peak hours of the day. The calculations for fuel consumption, vehicle emissions, and total delay were based on these weighted delay values. Once the difference between the annual hours of delay for an at-grade intersection and the annual hours of delay for a grade-separated interchange was determined, the delay savings were calculated. Also, an estimate was made on the total annual monetary benefits resulting from the delay reduction using an assumed per hour value of travel time. The reduction in total delay ranged from 53 percent for a low demand level to 84 percent for higher levels of demand. User travel time benefits were not the only economic indicators used in this analysis, vehicle running costs were also analyzed.

b) Vehicle Running Costs

The primary costs associated with vehicle operations are the fuel consumption costs. Fuel consumption at an intersection varies depending on the operation being performed by the vehicle. In other words, there are different rates of consumption for vehicles, (1) traveling at constant speeds; (2) idling; (3) stopping; and (4) performing other speed-change cycles. Four equations are presented in Chapter 5 that are used for estimating fuel consumption. The estimated annual savings in fuel ranged from 47 thousand gallons at low demand levels to 461 thousand gallons at higher demand levels. For ADT levels of 40,000 and 50,000 vehicles per day, fuel reduction through the intersection was estimated to be 21 and 29 percent, respectively. Other costs associated with vehicle operations include oil consumption, tire wear, maintenance/repair, and depreciation. These costs were not considered because they were assumed to remain unchanged between the alternatives. Finally, the report presents the costs associated with accidents.

c) Accident Costs

Accidents can be grouped into three general categories, namely property damage, personal injury, and fatal accidents. Damage to property is the most common type of intersection accident followed by personal injury accidents and fatal injuries. The costs associated with accidents are fairly easy to estimate given the availability of insurance claim data. However, if an intersection is to be evaluated, then a substantial record of accident types and rates are necessary. Also, a substantial amount of time is necessary to accurately estimate the change in accident rates resulting from a roadway improvement. Three years of accident data was used for the AGI. Only six months of accident reports were available for the urban GSI. Therefore, the benefit attributable to the improvement in terms of the reduction in accidents were viewed with caution. The accident rate reduction (attributable to the urban GSI) was 43 percent for those vehicles entering the signalized portion of the intersection. This translated into a 66 percent reduction in the accident rate (based on the total traffic entering the the GSI). Personal injury accidents displayed the highest reduction, 88 percent, followed by property damage at 60 percent. Overall, a minimum 82 percent reduction in annual accident costs were expected. Once all the benefits attributable to the reduction in vehicle running costs, user travel time, and accidents were estimated, a benefit-cost analysis could be performed.

Overall it was shown that an urban GSI conservatively provided substantial economic benefits as a replacement to an AGI. Benefit-cost ratios ranging from 2.5 to 3.5 as a minimum, depending on the traffic demand levels, were attributable towards this improvement.

The Kruger Method

In a dissertation prepared by T. Kruger [19], a method was developed to aid in locating grade separations along arterial streets. Kruger suggests that the planning and design of intersection controls along arterial streets can be based on the determination of an average target travel speed along a given arterial length. Intersection delay (which influences travel speed) plays a significant role in the analysis of signal controlled arterials. For planning purposes it is suggested that the process of locating grade separations at existing surface intersections can be based on green time assignments and intersection spacing.

Ideally the green time for traffic along arterial streets should be assigned such that the required average travel speed can be achieved. However, this is only possible when the intersection delay is within appropriate limits. If after the required green time has been assigned to the main arterial street, the traffic demand on the cross street (minor approach) results in excessive delays and queue lengths, then grade separation of the intersection will be indicated

[19]. In other words, if the maximum reasonable green times assigned are not adequate to yield the average target travel speed, then the effective spacing between signal controlled intersections should be increased through the use of grade separations at selected intersections.

a) Iterative Process

The general process necessary for determining the location of grade separation structures is depicted schematically in Figure 4-1, and the following procedure discusses this process in greater detail.

Step 1. Input the demand volume for a selected length of the arterial. The length of the segment is made up of relatively homogeneous street sections between the signalized intersections. It is assumed that all relevant surface treatments have been exhausted (i.e. the maximum number of through lanes have been provided and any signal timing plans have been optimized).

Step 2. Make an estimate of the maximum possible green time that can be allocated for the through traffic on the arterial. Input this into the model to initiate the process.

Step 3. Based on the assumed signal settings and demand volumes, estimate the resulting average travel speed. This can be a tediously difficult step, but existing simulation models including, NETSIM, TRANSYT, HCM, or the TEXAS Model can simplify this estimation considerably.

Step 4. Using the calculated average travel speed, make a comparison to the operational criteria set for strategic arterials. This criteria includes a free-flow speed of 40 mph, an average travel speed of approximately 30 mph (when the V/C ratio is in the vicinity of 0.9), and maximum flow rates of 800 to 1000 passenger car equivalents per hour per lane.

Step 5. If this criteria is met, then the other approaches to the intersection can be analyzed. Simple procedures such as considering V/C ratios, or analysis by methods outlined in the *Highway Capacity Manual* [28] may be sufficient. However, HCM methods are most applicable for moderate ranges of some of the parameters and therefore, may fail to provide a true picture of traffic operations at extreme values (i.e. very high or low (g+y)/c ratios).

Consequently, more detailed models may need to be employed and is dependent on how critical the case may be.

Step 6. If the delays or V/C ratios are excessive on the cross streets, relative to that of the arterial street, then a grade separation may be indicated for that location. With a grade separation provided, the effective segment length should be increased and the procedure repeated (starting with Step 1) for the longer segment to the next signalized intersection.

Step 7. If however the traffic operations at the intersection are considered adequate, and the signalization adequately provides traffic operations compatible with the strategic arterial criteria, then the next segment can be analyzed.

Step 8. If the selected signalization does not provide for the average target travel speeds, then the green time for the arterial street approaches must be increased and the procedure is repeated from Step 2. However, if the allocated green time for the arterial street is already considered to be at a maximum, then the intersection, or an upstream intersection, is a candidate for grade separation. This will effectively increase the segment length and the process returns to step one.

This procedure, although tedious, adequately provides a method for warranting grade separations along arterial streets.

SUMMARY

This chapter has examined various warranting conditions that should be considered when planning the improvement of intersections through the use of grade-separated interchanges. Also, four methods were discussed for evaluating the benefits associated with upgrading surface intersections to simple grade-separated interchanges. These methods were employed to help planners justify improvements to arterial streets.

Chapter 5 provides a benefit analysis of four urban arterial intersections in Austin, Texas. This analysis incorporates various assumptions and procedures from the methods that were presented previously.

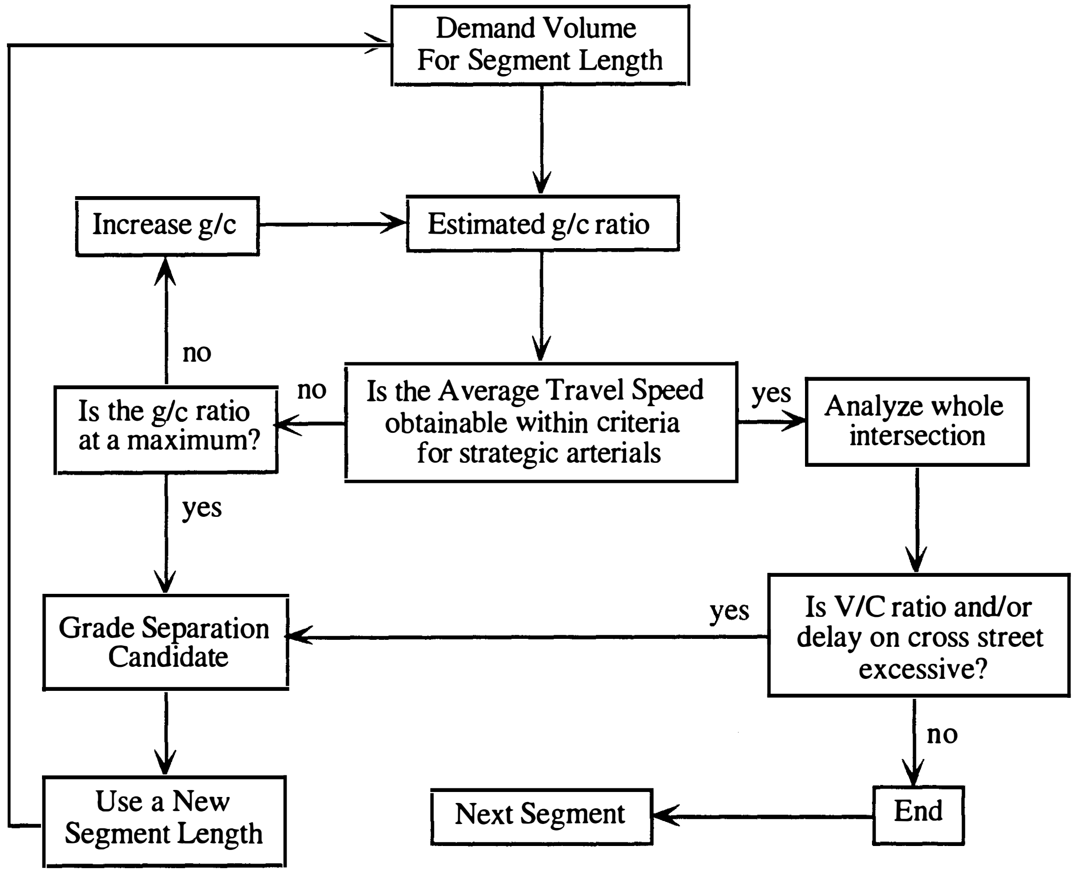


Figure 4-1. Decision Process for Determining Signal Green Time and Grade Separation Locations on Strategic Arterials. Source: Ref. 19, Fig. 5.5.

CHAPTER 5
BENEFIT ANALYSIS FOR URBAN ARTERIAL INTERSECTIONS

INTRODUCTION

This chapter presents a simplified method for evaluating vehicle, or user, benefits attributable to intersection improvements (i.e. upgrading at-grade intersections to grade-separated interchanges). Because of the simple nature of this benefit analysis, only a limited amount of data, along with some underlying assumptions, are needed for evaluating an intersection. Major intersections along an urban arterial street in Austin, Texas, were analyzed. They include: (1) Riverside & Congress; (2) Oltorf & Congress; (3) Stassney & Congress; and (4) William Cannon & Congress. Overall, the analysis discussed in this chapter is meant to be used as a sketch planning tool. This analysis makes a comparison between the operations for an at-grade intersection (AGI) and a grade-separated interchange (GSI) in terms of the delay, vehicle running cost, and user travel time cost. Finally, a benefit-cost ratio is used to determine the cost-effectiveness of an interchange.

ECONOMIC CONSIDERATIONS

The factors that should be considered in an engineering economic analysis of roadway improvements are those affected by highway design and traffic conditions. These factors include: (1) vehicle running, or operational costs consisting of fuel consumption, oil consumption, tire wear, maintenance/repair, and depreciation; (2) user travel time costs; and (3) traffic accident costs [2]. These factors are discussed in more detail in the following sections. Elements of total operating costs that may be omitted from an economic analysis include: (1) any part of vehicle depreciation that is not associated with traveling on the roadway (i.e. not mileage-dependent); (2) interest charges for the investment in the vehicle; (3) license, toll, garage, or parking fees; and (4) insurance premiums. These elements can be omitted because they are not controlled by roadway design and traffic conditions [2].

Vehicle Operating Costs

The primary costs associated with the operation of motor vehicles include fuel and non-fuel running costs. Fuel operating costs are primarily due to vehicles slowing down to and speeding up from a stop caused by intersection traffic control devices. Also, fuel operating costs are incurred due to idling while stopped [2,30]. Non-fuel running costs include tire wear, oil consumption, vehicle maintenance and repair, and vehicle depreciation [2,3,30,31]. Elements that influence operating costs include: (1) vehicle characteristics (weight, state-of-repair, and engine horsepower/efficiency); (2) tire types and conditions; (3) roadway characteristics (design and maintenance); and (4) environmental factors (weather and topography). Because of the considerable number of factors that influence vehicle operation, there is considerable difficulty in estimating vehicle running costs. However, Zaniewski et. al. [31] provides a detailed report on vehicle operating costs. Also, the report provides cost tables for the estimation of running costs for a variety of vehicle classes (i.e. small-medium-large passenger cars, single-unit trucks, and semi-trucks). These tables include estimates for total operating costs at constant speeds on grades, total costs for speed-change cycles, and total costs on horizontal curves. The following section provides a method for estimating vehicle operating costs.

a) Estimation of Vehicle Operating Costs. To maintain a conservative approach towards estimating vehicle running costs, non-fuel costs (i.e. tire wear, oil consumption, maintenance/repair, and depreciation) will not be considered in this study. It is assumed that these factors will be unchanged for the before and after improvement of the intersection [30]. Therefore, only fuel consumption for stopping, speed-change cycles, and idling will be considered. Equations 5.1 through 5.3 are relationships for estimating the excess fuel consumed due to stopping, speed-change cycles, and idling [17,30]. Also, Equation 5.4 estimates the fuel consumed for constant speeds.

Stopping: $$AFC1 = \frac{[0.5497\log(1.3D_S) - 0.1404](V)(FCR1)}{1000} \qquad \text{(Eq. 5.1)}$$

Speed Changes: $$AFC2 = \frac{[(V)(FCR2)(0.04D_S + 0.03)]}{3600(HS)} \qquad \text{(Eq. 5.2)}$$

Idling: $$AFC3 = \frac{(V)(D_S)(FCR3)}{3600} \qquad \text{(Eq. 5.3)}$$

Constant Speed: $$FC = \frac{(FCR)(V)(L)}{1000} \qquad \text{(Eq 5.4)}$$

where AFC = additional, or excess fuel consumed, (gallons),

FC = constant speed fuel consumption, (gallons),

V = total traffic entering intersection, (veh/unit of time),

D_S = stopped time delay, (sec/veh),

L = length of the roadway being analyzed, (miles),

HS = number of vehicle-hours per thousand speed change cycles, (see Table 5-1),

FCR = fuel consumption rate; consumption rates for stopping and speed-change cycles are provided in Table 5-2; for constant speed consumption rates see Table 5-3; and for idling use 0.563 gallons per hour [31].

TABLE 5-1. EXCESS HOURS CONSUMED PER THOUSAND SPEED-CHANGE CYCLES BEYOND HOURS CONSUMED BY CONTINUING AT INITIAL SPEED (FOR PASSENGER CARS)

Initial Speed (mph)	Speed Reduced To and Returned From (mph)										
	Stop	5	10	15	20	25	30	35	40	45	50
5	1.02										
10	1.51	0.62									
15	2.00	1.12	0.46								
20	2.49	1.62	0.93	0.35							
25	2.98	2.11	1.40	0.80	0.28						
30	3.46	2.60	1.87	1.24	0.70	0.23					
35	3.94	3.09	2.34	1.69	1.11	0.60	0.19				
40	4.42	3.58	2.81	2.13	1.52	0.97	0.51	0.16			
45	4.90	4.06	3.28	2.57	1.93	1.34	0.83	0.42	0.13		
50	5.37	4.54	3.75	3.01	2.34	1.71	1.15	0.68	0.35	0.11	
55	5.84	5.02	4.21	3.45	2.74	2.08	1.47	0.94	0.57	0.28	0.09

Source: Ref. 17, Table 2.

TABLE 5-2. EXCESS FUEL CONSUMPTION FOR SPEED-CHANGE CYCLES (GAL/1000 CYCLES) - MEDIUM PASSENGER CARS

Initial Speed (mph)	Speed Reduced To and Returned From, (mph)												
	Stop	5	10	15	20	25	30	35	40	45	50	55	60
5	1.00												
10	1.98	0.98											
15	3.01	2.01	1.03										
20	4.17	3.17	2.19	1.16									
25	5.42	4.42	3.44	2.41	1.25								
30	6.80	5.80	4.82	3.79	2.63	1.38							
35	8.67	7.67	6.69	5.66	4.50	3.25	1.87						
40	10.70	9.70	8.72	7.69	6.53	5.28	3.90	2.03					
45	12.90	11.90	10.90	9.87	8.71	7.46	6.08	4.21	2.18				
50	15.30	14.30	13.30	12.30	11.10	9.85	8.47	6.60	4.57	2.39			
55	17.90	16.90	15.90	14.90	13.70	12.40	11.00	9.18	7.15	4.97	2.58		
60	20.80	19.80	18.80	17.80	16.60	15.30	13.90	12.10	10.00	7.85	5.46	2.88	
65	24.30	23.30	22.30	21.30	20.10	18.80	17.40	15.60	13.50	11.30	8.95	6.37	3.49

Source: Ref. 31, Table B.35.

TABLE 5-3. CONSTANT SPEED FUEL CONSUMPTION (GAL/1000 MILES) - MEDIUM PASSENGER CARS

Grade	Speed, (mph)												
(%)	5	10	15	20	25	30	35	40	45	50	55	60	65
7	82.00	82.00	75.50	68.50	68.30	68.00	71.80	75.50	80.00	84.00	93.50	103.0	104.0
6	77.50	77.50	70.80	64.00	63.00	62.00	65.30	68.50	73.00	77.30	87.00	96.00	97.80
5	74.00	74.00	67.50	61.00	59.80	58.50	61.30	64.00	68.50	72.50	80.30	87.50	91.50
4	73.00	73.00	66.50	60.00	57.80	55.50	58.30	60.50	64.50	68.00	73.00	77.50	84.80
3	71.50	71.50	64.50	57.50	55.50	53.50	56.30	58.80	61.00	63.00	66.00	68.50	78.30
2	68.00	68.00	60.80	53.50	52.30	50.50	53.00	55.50	56.80	58.00	60.50	62.50	72.50
1	61.50	61.50	54.30	47.00	46.30	45.00	46.00	46.50	49.30	51.50	54.50	57.00	64.50
0	55.40	55.40	47.30	38.70	38.00	37.30	37.60	38.00	40.50	43.00	47.90	52.80	57.60
-1	52.00	52.00	41.80	31.00	30.30	29.50	31.80	33.50	34.80	36.00	41.00	45.50	51.00
-2	50.80	50.80	39.70	28.00	25.80	22.50	26.30	29.50	30.30	31.00	34.80	38.50	45.00
-3	51.30	51.30	38.90	26.90	23.70	20.50	23.30	25.50	26.50	27.80	31.50	35.00	41.30
-4	52.00	52.00	39.90	27.30	24.00	20.30	20.70	21.00	23.00	25.00	28.80	32.00	37.50
-5	53.00	53.00	40.00	27.30	23.80	20.00	19.30	18.50	20.50	22.80	26.00	29.50	34.50
-6	53.50	53.50	40.60	27.30	23.50	19.80	18.00	16.30	18.90	21.00	29.00	27.00	30.80
-7	54.30	54.30	40.70	27.30	23.80	19.50	17.30	15.00	17.30	19.00	21.30	23.50	26.80

Source: Ref. 31, Table B.2.

The following assumptions were incorporated in the estimation of vehicle running costs:

1) The total delay per vehicle, D_t, which includes the delay due to slowing down and accelerating to the running speed [30], is estimated to be 1.3 times the stopped time delay, D_s, [17,28]. (i.e. $D_t = 1.3[D_s]$).
2) The grade-separated interchange is assumed to remove 40 percent of the total entering volume from the surface intersection.
3) The overall average running speed for the intersection was taken as 30 mph.
4) For speed-change cycles, the average reduction in speed for vehicles was taken as half the average running speed. Therefore, for a speed of 30 mph a vehicle is assumed to slow down to 15 mph hour and then, accelerate back to the original 30 mph [17,30].
5) The intersection approaches are level (i.e. no grade changes).
6) The length of roadway was taken as half a mile, since it was assumed that the intersection would affect travel speed for a quarter mile on either side of its location [30].
7) Since the case study evaluates urban arterial intersections, it is assumed that the majority of vehicles will be passenger cars, therefore no truck or semi-trailer data was considered in this study.
8) The average price of fuel was taken as $1.15 per gallon.

Appendix A provides fuel consumption cost estimates for the intersections of this study.

User Travel Time Costs

It is assumed that for every roadway user there is an associated monetary travel time value. This value is essential for estimating user benefits associated with a given roadway improvement. In other words, if an improvement to a roadway facility can save user time, then the improvement is seen as an overall benefit, and a monetary value can be placed on the saved travel time. Generally, a unit value of time is selected or assumed (typically this value is expressed in dollars per vehicle-hour). To calculate the the value of travel time savings, the unit value of time is multiplied by the amount of user, or vehicle time saved. Similarly, travel times with and without a given improvement can be multiplied by this unit value of time, where the difference between these two products represent the value of time savings [2]. This section describes a simple method for estimating the unit value of user time.

a) Estimation of the Unit Value of User Time. The per capita personal income for the metropolitan area of Austin, Texas, was used to estimate user time. Table 5-4 provides the personal income of Austin residents for the years 1978 through 1986. Using this data, an estimate of \$23,300 was found for the per capita personal income for 1991. The unit value of user time was estimated by dividing the per capita income by the total number of hours in a year (8760 hours). The total number of hours per year were used, instead of an assumed number of working hours per year. This provided a more conservative estimate of user time. Finally this value was multiplied by an assumed vehicle occupancy of 1.25 persons to obtain a unit value of user time equivalent to \$3.32 per vehicle-hour. See the following expression:

$$\left(\frac{\$23300}{\text{person-yr}}\right) \cdot \left(\frac{\text{year}}{8760\text{ hrs}}\right) \cdot \left(\frac{1.25\text{ persons}}{\text{vehicle}}\right) = \frac{\$3.32}{\text{veh-hr}} \qquad \text{(Eq. 5.5)}$$

Accident Costs

Accidents can be grouped into three general categories, namely property damage, personal injury, and fatal accidents [2,30]. Damage to property is the most common type of intersection accident followed by personal injury accidents and fatal injuries. The costs associated with accidents are fairly easy to estimate given the availability of insurance claim data. However, if an intersection is to be evaluated, then a substantial record of accident types and rates are necessary. Also, a substantial amount of time is necessary to accurately estimate the change in the accident rate resulting from a roadway improvement. Without the examination of several case studies, it may be impossible to predict how a given improvement will affect the accident rate of an intersection (i.e. the safety of a given intersection).

For the purposes of this thesis it is assumed that the accident rate is negligible and therefore, no accident related data was evaluated. Overall, eliminating costs associated with traffic accidents will provide a more conservative estimate of roadway user costs, or benefits.

Grade Separation Costs

Because of differences in grade separation configurations, property/land costs, construction times, and various other site-specific conditions, it is difficult to estimate the overall cost of a given interchange. Rymer and Urbanik [25] suggest that simple diamond forms cost somewhere between 3 and 5+ million dollars (1989 dollars). Bonilla [7] suggests that conventional forms cost somewhere between 3 and 9+ million dollars (1985 dollars) depending on direct construction costs plus delays incurred due to traffic diversion during construction. In

TABLE 5-4. AUSTIN PER CAPITA PERSONAL INCOME FOR THE YEARS 1978 TO 1986 AND ESTIMATES FOR 1991

Per Capita Personal Income for given Year*		Yearly Growth Rate	Estimates for 1991 Per Capita Personal Income (dollars)		
(year)	(dollars)	(%)	Method (1)	Method (2)	Method (3)
1978	7,341	9.89	25,015	22,526	-
1979	8,067	12.37	32,703	22,709	-
1980	9,065	14.54	40,353	23,409	-
1981	10,383	7.62	21,636	24,597	-
1982	11,174	6.83	20,248	24,284	-
1983	11,937	12.95	31,625	23,798	-
1984	13,483	7.77	22,758	24,659	-
1985	14,530	0.10	14,614	24,378	-
1986	14,544	-	-	-	-
	Averages:	9.01	26,119	23,795	19,958

Overall Average 1991 Per Capita Income: $\boxed{\$23,291}$

* Source: Ref. 17, pg. 505.
Method (1) Income estimates are based on each years individual growth rate.
Method (2) Income estimates are based on an overall average growth rate of 9.01%.
Method (3) Income estimate was extrapolated by curve fitting the (1978-1986) income data.

another report by Bonilla and Urbanik [6], the cost for conventional structures ranges from a low of 1.6 million dollars (1987 dollars) to a high of 6.2 million dollars, while for prefabricated structures the cost ranges between 2.8 and 10.8 million dollars (see Table 5-5). Witkowski [30] uses a construction cost of 5.5 million dollars (1988 dollars) for an urban grade-separated interchange. Finally, Byington [8] presents a cost of $\$28/ft^2$ to $\$45/ft^2$ (1980 dollars; based on a composite of bid prices for conventional structures built in the United States and furnished through the Federal Highway Administration's Bridge Division).

TABLE 5-5. DIRECT CONSTRUCTION COSTS OF TYPICAL FLYOVERS (1985 MILLION DOLLARS)

		Number of Lanes		
Construction	Type*	Two	Four	Six
Conventional	Low	1.62	2.17	2.72
	High	4.19	5.19	6.20
Prefabricated	Low	2.85	4.49	6.13
	High	6.23	8.49	10.75

Source: Ref. 6, Table 21.
* Low means designed for 35 mph with limited right shoulders; High means designed for 60 mph with full right shoulders and an 8 ft median provided with CMB.

From these sources it is apparent that there is considerable variation in the cost of grade-separated interchanges. Therefore, it is difficult estimate the cost of interchanges. Despite this difficulty, a cost of 6 million dollars was used in the benefit analysis of this chapter. This cost was chosen so that relative comparisons of intersection improvements could be made. Since the analysis of this chapter is based on a 4x4 high-type structure, the GSI cost was taken from Table 5-5 (assuming a nominal inflation rate of 2.5 percent). This value falls well within the bounds of costs found within the literature.

INTERSECTION DELAY ESTIMATION

Since the costs associated with operating a vehicle are dependent on the amount of delay incurred to the driver, a method for estimating total intersection, or system delay is essential for this study. Also, since the purpose of this study is to provide planners with a simple method for evaluating user benefits (i.e. a sketch-planning tool), the method chosen to estimate vehicular delay only required the knowledge of the average daily traffic (ADT) and a known, or assumed daily distribution of traffic. Such a method was outlined in Section 4.3.2 of this thesis. Equation 4.2a (shown here as Equation 5.6) was used for the benefit analysis of this study.

$$\text{Delay}_{(4x4)} = (1.1778)\, e^{V(0.00072452)} \qquad \text{(Eq. 5.6)}$$

where V = total volume entering intersection, (veh/hr).

This equation was chosen because it was assumed that the intersections of this study would be upgraded, through surface treatments (i.e. signal optimization, channelization, pavement re-striping, etc.), to a condition similar to a 4x4 high-type intersection. These improvements would be used to reduce vehicular delay. Therefore, this equation will provide conservative estimates for delay (if compared to the actual delay of the existing intersection). The estimated values for delay were summed over the day for both the AGI and the GSI. Benefits were based solely on delay reductions. Appendix A provides delay estimates for the intersections of this study. The following assumptions were used to estimate delay values (also, see the assumptions from Section 5.2.1):

1) Yearly delay was based on 250 working days [25].
2) The project life is considered to be 20 years [25,30].
3) The first year of the analysis period was assumed to begin with the opening of the grade-separated interchange (GSI).
4) The ADT was increased to reflect an average yearly growth rate of 2.5 percent [9].

BENEFIT-COST ANALYSIS

Once the delay estimates and associated user benefits were found for year 1 and year 20, relative comparisons could be made. However, this required computing the net present worth of user benefits [2,5,25,30]. A 7 percent interest rate was assumed for this evaluation. Table 5-6 summarizes the results of the benefit analysis. Also, Appendix A provides all the relevant data necessary for this analysis. Benefit-cost ratios, provided in Table 5-6, are used to determine if grade separation is warranted (i.e. if the user delay reduction over the project's life is equal to or exceeds the construction cost of the grade-separated interchange). This will be discussed in the the following concluding chapter.

SUMMARY OF RESULTS

Table 5-6 presents selected results from the simulation data provided in Appendix A. This table provides the existing (i.e. year 1) average daily traffic (ADT) for major arterial intersections along Congress Avenue. The projected ADT for an assumed project life of 20 years is shown in brackets for each intersection. These future ADTs were estimated using an assumed annual traffic growth rate of 2.5 percent. Using each intersections ADT, and their corresponding hourly distribution of traffic, the at-grade hourly delay was estimated using Equation 5.6. Using an assumed 250 working days, the total annual delay was calculated. The difference between the total annual delay for each GSI and AGI (i.e. delay savings) are shown in Table 5-6 for years 1 and 20 (year 20 shown in brackets). Equations 5.1 through 5.4 were used to estimate the annual fuel consumption for each GSI and AGI. The difference between the total annual fuel consumed for each GSI and AGI (i.e. fuel savings) are also shown in Table 5-6 for years 1 and 20. Benefits in the form of dollars saved were found by applying a monetary value to both the delay savings (i.e. user travel time savings) and the fuel savings. These benefits are shown in Table 5-6 for the years 1 and 20 for each intersection. A net present worth approach was used to estimate the current economic value of user benefits (based on a 7 percent interest rate and a 20 year project life). These values are also shown in Table 5-6. Finally, using the current benefit of each intersection and a GSI construction cost of 6 million dollars, benefit-cost ratios were calculated. Overall, this chapter provided a simple method for evaluating the cost-effectiveness of constructing grade-separated interchanges at urban arterial intersections.

TABLE 5-6. SUMMARY OF BENEFIT ANALYSIS

Description of Data	Summary Data for Major Arterial Intersections Along Congress Avenue			
	Riverside	Oltorf	Stassney	William Cannon
ADT for Year 1 and [Year 20], (vehicles x 10^3)	52.4 [85.8]	44.8 [73.5]	35.7 [58.5]	51.3 [84.1]
Annual Traffic Growth Rate, (%)	2.5	2.5	2.5	2.5
Annual Delay Savings for Year 1 and [Year 20], (veh-hr)	32.8 [231.8]	19.9 [104.1]	12.3 [55.8]	28.3 [183.7]
Annual Fuel Savings for Year 1 and [Year 20], (gallons)	137.6 [344.4]	110.8 [236.5]	84.4 [167.8]	131.4 [310.2]
Year 1 Benefits: (\$ x 10^3)				
User Travel Time Savings	108.9	66.2	40.7	94.0
Vehicle Fuel Savings	158.3	127.4	97.1	151.1
Total Savings	267.2	193.6	137.8	245.1
Year 20 Benefits: (\$ x 10^3)				
User Travel Time Savings	769.7	345.7	185.1	609.9
Vehicle Fuel Savings	396.1	272.0	192.9	356.7
Total Savings	1165.8	617.7	378.0	966.6
Present Worth of Benefits: (\$ x 10^3)				
User Travel Time Savings	3714.8	1784.4	990.7	2995.5
Vehicle Fuel Savings	2598.4	1910.0	1399.9	2397.6
Total Savings	6313.2	3694.4	2390.6	5393.1
GSI Construction Cost, (\$ x 10^3)	6000.0	6000.0	6000.0	6000.0
Benefit-Cost Ratio	1.05	0.62	0.40	0.90

CHAPTER 6 CONCLUSION

It has been well documented how grade-separated interchanges can increase intersection capacity, thereby improving mobility and decreasing delay. However, along urban arterials where adjacent right-of-way is generally restricted, and where the acquisition of property is a difficult and expensive venture, it is not always cost-effective to construct interchanges (especially ones with land-hungry configurations). Therefore, this study considered simple diamond-type forms that require a nominal amount of right-of-way (the minimum R.O.W. width required for a four-lane overpass with two-lane ramp terminals is 123 feet; see Table 3-2, page 47). This study also assumed that surface treatments such as signal optimization, pavement re-striping, channelization, and other cost or time effective improvements would be implemented before the construction of a GSI. In other words, grade separation is considered a viable option, only when all at-grade improvements have been exhausted.

Several warrants, proposed by AASHTO [3], were discussed in Chapter 4. These warrants included considerations for the design designation of the roadway, for the elimination of bottlenecks (i.e. excessive traffic volumes), for the reduction of accidents, for the topography of the intersection, and for the benefits to roadway users. Of these warrants, considerations for traffic volumes and driver benefits were primarily applicable to the arterial street analysis.

The case study described in Chapter 5 applied these warrants, in a benefit analysis of four major intersections along an urban arterial (Congress Avenue) in Austin, Texas. Traffic volumes were used to estimate the delay incurred by the driver. These delay values were used in the estimation of excess fuel consumption. Comparisons of user travel time savings and fuel consumption reductions were made between the AGI and the GSI. Costs were applied to these reductions, and a net present worth was estimated for the assumed project life. The results of this analysis are found in Table 5-6. Based on these results it is possible to determine whether grade separation is justified. Overall, this justification is dependent on the benefit-cost ratio (i.e. if the user benefit of delay reductions over the project's life is equal to or exceeds the construction cost of the GSI).

The benefit-cost ratio of Riverside was estimated to be 1.05. This was the only ratio, of the four intersections analyzed, that exceeded 1.0. However, because much of the analysis was dependent on several underlying assumptions, this ratio should be viewed with caution. This ratio suggests, at the very least, that grade separation may be warranted at Riverside and therefore, merits further investigation (i.e. before a final decision can be made on the justification of grade separation, further study is recommended)

The benefit-cost ratio for William Cannon was estimated to be 0.90. Because this ratio is nearly equal to one, grade separation may still be justified. Therefore, it is also recommended that this intersection be investigated further. However, grade separation does not appear to be warranted for either Oltorf or Stassney, which have benefit-cost ratios of 0.62 and 0.40 respectively. Overall, it appears that once an intersection has an ADT greater than 50,000 vehicles per day, grade separation may be warranted (based on a 20-year project life).

It should be noted that the evaluation of these benefit-cost ratios is primarily dependent on the cost of an interchange. If the cost of constructing a GSI was considerably less than the proposed 6 million dollars, say 3 million dollars, then the recommendations of this chapter would change considerably. Likewise, the recommendations of this chapter would change if the cost of a GSI was greater than 6 million dollars. However, the order in which intersections should be considered for further study would not change. In other words, based on the assumptions and procedures of the benefit analysis of Chapter 5, Riverside will always have the greatest benefit-cost ratio. Therefore, any further analysis of intersections along Congress should begin with Riverside followed by William Cannon, Oltorf, and Stassney.

This study was limited to the analysis of intersections along an urban arterial street. However, this analysis assumed that the intersections were isolated and operated identically (i.e. it was assumed that each intersection would be upgraded to a 4x4 high-type AGI; see Chapter 3). No consideration was given for the impact a GSI would have on the operation of surrounding intersections, nor was there any consideration for the existing operational condition of each intersection and the cost of surface treatments. Also, there was no consideration for the impact a GSI would have on adjacent property and businesses. However, Tables 3-4 and 3-5 provide the required lengths for grade separations (i.e. lengths that represent the section of roadway that may affect adjacent facilities). All of these considerations merit further investigation or research, but were beyond the scope of this study. Another consideration worth further investigation is the shift in demand that may result from the construction of an interchange. The analysis of this report assumed that the traffic demand levels remained constant over the design life. Overall, this study provided a simple method (i.e. a sketch-planning tool) for isolating urban arterial intersections that are good grade separation candidates.

APPENDIX A.

ESTIMATES FOR DELAY AND FUEL SAVINGS AT URBAN ARTERIAL INTERSECTIONS

Table A-1 through A-8.	Riverside & Congress Simulation Data
Table A-9 through A-16.	Oltorf & Congress Simulation Data
Table A-17 through A-24.	Stassney & Congress Simulation Data
Table A-25 through A-32.	William Cannon & Congress Simulation Data

Table A-1. Delay Estimations for Riverside & Congress (Year 1)

Time of Day	Approach Volumes (veh/hr) Congress	Riverside	Total	System Delay (veh-hr/hr) At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	256	357	613	1.84	1.54	0.30
1 - 2	188	206	394	1.57	1.40	0.17
2 - 3	148	168	316	1.48	1.35	0.13
3 - 4	69	84	153	1.32	1.26	0.06
4 - 5	62	91	153	1.32	1.26	0.06
5 - 6	139	150	289	1.45	1.34	0.12
6 - 7	378	586	964	2.37	1.79	0.58
7 - 8	1693	1896	3589	15.86	5.61	10.26
8 - 9	1314	1923	3237	12.29	4.81	7.48
9 - 10	1108	1316	2424	6.82	3.38	3.44
10 - 11	1205	1357	2562	7.54	3.59	3.95
11 - 12noon	1553	1717	3270	12.59	4.88	7.71
12 - 1pm	1859	2077	3936	20.40	6.52	13.88
1 - 2	1759	1943	3702	17.22	5.89	11.33
2 - 3	1409	1729	3138	11.44	4.61	6.83
3 - 4	1465	1830	3295	12.82	4.93	7.89
4 - 5	1717	2197	3914	20.07	6.46	13.62
5 - 6	2285	2434	4719	35.97	9.16	26.81
6 - 7	1324	1955	3279	12.67	4.90	7.77
7 - 8	928	1539	2467	7.04	3.44	3.59
8 - 9	732	1162	1894	4.65	2.68	1.96
9 - 10	677	1107	1784	4.29	2.56	1.73
10 - 11	567	833	1400	3.25	2.16	1.08
11pm - 12mid	368	530	898	2.26	1.74	0.52
TOTALS:	23203	29187	52390 ADT	218.50	87.25	131.25

Annual Delay Savings $\boxed{32,813}$ (veh-hr)

Annual User-Time Benefit $\boxed{\$108,940}$ (@ $3.32 per veh-hr)

Table A-2. Fuel Consumption Estimates for Riverside & Congress (Year 1)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel Consumed (gallons) At-Grade	Total Fuel Consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	613	8.30	11.58	14.20	8.95	5.25
1 - 2	394	11.01	16.37	9.51	6.10	3.41
2 - 3	316	12.98	19.74	7.83	5.07	2.76
3 - 4	153	23.82	37.97	4.26	2.87	1.39
4 - 5	153	23.82	37.97	4.26	2.87	1.39
5 - 6	289	13.91	21.33	7.25	4.71	2.53
6 - 7	964	6.80	8.57	21.74	13.46	8.28
7 - 8	3589	12.24	7.21	88.08	48.94	39.15
8 - 9	3237	10.52	6.86	77.58	43.85	33.73
9 - 10	2424	7.79	6.43	55.66	32.57	23.09
10 - 11	2562	8.15	6.46	59.19	34.44	24.75
11 - 12noon	3270	10.66	6.89	78.54	44.32	34.22
12 - 1pm	3936	14.35	7.64	99.20	54.09	45.11
1 - 2	3702	12.88	7.34	91.61	50.60	41.01
2 - 3	3138	10.10	6.78	74.75	42.44	32.31
3 - 4	3295	10.77	6.91	79.27	44.68	34.59
4 - 5	3914	14.20	7.61	98.47	53.76	44.71
5 - 6	4719	21.11	8.96	127.96	66.29	61.67
6 - 7	3279	10.70	6.90	78.80	44.45	34.35
7 - 8	2467	7.90	6.44	56.75	33.15	23.60
8 - 9	1894	6.79	6.54	42.71	25.50	17.21
9 - 10	1784	6.66	6.62	40.13	24.06	16.07
10 - 11	1400	6.42	7.14	31.34	19.06	12.28
11pm - 12mid	898	6.96	8.94	20.31	12.61	7.70
TOTALS:	52390			1269	719	551

Annual Fuel Savings 137,637 (gallons)

Annual Fuel Benefit $158,283 (@ $1.15 per gallon)

Table A-3. Fuel Consumption Data for Riverside & Congress (AGI) (Year 1)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.78	0.19	0.80	11.43	14.20	12mid - 1am
	1.33	0.16	0.68	7.35	9.51	1 - 2
	1.15	0.15	0.64	5.89	7.83	2 - 3
	0.71	0.13	0.57	2.85	4.26	3 - 4
	0.71	0.13	0.57	2.85	4.26	4 - 5
	1.08	0.14	0.63	5.39	7.25	5 - 6
	2.49	0.25	1.03	17.98	21.74	6 - 7
	12.69	1.58	6.87	66.93	88.08	7 - 8
	10.65	1.24	5.32	60.37	77.58	8 - 9
	6.80	0.70	2.95	45.21	55.66	9 - 10
	7.37	0.77	3.26	47.78	59.19	10 - 11
	10.83	1.27	5.45	60.99	78.54	11 - 12noon
	14.94	2.02	8.83	73.41	99.20	12 - 1pm
	13.40	1.71	7.46	69.04	91.61	1 - 2
	10.12	1.16	4.95	58.52	74.75	2 - 3
	10.97	1.29	5.55	61.45	79.27	3 - 4
	14.79	1.99	8.69	73.00	98.47	4 - 5
	20.87	3.50	15.58	88.01	127.96	5 - 6
	10.88	1.28	5.49	61.15	78.80	6 - 7
	6.97	0.72	3.05	46.01	56.75	7 - 8
	4.89	0.49	2.01	35.32	42.71	8 - 9
	4.55	0.45	1.86	33.27	40.13	9 - 10
	3.49	0.34	1.41	26.11	31.34	10 - 11
	2.35	0.24	0.98	16.75	20.31	11pm - 12mid
TOTALS:	176	22	95	977	1269	

Table A-4. Fuel Consumption Data for Riverside & Congress (GSI) (Year 1)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		N
	1.27	0.15	0.67	6.86	8.95	12mid - 1am
	0.95	0.14	0.61	4.41	6.10	1 - 2
	0.82	0.13	0.59	3.54	5.07	2 - 3
	0.49	0.12	0.55	1.71	2.87	3 - 4
	0.49	0.12	0.55	1.71	2.87	4 - 5
	0.77	0.13	0.58	3.23	4.71	5 - 6
	1.71	0.18	0.78	10.79	13.46	6 - 7
	5.77	0.58	2.43	40.16	48.94	7 - 8
	5.04	0.50	2.08	36.22	43.85	8 - 9
	3.63	0.35	1.46	27.12	32.57	9 - 10
	3.84	0.38	1.55	28.67	34.44	10 - 11
	5.11	0.51	2.11	36.59	44.32	11 - 12noon
	6.55	0.67	2.82	44.04	54.09	12 - 1pm
	6.01	0.61	2.55	41.43	50.60	1 - 2
	4.85	0.48	2.00	35.11	42.44	2 - 3
	5.16	0.51	2.14	36.87	44.68	3 - 4
	6.50	0.67	2.80	43.80	53.76	4 - 5
	8.58	0.93	3.97	52.81	66.29	5 - 6
	5.13	0.51	2.12	36.69	44.45	6 - 7
	3.69	0.36	1.49	27.61	33.15	7 - 8
	2.86	0.28	1.16	21.19	25.50	8 - 9
	2.72	0.27	1.11	19.96	24.06	9 - 10
	2.24	0.22	0.94	15.67	19.06	10 - 11
	1.63	0.18	0.75	10.05	12.61	11pm - 12mid
TOTALS:	86	9	38	586	$\boxed{719}$	

Table A-5. Delay Estimations for Riverside & Congress (Year 20)

Growth Rate 2.50%				System Delay (veh-hr/hr)		
Time of Day	Year 1 Volume	Daily Traffic Distribution	Year 20 Volume	At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	613	0.012	1004	2.44	1.82	0.62
1 - 2	394	0.008	646	1.88	1.56	0.32
2 - 3	316	0.006	518	1.71	1.48	0.24
3 - 4	153	0.003	251	1.41	1.31	0.10
4 - 5	153	0.003	251	1.41	1.31	0.10
5 - 6	289	0.006	474	1.66	1.45	0.21
6 - 7	964	0.018	1580	3.70	2.34	1.36
7 - 8	3589	0.069	5881	83.47	15.18	68.29
8 - 9	3237	0.062	5304	54.96	11.82	43.15
9 - 10	2424	0.046	3972	20.94	6.62	14.31
10 - 11	2562	0.049	4198	24.66	7.31	17.36
11 - 12noon	3270	0.062	5358	57.16	12.10	45.06
12 - 1pm	3936	0.075	6450	126.03	19.44	106.59
1 - 2	3702	0.071	6066	95.46	16.46	79.00
2 - 3	3138	0.060	5142	48.87	11.01	37.86
3 - 4	3295	0.063	5399	58.88	12.31	46.57
4 - 5	3914	0.075	6414	122.78	19.14	103.64
5 - 6	4719	0.090	7733	319.29	33.96	285.33
6 - 7	3279	0.063	5373	57.77	12.17	45.60
7 - 8	2467	0.047	4042	22.03	6.83	15.20
8 - 9	1894	0.036	3104	11.16	4.54	6.62
9 - 10	1784	0.034	2923	9.79	4.20	5.60
10 - 11	1400	0.027	2294	6.21	3.19	3.01
11pm - 12mid	898	0.017	1471	3.42	2.23	1.19
TOTALS:	52390	1.000	85847 ADT	1137.10	209.78	927.32

Annual Delay Savings 231,830 (veh-hr)   Annual User-Time Benefit $769,676 (@ $3.32 per veh-hr)

Table A-6. Fuel Consumption Estimates for Riverside & Congress (Year 20)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel consumed (gallons) At-Grade	Total Fuel consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	1004	6.72	8.37	22.62	13.98	8.64
1 - 2	646	8.06	11.15	14.89	9.37	5.53
2 - 3	518	9.17	13.15	12.16	7.71	4.45
3 - 4	251	15.60	24.18	6.41	4.20	2.21
4 - 5	251	15.60	24.18	6.41	4.20	2.21
5 - 6	474	9.71	14.10	11.22	7.14	4.08
6 - 7	1580	6.49	6.84	35.41	21.39	14.02
7 - 8	5881	39.31	11.92	185.77	86.23	99.54
8 - 9	5304	28.69	10.28	154.13	76.02	78.11
9 - 10	3972	14.60	7.69	100.40	54.63	45.77
10 - 11	4198	16.27	8.03	108.19	58.08	50.11
11 - 12noon	5358	29.54	10.42	156.82	76.95	79.87
12 - 1pm	6450	54.11	13.91	225.26	97.01	128.25
1 - 2	6066	43.58	12.52	197.57	89.66	107.91
2 - 3	5142	26.32	9.88	146.37	73.26	73.10
3 - 4	5399	30.20	10.53	158.88	77.66	81.23
4 - 5	6414	53.01	13.77	222.44	96.30	126.14
5 - 6	7733	114.34	20.27	368.12	124.76	243.36
6 - 7	5373	29.78	10.46	157.56	77.20	80.35
7 - 8	4042	15.09	7.80	102.78	55.70	47.08
8 - 9	3104	9.96	6.75	73.78	41.96	31.82
9 - 10	2923	9.28	6.63	68.78	39.43	29.35
10 - 11	2294	7.49	6.42	52.40	30.82	21.59
11pm - 12mid	1471	6.44	7.00	32.95	19.99	12.97
TOTALS:	85847			2621	1244	1378

Annual Fuel Savings 344,423 (gallons)

Annual Fuel Benefit $396,087 (@ $1.15 per gallon)

Table A-7. Fuel Consumption Data for Riverside & Congress (AGI) (Year 20)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	2.58	0.25	1.06	18.73	22.62	12mid - 1am
	1.85	0.19	0.81	12.04	14.89	1 - 2
	1.59	0.17	0.74	9.66	12.16	2 - 3
	0.99	0.14	0.61	4.68	6.41	3 - 4
	0.99	0.14	0.61	4.68	6.41	4 - 5
	1.50	0.17	0.72	8.83	11.22	5 - 6
	3.96	0.39	1.60	29.46	35.41	6 - 7
	31.94	8.00	36.15	109.68	185.77	7 - 8
	26.10	5.30	23.80	98.92	154.13	8 - 9
	15.19	2.07	9.07	74.08	100.40	9 - 10
	16.79	2.43	10.68	78.30	108.19	10 - 11
	26.62	5.51	24.75	99.93	156.82	11 - 12noon
	38.38	12.02	54.58	120.28	225.26	12 - 1pm
	33.96	9.13	41.34	113.13	197.57	1 - 2
	24.58	4.73	21.16	95.90	146.37	2 - 3
	27.01	5.67	25.50	100.70	158.88	3 - 4
	37.95	11.71	53.17	119.61	222.44	4 - 5
	55.40	30.22	138.28	144.21	368.12	5 - 6
	26.76	5.57	25.02	100.21	157.56	6 - 7
	15.67	2.17	9.54	75.39	102.78	7 - 8
	9.94	1.13	4.83	57.88	73.78	8 - 9
	9.02	1.00	4.24	54.52	68.78	9 - 10
	6.29	0.64	2.69	42.78	52.40	10 - 11
	3.67	0.36	1.48	27.44	32.95	11pm - 12mid
TOTALS:	419	109	492	1601	2621	

Table A-8. Fuel Consumption Data for Riverside & Congress (GSI) (Year 20)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.76	0.19	0.79	11.24	13.98	12mid - 1am
	1.31	0.16	0.68	7.22	9.37	1 - 2
	1.14	0.15	0.64	5.79	7.71	2 - 3
	0.70	0.13	0.57	2.81	4.20	3 - 4
	0.70	0.13	0.57	2.81	4.20	4 - 5
	1.07	0.14	0.63	5.30	7.14	5 - 6
	2.46	0.24	1.01	17.68	21.39	6 - 7
	12.33	1.52	6.58	65.81	86.23	7 - 8
	10.36	1.19	5.12	59.35	76.02	8 - 9
	6.63	0.68	2.87	44.45	54.63	9 - 10
	7.19	0.75	3.16	46.98	58.08	10 - 11
	10.53	1.22	5.24	59.96	76.95	11 - 12noon
	14.49	1.93	8.42	72.17	97.01	12 - 1pm
	13.01	1.64	7.13	67.88	89.66	1 - 2
	9.84	1.11	4.77	57.54	73.26	2 - 3
	10.67	1.24	5.33	60.42	77.66	3 - 4
	14.35	1.90	8.29	71.77	96.30	4 - 5
	20.21	3.31	14.71	86.53	124.76	5 - 6
	10.58	1.23	5.27	60.12	77.20	6 - 7
	6.80	0.70	2.96	45.24	55.70	7 - 8
	4.79	0.47	1.97	34.73	41.96	8 - 9
	4.46	0.44	1.82	32.71	39.43	9 - 10
	3.43	0.34	1.38	25.67	30.82	10 - 11
	2.32	0.23	0.97	16.47	19.99	11pm - 12mid
TOTALS:	171	21	91	961	1244	

Table A-9. Delay Estimations for Oltorf & Congress (Year 1)

Time of Day	Approach Volumes (veh/hr) Congress	Oltorf	Total	System Delay (veh-hr/hr) At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	125	198	323	1.49	1.36	0.13
1 - 2	177	138	315	1.48	1.35	0.13
2 - 3	173	136	309	1.47	1.35	0.13
3 - 4	79	59	138	1.30	1.25	0.05
4 - 5	90	49	139	1.30	1.25	0.05
5 - 6	168	114	282	1.44	1.33	0.11
6 - 7	492	307	799	2.10	1.67	0.43
7 - 8	1840	1039	2879	9.48	4.12	5.37
8 - 9	1341	1084	2425	6.83	3.38	3.45
9 - 10	1217	982	2199	5.79	3.06	2.73
10 - 11	1447	1022	2469	7.05	3.45	3.60
11 - 12noon	1800	1184	2984	10.23	4.31	5.92
12 - 1pm	1941	1474	3415	13.98	5.20	8.79
1 - 2	1843	1399	3242	12.34	4.82	7.52
2 - 3	1691	1251	2942	9.93	4.23	5.69
3 - 4	1676	1302	2978	10.19	4.30	5.89
4 - 5	1950	1495	3445	14.29	5.27	9.03
5 - 6	1935	1489	3424	14.08	5.22	8.86
6 - 7	1590	1294	2884	9.52	4.13	5.39
7 - 8	1063	1019	2082	5.32	2.91	2.41
8 - 9	919	795	1714	4.08	2.48	1.60
9 - 10	812	743	1555	3.63	2.32	1.32
10 - 11	591	564	1155	2.72	1.95	0.77
11pm - 12mid	400	331	731	2.00	1.62	0.38
TOTALS:	25360	19468	$\boxed{44828}$ ADT	152.05	72.30	$\boxed{79.75}$

Annual Delay Savings $\boxed{19,937}$ (veh-hr)   Annual User-Time Benefit $\boxed{\$66,190}$ (@ $3.32 per veh-hr)

Table A-10. Fuel Consumption Estimates for Oltorf & Congress (Year 1)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel Consumed (gallons) At-Grade	Total Fuel Consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	323	12.76	19.37	7.98	5.16	2.82
1 - 2	315	13.01	19.79	7.81	5.06	2.75
2 - 3	309	13.20	20.12	7.68	4.98	2.70
3 - 4	138	26.12	41.83	3.92	2.66	1.26
4 - 5	139	25.95	41.54	3.94	2.68	1.27
5 - 6	282	14.19	21.79	7.09	4.62	2.47
6 - 7	799	7.28	9.63	18.18	11.34	6.84
7 - 8	2879	9.12	6.60	67.58	38.81	28.77
8 - 9	2425	7.79	6.43	55.69	32.58	23.11
9 - 10	2199	7.30	6.43	50.05	29.54	20.51
10 - 11	2469	7.90	6.44	56.80	33.18	23.63
11 - 12noon	2984	9.50	6.67	70.45	40.27	30.17
12 - 1pm	3415	11.34	7.02	82.80	46.41	36.40
1 - 2	3242	10.54	6.86	77.73	43.92	33.81
2 - 3	2942	9.34	6.64	69.29	39.69	29.61
3 - 4	2978	9.47	6.66	70.28	40.19	30.09
4 - 5	3445	11.49	7.05	83.70	46.84	36.86
5 - 6	3424	11.38	7.03	83.07	46.54	36.54
6 - 7	2884	9.14	6.60	67.71	38.88	28.83
7 - 8	2082	7.08	6.45	47.20	27.99	19.22
8 - 9	1714	6.59	6.68	38.50	23.14	15.36
9 - 10	1555	6.47	6.87	34.85	21.07	13.78
10 - 11	1155	6.52	7.78	25.91	15.91	10.00
11pm - 12mid	731	7.58	10.22	16.72	10.47	6.26
TOTALS:	44828			1055	612	443

Annual Fuel Savings 110,756 (gallons)

Annual Fuel Benefit $127,369 (@ $1.15 per gallon)

Table A-11. Fuel Consumption Data for Oltorf & Congress (AGI) (Year 1)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.16	0.15	0.64	6.02	7.98	12mid - 1am
	1.15	0.15	0.64	5.87	7.81	1 - 2
	1.13	0.15	0.64	5.76	7.68	2 - 3
	0.66	0.13	0.56	2.57	3.92	3 - 4
	0.66	0.13	0.56	2.59	3.94	4 - 5
	1.07	0.14	0.63	5.26	7.09	5 - 6
	2.15	0.22	0.91	14.90	18.18	6 - 7
	8.81	0.97	4.11	53.69	67.58	7 - 8
	6.80	0.70	2.96	45.23	55.69	8 - 9
	5.93	0.60	2.51	41.01	50.05	9 - 10
	6.98	0.73	3.05	46.05	56.80	10 - 11
	9.33	1.04	4.43	55.65	70.45	11 - 12noon
	11.66	1.40	6.06	63.69	82.80	12 - 1pm
	10.68	1.24	5.34	60.46	77.73	1 - 2
	9.12	1.01	4.30	54.87	69.29	2 - 3
	9.30	1.03	4.41	55.54	70.28	3 - 4
	11.83	1.43	6.19	64.25	83.70	4 - 5
	11.71	1.41	6.10	63.86	83.07	5 - 6
	8.83	0.97	4.12	53.79	67.71	6 - 7
	5.51	0.55	2.31	38.83	47.20	7 - 8
	4.34	0.43	1.77	31.97	38.50	8 - 9
	3.89	0.38	1.57	29.00	34.85	9 - 10
	2.90	0.29	1.18	21.54	25.91	10 - 11
	2.02	0.21	0.87	13.63	16.72	11pm - 12mid
TOTALS:	138	15	66	836	1055	

Table A-12. Fuel Consumption Data for Oltorf & Congress (GSI) (Year 1)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	0.83	0.13	0.59	3.61	5.16	12mid - 1am
	0.82	0.13	0.58	3.52	5.06	1 - 2
	0.81	0.13	0.58	3.46	4.98	2 - 3
	0.46	0.12	0.54	1.54	2.66	3 - 4
	0.46	0.12	0.54	1.56	2.68	4 - 5
	0.76	0.13	0.58	3.16	4.62	5 - 6
	1.51	0.17	0.72	8.94	11.34	6 - 7
	4.38	0.43	1.78	32.22	38.81	7 - 8
	3.63	0.35	1.46	27.14	32.58	8 - 9
	3.29	0.32	1.33	24.61	29.54	9 - 10
	3.70	0.36	1.49	27.63	33.18	10 - 11
	4.57	0.45	1.87	33.39	40.27	11 - 12noon
	5.40	0.54	2.25	38.21	46.41	12 - 1pm
	5.05	0.50	2.09	36.28	43.92	1 - 2
	4.49	0.44	1.83	32.92	39.69	2 - 3
	4.56	0.45	1.86	33.32	40.19	3 - 4
	5.46	0.55	2.28	38.55	46.84	4 - 5
	5.42	0.54	2.26	38.31	46.54	5 - 6
	4.39	0.43	1.79	32.27	38.88	6 - 7
	3.12	0.31	1.26	23.30	27.99	7 - 8
	2.63	0.26	1.07	19.18	23.14	8 - 9
	2.43	0.24	1.00	17.40	21.07	9 - 10
	1.94	0.20	0.84	12.92	15.91	10 - 11
	1.42	0.16	0.70	8.18	10.47	11pm - 12mid
TOTALS:	72	7	31	502	612	

Table A-13. Delay Estimations for Oltorf & Congress (Year 20)

Growth Rate 2.50%

Time of Day	Year 1 Volume	Daily Traffic Distribution	Year 20 Volume	System Delay (veh-hr/hr) At-Grade	System Delay (veh-hr/hr) Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	323	0.007	529	1.73	1.48	0.25
1 - 2	315	0.007	516	1.71	1.47	0.24
2 - 3	309	0.007	506	1.70	1.47	0.23
3 - 4	138	0.003	226	1.39	1.30	0.09
4 - 5	139	0.003	228	1.39	1.30	0.09
5 - 6	282	0.006	462	1.65	1.44	0.21
6 - 7	799	0.018	1309	3.04	2.08	0.96
7 - 8	2879	0.064	4718	35.93	9.16	26.78
8 - 9	2425	0.054	3974	20.96	6.63	14.33
9 - 10	2199	0.049	3603	16.03	5.64	10.39
10 - 11	2469	0.055	4046	22.08	6.84	15.25
11 - 12noon	2984	0.067	4890	40.70	9.87	30.83
12 - 1pm	3415	0.076	5596	67.90	13.41	54.48
1 - 2	3242	0.072	5312	55.29	11.86	43.43
2 - 3	2942	0.066	4821	38.72	9.58	29.15
3 - 4	2978	0.066	4880	40.41	9.83	30.59
4 - 5	3445	0.077	5645	70.36	13.70	56.65
5 - 6	3424	0.076	5611	68.62	13.50	55.13
6 - 7	2884	0.064	4726	36.15	9.19	26.96
7 - 8	2082	0.046	3412	13.95	5.19	8.76
8 - 9	1714	0.038	2809	9.01	3.99	5.02
9 - 10	1555	0.035	2548	7.46	3.57	3.90
10 - 11	1155	0.026	1893	4.64	2.68	1.96
11pm - 12mid	731	0.016	1198	2.81	1.98	0.82
TOTALS:	44828	1.000	73456 ADT	563.63	147.15	416.48

Annual Delay Savings: 104,120 (veh-hr)

Annual User-Time Benefit (@ $3.32 per veh-hr): $345,677

Table A-14. Fuel Consumption Estimates for Oltorf & Congress (Year 20)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel consumed (gallons) At-Grade	Total Fuel consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	529	9.04	12.93	12.41	7.86	4.54
1 - 2	516	9.18	13.18	12.13	7.69	4.43
2 - 3	506	9.30	13.38	11.92	7.57	4.35
3 - 4	226	16.99	26.52	5.87	3.87	2.01
4 - 5	228	16.89	26.35	5.91	3.89	2.02
5 - 6	462	9.87	14.38	10.97	6.99	3.98
6 - 7	1309	6.43	7.34	29.32	17.89	11.42
7 - 8	4718	21.09	8.96	127.90	66.27	61.63
8 - 9	3974	14.61	7.70	100.45	54.66	45.80
9 - 10	3603	12.32	7.23	88.53	49.15	39.38
10 - 11	4046	15.12	7.80	102.89	55.75	47.15
11 - 12noon	4890	23.05	9.31	135.09	69.07	66.02
12 - 1pm	5596	33.60	11.06	169.26	81.10	88.16
1 - 2	5312	28.82	10.30	154.53	76.16	78.37
2 - 3	4821	22.24	9.17	132.17	67.94	64.23
3 - 4	4880	22.93	9.29	134.67	68.91	65.76
4 - 5	5645	34.51	11.20	171.98	81.97	90.01
5 - 6	5611	33.87	11.10	170.07	81.36	88.71
6 - 7	4726	21.18	8.97	128.23	66.40	61.83
7 - 8	3412	11.32	7.02	82.70	46.36	36.35
8 - 9	2809	8.89	6.56	65.68	37.83	27.85
9 - 10	2548	8.11	6.46	58.83	34.25	24.58
10 - 11	1893	6.79	6.54	42.68	25.48	17.19
11pm - 12mid	1198	6.49	7.64	26.85	16.46	10.39
TOTALS:	73456			1981	1035	946

Annual Fuel Savings 236,543 (gallons)

Annual Fuel Benefit $272,024 (@ $1.15 per gallon)

Table A-15. Fuel Consumption Data for Oltorf & Congress (AGI) (Year 20)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.61	0.18	0.75	9.87	12.41	12mid - 1am
	1.59	0.17	0.74	9.63	12.13	1 - 2
	1.56	0.17	0.74	9.44	11.92	2 - 3
	0.92	0.14	0.60	4.22	5.87	3 - 4
	0.92	0.14	0.60	4.25	5.91	4 - 5
	1.47	0.17	0.71	8.62	10.97	5 - 6
	3.26	0.32	1.32	24.42	29.32	6 - 7
	20.86	3.50	15.56	87.98	127.90	7 - 8
	15.20	2.07	9.08	74.11	100.45	8 - 9
	12.78	1.60	6.94	67.20	88.53	9 - 10
	15.70	2.18	9.56	75.45	102.89	10 - 11
	22.32	3.95	17.63	91.19	135.09	11 - 12noon
	28.97	6.53	29.40	104.36	169.26	12 - 1pm
	26.18	5.33	23.94	99.08	154.53	1 - 2
	21.73	3.76	16.77	89.91	132.17	2 - 3
	22.24	3.92	17.50	91.01	134.67	3 - 4
	29.47	6.76	30.47	105.28	171.98	4 - 5
	29.12	6.60	29.72	104.64	170.07	5 - 6
	20.92	3.52	15.65	88.14	128.23	6 - 7
	11.64	1.40	6.04	63.63	82.70	7 - 8
	8.47	0.92	3.90	52.38	65.68	8 - 9
	7.31	0.77	3.23	47.52	58.83	9 - 10
	4.88	0.48	2.01	35.30	42.68	10 - 11
	3.00	0.29	1.21	22.34	26.85	11pm - 12mid
TOTALS:	312	55	244	1370	$\boxed{1981}$	

Table A-16. Fuel Consumption Data for Oltorf & Congress (GSI) (Year 20)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.15	0.15	0.64	5.92	7.86	12mid - 1am
	1.13	0.15	0.64	5.78	7.69	1 - 2
	1.12	0.15	0.64	5.67	7.57	2 - 3
	0.65	0.13	0.56	2.53	3.87	3 - 4
	0.65	0.13	0.56	2.55	3.89	4 - 5
	1.05	0.14	0.62	5.17	6.99	5 - 6
	2.13	0.22	0.90	14.65	17.89	6 - 7
	8.58	0.93	3.97	52.79	66.27	7 - 8
	6.64	0.68	2.87	44.47	54.66	8 - 9
	5.80	0.59	2.44	40.32	49.15	9 - 10
	6.81	0.70	2.96	45.27	55.75	10 - 11
	9.08	1.00	4.27	54.71	69.07	11 - 12noon
	11.33	1.35	5.81	62.62	81.10	12 - 1pm
	10.38	1.20	5.14	59.45	76.16	1 - 2
	8.87	0.97	4.15	53.94	67.94	2 - 3
	9.05	1.00	4.26	54.60	68.91	3 - 4
	11.49	1.37	5.93	63.17	81.97	4 - 5
	11.38	1.36	5.85	62.78	81.36	5 - 6
	8.60	0.94	3.98	52.88	66.40	6 - 7
	5.39	0.54	2.25	38.18	46.36	7 - 8
	4.26	0.42	1.73	31.43	37.83	8 - 9
	3.82	0.37	1.54	28.51	34.25	9 - 10
	2.86	0.28	1.16	21.18	25.48	10 - 11
	1.99	0.20	0.86	13.40	16.46	11pm - 12mid
TOTALS:	134	15	64	822	$\boxed{1035}$	

Table A-17. Delay Estimations for Stassney & Congress (Year 1)

Time of Day	Approach Volumes (veh/hr) Congress	Stassney	Total	System Delay (veh-hr/hr) At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	100	175	275	1.44	1.33	0.11
1 - 2	68	99	167	1.33	1.27	0.06
2 - 3	60	80	140	1.30	1.25	0.05
3 - 4	56	50	106	1.27	1.23	0.04
4 - 5	40	65	105	1.27	1.23	0.04
5 - 6	120	174	294	1.46	1.34	0.12
6 - 7	373	699	1072	2.56	1.88	0.68
7 - 8	1208	1701	2909	9.69	4.17	5.52
8 - 9	759	1208	1967	4.90	2.77	2.13
9 - 10	675	879	1554	3.63	2.31	1.32
10 - 11	750	848	1598	3.75	2.36	1.39
11 - 12noon	912	1001	1913	4.71	2.71	2.00
12 - 1pm	1043	1215	2258	6.05	3.14	2.90
1 - 2	963	1178	2141	5.56	2.99	2.57
2 - 3	944	1202	2146	5.58	2.99	2.58
3 - 4	1034	1360	2394	6.67	3.33	3.34
4 - 5	1228	1870	3098	11.11	4.53	6.59
5 - 6	1533	1981	3514	15.02	5.43	9.60
6 - 7	1014	1507	2521	7.32	3.52	3.79
7 - 8	657	1194	1851	4.50	2.63	1.87
8 - 9	516	832	1348	3.13	2.12	1.01
9 - 10	404	677	1081	2.58	1.88	0.69
10 - 11	259	506	765	2.05	1.64	0.41
11pm - 12mid	164	310	474	1.66	1.45	0.21
TOTALS:	14880	20811	$\boxed{35691}$ ADT	108.54	59.51	$\boxed{49.03}$

Annual Delay Savings $\boxed{12,257}$ (veh-hr)

Annual User-Time Benefit $\boxed{\$40,693}$ (@ $3.32 per veh-hr)

Table A-18. Fuel Consumption Estimates for Stassney & Congress (Year 1)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel Consumed (gallons) At-Grade	Total Fuel Consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	275	14.48	22.28	6.94	4.53	2.42
1 - 2	167	22.04	35.00	4.57	3.06	1.51
2 - 3	140	25.78	41.27	3.97	2.69	1.27
3 - 4	106	33.23	53.70	3.20	2.22	0.98
4 - 5	105	33.52	54.19	3.17	2.20	0.97
5 - 6	294	13.73	21.01	7.35	4.78	2.57
6 - 7	1072	6.62	8.08	24.09	14.84	9.25
7 - 8	2909	9.23	6.62	68.39	39.23	29.16
8 - 9	1967	6.90	6.50	44.44	26.46	17.98
9 - 10	1554	6.47	6.87	34.82	21.06	13.77
10 - 11	1598	6.50	6.81	35.83	21.63	14.20
11 - 12noon	1913	6.82	6.53	43.16	25.75	17.41
12 - 1pm	2258	7.42	6.42	51.51	30.33	21.17
1 - 2	2141	7.19	6.44	48.64	28.77	19.87
2 - 3	2146	7.20	6.44	48.76	28.84	19.92
3 - 4	2394	7.72	6.43	54.90	32.16	22.74
4 - 5	3098	9.93	6.75	73.62	41.88	31.75
5 - 6	3514	11.84	7.13	85.78	47.84	37.94
6 - 7	2521	8.04	6.45	58.13	33.88	24.25
7 - 8	1851	6.74	6.57	41.70	24.93	16.76
8 - 9	1348	6.43	7.25	30.18	18.39	11.79
9 - 10	1081	6.60	8.05	24.29	14.96	9.33
10 - 11	765	7.42	9.91	17.45	10.90	6.55
11pm - 12mid	474	9.70	14.09	11.23	7.15	4.08
TOTALS:	35691			826	488	338

Annual Fuel Savings (gallons): 84,409

Annual Fuel Benefit (@ $1.15 per gallon): $97,070

Table A-19. Fuel Consumption Data for Stassney & Congress (AGI) (Year 1)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.05	0.14	0.62	5.13	6.94	12mid - 1am
	0.75	0.13	0.58	3.11	4.57	1 - 2
	0.66	0.13	0.56	2.61	3.97	2 - 3
	0.55	0.12	0.55	1.98	3.20	3 - 4
	0.54	0.12	0.55	1.96	3.17	4 - 5
	1.09	0.14	0.63	5.48	7.35	5 - 6
	2.72	0.27	1.11	19.99	24.09	6 - 7
	8.96	0.99	4.20	54.25	68.39	7 - 8
	5.13	0.51	2.12	36.68	44.44	8 - 9
	3.89	0.38	1.57	28.98	34.82	9 - 10
	4.01	0.39	1.62	29.80	35.83	10 - 11
	4.95	0.49	2.04	35.68	43.16	11 - 12noon
	6.15	0.63	2.62	42.11	51.51	12 - 1pm
	5.72	0.58	2.41	39.93	48.64	1 - 2
	5.74	0.58	2.41	40.02	48.76	2 - 3
	6.68	0.69	2.89	44.65	54.90	3 - 4
	9.91	1.12	4.81	57.78	73.62	4 - 5
	12.24	1.50	6.51	65.54	85.78	5 - 6
	7.20	0.75	3.17	47.02	58.13	6 - 7
	4.75	0.47	1.95	34.52	41.70	7 - 8
	3.36	0.33	1.35	25.14	30.18	8 - 9
	2.74	0.27	1.12	20.16	24.29	9 - 10
	2.08	0.21	0.89	14.27	17.45	10 - 11
	1.50	0.17	0.72	8.84	11.23	11pm - 12mid
TOTALS:	102	11	47	666	$\boxed{826}$	

Table A-20. Fuel Consumption Data for Stassney & Congress (GSI) (Year 1)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	0.74	0.13	0.57	3.08	4.53	12mid - 1am
	0.53	0.12	0.55	1.87	3.06	1 - 2
	0.46	0.12	0.54	1.57	2.69	2 - 3
	0.38	0.12	0.53	1.19	2.22	3 - 4
	0.38	0.12	0.53	1.17	2.20	4 - 5
	0.78	0.13	0.58	3.29	4.78	5 - 6
	1.84	0.19	0.81	12.00	14.84	6 - 7
	4.43	0.44	1.81	32.55	39.23	7 - 8
	2.96	0.29	1.20	22.01	26.46	8 - 9
	2.42	0.24	1.00	17.39	21.06	9 - 10
	2.48	0.25	1.02	17.88	21.63	10 - 11
	2.89	0.28	1.17	21.41	25.75	11 - 12noon
	3.37	0.33	1.36	25.27	30.33	12 - 1pm
	3.20	0.31	1.29	23.96	28.77	1 - 2
	3.21	0.31	1.30	24.01	28.84	2 - 3
	3.58	0.35	1.44	26.79	32.16	3 - 4
	4.78	0.47	1.96	34.67	41.88	4 - 5
	5.61	0.56	2.35	39.32	47.84	5 - 6
	3.78	0.37	1.53	28.21	33.88	6 - 7
	2.81	0.28	1.14	20.71	24.93	7 - 8
	2.17	0.22	0.92	15.08	18.39	8 - 9
	1.85	0.19	0.82	12.10	14.96	9 - 10
	1.47	0.17	0.71	8.56	10.90	10 - 11
	1.07	0.14	0.63	5.30	7.15	11pm - 12mid
TOTALS:	57	6	26	399	$\boxed{488}$	

Table A-21. Delay Estimations for Stassney & Congress (Year 20)

Growth Rate 2.50%				System Delay (veh-hr/hr)		
Time of Day	Year 1 Volume	Daily Traffic Distribution	Year 20 Volume	At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	275	0.008	451	1.63	1.43	0.20
1 - 2	167	0.005	274	1.44	1.33	0.11
2 - 3	140	0.004	229	1.39	1.30	0.09
3 - 4	106	0.003	174	1.34	1.27	0.07
4 - 5	105	0.003	172	1.33	1.27	0.06
5 - 6	294	0.008	482	1.67	1.45	0.22
6 - 7	1072	0.030	1757	4.21	2.53	1.68
7 - 8	2909	0.082	4767	37.23	9.35	27.88
8 - 9	1967	0.055	3223	12.17	4.78	7.39
9 - 10	1554	0.044	2546	7.45	3.56	3.89
10 - 11	1598	0.045	2619	7.85	3.68	4.18
11 - 12noon	1913	0.054	3135	11.41	4.60	6.81
12 - 1pm	2258	0.063	3700	17.19	5.88	11.31
1 - 2	2141	0.060	3508	14.96	5.41	9.55
2 - 3	2146	0.060	3516	15.05	5.43	9.62
3 - 4	2394	0.067	3923	20.20	6.48	13.72
4 - 5	3098	0.087	5076	46.60	10.70	35.90
5 - 6	3514	0.098	5758	76.36	14.39	61.97
6 - 7	2521	0.071	4131	23.49	7.10	16.40
7 - 8	1851	0.052	3033	10.60	4.40	6.20
8 - 9	1348	0.038	2209	5.84	3.08	2.76
9 - 10	1081	0.030	1771	4.25	2.54	1.71
10 - 11	765	0.021	1254	2.92	2.03	0.89
11pm - 12mid	474	0.013	777	2.07	1.65	0.42
TOTALS:	35691	1.000	58484 ADT	328.67	105.66	223.00

Annual Delay Savings 55,751 (veh-hr)

Annual User-Time Benefit $185,094 (@ $3.32 per veh-hr)

Table A-22. Fuel Consumption Estimates for Stassney & Congress (Year 20)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel consumed (gallons) At-Grade	Total Fuel consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	451	10.03	14.67	10.72	6.84	3.88
1 - 2	274	14.53	22.37	6.91	4.51	2.40
2 - 3	229	16.79	26.18	5.95	3.91	2.03
3 - 4	174	21.30	33.75	4.72	3.16	1.56
4 - 5	172	21.47	34.05	4.68	3.13	1.55
5 - 6	482	9.60	13.91	11.39	7.25	4.14
6 - 7	1757	6.63	6.64	39.49	23.70	15.79
7 - 8	4767	21.63	9.06	129.92	67.06	62.85
8 - 9	3223	10.46	6.85	77.19	43.65	33.53
9 - 10	2546	8.10	6.46	58.79	34.23	24.56
10 - 11	2619	8.30	6.48	60.65	35.21	25.44
11 - 12noon	3135	10.08	6.77	74.66	42.40	32.26
12 - 1pm	3700	12.87	7.34	91.55	50.57	40.98
1 - 2	3508	11.81	7.12	85.61	47.76	37.85
2 - 3	3516	11.85	7.13	85.86	47.88	37.98
3 - 4	3923	14.26	7.63	98.76	53.89	44.87
4 - 5	5076	25.42	9.73	143.35	72.16	71.19
5 - 6	5758	36.73	11.54	178.43	84.00	94.43
6 - 7	4131	15.75	7.93	105.83	57.05	48.78
7 - 8	3033	9.68	6.70	71.81	40.96	30.84
8 - 9	2209	7.32	6.43	50.30	29.68	20.62
9 - 10	1771	6.64	6.63	39.83	23.89	15.94
10 - 11	1254	6.45	7.48	28.08	17.18	10.91
11pm - 12mid	777	7.37	9.81	17.70	11.05	6.65
TOTALS:	58484			1482	811	671

Annual Fuel Savings 167,763 (gallons)

Annual Fuel Benefit $192,927 (@ $1.15 per gallon)

Table A-23. Fuel Consumption Data for Stassney & Congress (AGI) (Year 20)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.45	0.17	0.71	8.40	10.72	12mid - 1am
	1.04	0.14	0.62	5.10	6.91	1 - 2
	0.93	0.14	0.60	4.28	5.95	2 - 3
	0.77	0.13	0.58	3.24	4.72	3 - 4
	0.77	0.13	0.58	3.21	4.68	4 - 5
	1.51	0.17	0.72	8.98	11.39	5 - 6
	4.47	0.44	1.82	32.76	39.49	6 - 7
	21.27	3.62	16.13	88.90	129.92	7 - 8
	10.58	1.23	5.27	60.11	77.19	8 - 9
	7.30	0.77	3.23	47.49	58.79	9 - 10
	7.61	0.81	3.40	48.84	60.65	10 - 11
	10.10	1.15	4.94	58.46	74.66	11 - 12noon
	13.39	1.71	7.44	69.00	91.55	12 - 1pm
	12.21	1.50	6.48	65.43	85.61	1 - 2
	12.25	1.50	6.52	65.58	85.86	2 - 3
	14.85	2.00	8.75	73.16	98.76	3 - 4
	23.98	4.51	20.18	94.68	143.35	4 - 5
	30.64	7.33	33.07	107.39	178.43	5 - 6
	16.30	2.31	10.17	77.04	105.83	6 - 7
	9.57	1.07	4.59	56.57	71.81	7 - 8
	5.97	0.61	2.53	41.20	50.30	8 - 9
	4.51	0.44	1.84	33.04	39.83	9 - 10
	3.13	0.31	1.26	23.38	28.08	10 - 11
	2.11	0.21	0.90	14.49	17.70	11pm - 12mid
TOTALS:	217	32	142	1091	$\boxed{1482}$	

Table A-24. Fuel Consumption Data for Stassney & Congress (GSI) (Year 20)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.04	0.14	0.62	5.04	6.84	12mid - 1am
	0.74	0.13	0.57	3.06	4.51	1 - 2
	0.66	0.13	0.56	2.57	3.91	2 - 3
	0.54	0.12	0.55	1.94	3.16	3 - 4
	0.54	0.12	0.55	1.93	3.13	4 - 5
	1.08	0.14	0.63	5.39	7.25	5 - 6
	2.68	0.26	1.09	19.66	23.70	6 - 7
	8.72	0.95	4.05	53.34	67.06	7 - 8
	5.02	0.50	2.07	36.07	43.65	8 - 9
	3.82	0.37	1.54	28.49	34.23	9 - 10
	3.94	0.39	1.59	29.30	35.21	10 - 11
	4.85	0.48	1.99	35.08	42.40	11 - 12noon
	6.01	0.61	2.55	41.40	50.57	12 - 1pm
	5.59	0.56	2.34	39.26	47.76	1 - 2
	5.61	0.56	2.35	39.35	47.88	2 - 3
	6.52	0.67	2.81	43.90	53.89	3 - 4
	9.64	1.08	4.63	56.81	72.16	4 - 5
	11.89	1.44	6.23	64.43	84.00	5 - 6
	7.02	0.73	3.07	46.23	57.05	6 - 7
	4.66	0.46	1.91	33.94	40.96	7 - 8
	3.30	0.32	1.33	24.72	29.68	8 - 9
	2.70	0.27	1.10	19.82	23.89	9 - 10
	2.06	0.21	0.88	14.03	17.18	10 - 11
	1.48	0.17	0.71	8.69	11.05	11pm - 12mid
TOTALS:	100	11	46	654	811	

Table A-25. Delay Estimations for William Cannon & Congress (Year 1)

Time of Day	Approach Volumes (veh/hr) Congress	William Cannon	Total	System Delay (veh-hr/hr) At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	77	505	582	1.80	1.52	0.28
1 - 2	57	292	349	1.52	1.37	0.15
2 - 3	43	251	294	1.46	1.34	0.12
3 - 4	28	168	196	1.36	1.28	0.07
4 - 5	37	201	238	1.40	1.31	0.09
5 - 6	121	467	588	1.80	1.52	0.28
6 - 7	394	1645	2039	5.16	2.86	2.30
7 - 8	729	3269	3998	21.33	6.70	14.64
8 - 9	536	2338	2874	9.45	4.11	5.34
9 - 10	514	1693	2207	5.83	3.07	2.75
10 - 11	520	1682	2202	5.81	3.07	2.74
11 - 12noon	610	1974	2584	7.66	3.62	4.04
12 - 1pm	691	2208	2899	9.62	4.15	5.47
1 - 2	668	2165	2833	9.17	4.04	5.14
2 - 3	657	2163	2820	9.09	4.01	5.07
3 - 4	797	2402	3199	11.96	4.73	7.23
4 - 5	1015	2985	4000	21.36	6.70	14.66
5 - 6	1192	3176	4368	27.89	7.87	20.03
6 - 7	827	2894	3721	17.45	5.94	11.52
7 - 8	518	2169	2687	8.25	3.79	4.46
8 - 9	351	1975	2326	6.35	3.24	3.12
9 - 10	318	1662	1980	4.94	2.79	2.16
10 - 11	210	1196	1406	3.26	2.17	1.09
11pm - 12mid	128	788	916	2.29	1.75	0.53
TOTALS:	11038	40268	51306 ADT	196.21	82.94	113.28

Annual Delay Savings: 28,320 (veh-hr)

Annual User-Time Benefit: $94,021 (@ $3.32 per veh-hr)

Table A-26. Fuel Consumption Estimates for William Cannon & Congress (Year 1)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel Consumed (gallons) At-Grade	Total Fuel Consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	582	8.54	12.03	13.53	8.55	4.99
1 - 2	349	12.03	18.13	8.54	5.51	3.03
2 - 3	294	13.73	21.01	7.35	4.78	2.57
3 - 4	196	19.18	30.20	5.21	3.46	1.75
4 - 5	238	16.28	25.33	6.13	4.03	2.11
5 - 6	588	8.49	11.94	13.66	8.62	5.04
6 - 7	2039	7.01	6.47	46.17	27.41	18.75
7 - 8	3998	14.78	7.73	101.27	55.02	46.25
8 - 9	2874	9.10	6.60	67.44	38.74	28.70
9 - 10	2207	7.31	6.43	50.25	29.65	20.60
10 - 11	2202	7.30	6.43	50.13	29.58	20.54
11 - 12noon	2584	8.21	6.47	59.76	34.74	25.02
12 - 1pm	2899	9.19	6.61	68.12	39.09	29.03
1 - 2	2833	8.97	6.57	66.33	38.17	28.16
2 - 3	2820	8.92	6.57	65.98	37.99	27.99
3 - 4	3199	10.35	6.83	76.49	43.31	33.18
4 - 5	4000	14.79	7.73	101.34	55.05	46.29
5 - 6	4368	17.68	8.31	114.34	60.71	53.62
6 - 7	3721	12.99	7.36	92.21	50.88	41.33
7 - 8	2687	8.50	6.51	62.45	36.15	26.29
8 - 9	2326	7.56	6.42	53.20	31.25	21.95
9 - 10	1980	6.91	6.49	44.75	26.63	18.12
10 - 11	1406	6.42	7.12	31.48	19.14	12.34
11pm - 12mid	916	6.91	8.84	20.70	12.84	7.86
TOTALS:	51306			1227	701	526

Annual Fuel Savings 131,382 (gallons)

Annual Fuel Benefit $151,089 (@ $1.15 per gallon)

Table A-27. Fuel Consumption Data for William Cannon & Congress (AGI) (Year 1)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.72	0.18	0.78	10.85	13.53	12mid - 1am
	1.22	0.15	0.66	6.51	8.54	1 - 2
	1.09	0.14	0.63	5.48	7.35	2 - 3
	0.84	0.13	0.59	3.66	5.21	3 - 4
	0.95	0.14	0.61	4.44	6.13	4 - 5
	1.73	0.18	0.78	10.97	13.66	5 - 6
	5.37	0.54	2.23	38.03	46.17	6 - 7
	15.36	2.11	9.24	74.56	101.27	7 - 8
	8.79	0.96	4.09	53.60	67.44	8 - 9
	5.96	0.60	2.52	41.16	50.25	9 - 10
	5.94	0.60	2.51	41.07	50.13	10 - 11
	7.46	0.79	3.32	48.19	59.76	11 - 12noon
	8.91	0.98	4.17	54.07	68.12	12 - 1pm
	8.59	0.93	3.97	52.84	66.33	1 - 2
	8.53	0.93	3.94	52.59	65.98	2 - 3
	10.45	1.21	5.18	59.66	76.49	3 - 4
	15.38	2.11	9.25	74.60	101.34	4 - 5
	18.06	2.73	12.08	81.46	114.34	5 - 6
	13.52	1.74	7.56	69.40	92.21	6 - 7
	7.92	0.84	3.57	50.11	62.45	7 - 8
	6.41	0.66	2.75	43.38	53.20	8 - 9
	5.17	0.52	2.14	36.93	44.75	9 - 10
	3.50	0.34	1.41	26.22	31.48	10 - 11
	2.39	0.24	0.99	17.08	20.70	11pm - 12mid
TOTALS:	165	20	85	957	1227	

Table A-28. Fuel Consumption Data for William Cannon & Congress (GSI) (Year 1)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.23	0.15	0.66	6.51	8.55	12mid - 1am
	0.87	0.13	0.59	3.91	5.51	1 - 2
	0.78	0.13	0.58	3.29	4.78	2 - 3
	0.59	0.12	0.56	2.19	3.46	3 - 4
	0.67	0.13	0.57	2.66	4.03	4 - 5
	1.23	0.15	0.66	6.58	8.62	5 - 6
	3.06	0.30	1.24	22.82	27.41	6 - 7
	6.70	0.69	2.90	44.74	55.02	7 - 8
	4.37	0.43	1.78	32.16	38.74	8 - 9
	3.30	0.32	1.33	24.70	29.65	9 - 10
	3.29	0.32	1.33	24.64	29.58	10 - 11
	3.88	0.38	1.57	28.91	34.74	11 - 12noon
	4.41	0.43	1.80	32.44	39.09	12 - 1pm
	4.30	0.42	1.75	31.70	38.17	1 - 2
	4.28	0.42	1.74	31.56	37.99	2 - 3
	4.97	0.49	2.05	35.80	43.31	3 - 4
	6.70	0.69	2.90	44.76	55.05	4 - 5
	7.62	0.81	3.41	48.88	60.71	5 - 6
	6.06	0.62	2.57	41.64	50.88	6 - 7
	4.05	0.40	1.64	30.07	36.15	7 - 8
	3.48	0.34	1.40	26.03	31.25	8 - 9
	2.98	0.29	1.21	22.16	26.63	9 - 10
	2.24	0.23	0.94	15.73	19.14	10 - 11
	1.65	0.18	0.76	10.25	12.84	11pm - 12mid
TOTALS:	83	9	36	574	701	

Table A-2~~5~~9. Delay Estimations for William Cannon & Congress (Year 20)

Growth Rate 2.50%				System Delay (veh-hr/hr)		
Time of Day	Year 1 Volume	Daily Traffic Distribution	Year 20 Volume	At-Grade	Grade Separation	Delay Savings (veh-hr/hr)
12mid - 1am	582	0.011	954	2.35	1.78	0.57
1 - 2	349	0.007	572	1.78	1.51	0.27
2 - 3	294	0.006	482	1.67	1.45	0.22
3 - 4	196	0.004	321	1.49	1.35	0.13
4 - 5	238	0.005	390	1.56	1.40	0.17
5 - 6	588	0.011	964	2.37	1.79	0.58
6 - 7	2039	0.040	3341	13.25	5.03	8.22
7 - 8	3998	0.078	6551	135.66	20.32	115.34
8 - 9	2874	0.056	4709	35.72	9.12	26.60
9 - 10	2207	0.043	3616	16.18	5.67	10.51
10 - 11	2202	0.043	3608	16.08	5.65	10.43
11 - 12noon	2584	0.050	4234	25.32	7.42	17.89
12 - 1pm	2899	0.057	4750	36.80	9.29	27.51
1 - 2	2833	0.055	4642	34.02	8.86	25.16
2 - 3	2820	0.055	4621	33.50	8.78	24.72
3 - 4	3199	0.062	5242	52.54	11.50	41.04
4 - 5	4000	0.078	6554	135.98	20.35	115.63
5 - 6	4368	0.085	7157	210.48	26.45	184.03
6 - 7	3721	0.073	6097	97.64	16.68	80.96
7 - 8	2687	0.052	4403	28.61	7.99	20.62
8 - 9	2326	0.045	3811	18.64	6.18	12.46
9 - 10	1980	0.039	3244	12.36	4.83	7.53
10 - 11	1406	0.027	2304	6.25	3.21	3.05
11pm - 12mid	916	0.018	1501	3.49	2.26	1.23
TOTALS:	51306	1.000	84071 ADT	923.73	188.86	734.86

Annual Delay Savings 183,716 (veh-hr)

Annual User-Time Benefit $609,936 (@ $3.32 per veh-hr)

Table A-30. Fuel Consumption Estimates for William Cannon & Congress (Year 20)

Time of Day	Volume (veh-hr/hr)	Stopped Delay (sec/veh) At-Grade	Stopped Delay (sec/veh) Grade Separation	Total Fuel consumed (gallons) At-Grade	Total Fuel consumed (gallons) Grade Separation	Fuel Savings
12mid - 1am	954	6.83	8.63	21.52	13.33	8.19
1 - 2	572	8.63	12.19	13.32	8.42	4.90
2 - 3	482	9.60	13.91	11.39	7.25	4.14
3 - 4	321	12.82	19.46	7.94	5.14	2.80
4 - 5	390	11.09	16.51	9.42	6.05	3.38
5 - 6	964	6.80	8.58	21.73	13.45	8.28
6 - 7	3341	10.99	6.95	80.62	45.34	35.27
7 - 8	6551	57.34	14.31	233.45	99.02	134.43
8 - 9	4709	21.00	8.94	127.57	66.13	61.43
9 - 10	3616	12.39	7.24	88.93	49.34	39.59
10 - 11	3608	12.34	7.23	88.68	49.22	39.46
11 - 12noon	4234	16.56	8.09	109.47	58.64	50.84
12 - 1pm	4750	21.45	9.02	129.24	66.80	62.44
1 - 2	4642	20.30	8.81	124.86	65.05	59.81
2 - 3	4621	20.08	8.77	124.01	64.71	59.30
3 - 4	5242	27.75	10.13	151.10	74.96	76.14
4 - 5	6554	57.45	14.33	233.72	99.09	134.63
5 - 6	7157	81.43	17.05	291.95	111.64	180.31
6 - 7	6097	44.34	12.63	199.65	90.24	109.41
7 - 8	4403	17.99	8.37	115.64	61.26	54.37
8 - 9	3811	13.54	7.48	95.11	52.22	42.89
9 - 10	3244	10.55	6.87	77.80	43.96	33.84
10 - 11	2304	7.51	6.42	52.65	30.95	21.70
11pm - 12mid	1501	6.45	6.95	33.62	20.37	13.25
TOTALS:	84071			2443	1203	1241

Annual Fuel Savings 310,205 (gallons)

Annual Fuel Benefit $356,736 (@ $1.15 per gallon)

Table A-31. Fuel Consumption Data for William Cannon & Congress (AGI) (Year 20)

	At-Grade Intersection (AGI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	2.47	0.25	1.02	17.79	21.52	12mid - 1am
	1.70	0.18	0.77	10.67	13.32	1 - 2
	1.51	0.17	0.72	8.98	11.39	2 - 3
	1.16	0.15	0.64	5.99	7.94	3 - 4
	1.32	0.16	0.68	7.27	9.42	4 - 5
	2.49	0.25	1.03	17.97	21.73	5 - 6
	11.23	1.33	5.74	62.31	80.62	6 - 7
	39.60	12.92	58.75	122.18	233.45	7 - 8
	20.79	3.48	15.47	87.83	127.57	8 - 9
	12.86	1.61	7.01	67.45	88.93	9 - 10
	12.81	1.60	6.97	67.29	88.68	10 - 11
	17.05	2.49	10.96	78.97	109.47	11 - 12noon
	21.13	3.58	15.94	88.59	129.24	12 - 1pm
	20.23	3.32	14.73	86.58	124.86	1 - 2
	20.06	3.27	14.51	86.18	124.01	2 - 3
	25.51	5.07	22.75	97.76	151.10	3 - 4
	39.64	12.95	58.89	122.24	233.72	4 - 5
	47.34	19.98	91.15	133.49	291.95	5 - 6
	34.31	9.34	42.28	113.71	199.65	6 - 7
	18.33	2.80	12.39	82.12	115.64	7 - 8
	14.11	1.85	8.07	71.08	95.11	8 - 9
	10.69	1.24	5.35	60.51	77.80	9 - 10
	6.32	0.65	2.71	42.97	52.65	10 - 11
	3.75	0.37	1.51	27.99	33.62	11pm - 12mid
TOTALS:	386	89	400	1568	2443	

Table A-32. Fuel Consumption Data for William Cannon & Congress (GSI) (Year 20)

	Grade Separated Interchange (GSI) Fuel Consumption, (gallons)					
	Stopped	Speed Changes	Idling	Constant Speed	Total	Time of Day
Fuel Consumption Rates:	6.800	3.790	0.563	37.300		
	1.70	0.18	0.77	10.67	13.33	12mid - 1am
	1.21	0.15	0.65	6.40	8.42	1 - 2
	1.08	0.14	0.63	5.39	7.25	2 - 3
	0.83	0.13	0.59	3.59	5.14	3 - 4
	0.94	0.14	0.60	4.36	6.05	4 - 5
	1.71	0.18	0.78	10.78	13.45	5 - 6
	5.25	0.52	2.18	37.39	45.34	6 - 7
	14.90	2.01	8.80	73.31	99.02	7 - 8
	8.55	0.93	3.95	52.70	66.13	8 - 9
	5.83	0.59	2.46	40.47	49.34	9 - 10
	5.81	0.59	2.45	40.38	49.22	10 - 11
	7.28	0.76	3.21	47.38	58.64	11 - 12noon
	8.67	0.95	4.02	53.16	66.80	12 - 1pm
	8.37	0.90	3.84	51.95	65.05	1 - 2
	8.31	0.90	3.80	51.71	64.71	2 - 3
	10.16	1.16	4.98	58.66	74.96	3 - 4
	14.92	2.01	8.81	73.34	99.09	4 - 5
	17.50	2.60	11.45	80.09	111.64	5 - 6
	13.13	1.66	7.22	68.23	90.24	6 - 7
	7.72	0.82	3.46	49.27	61.26	7 - 8
	6.26	0.64	2.67	42.65	52.22	8 - 9
	5.06	0.50	2.09	36.31	43.96	9 - 10
	3.44	0.34	1.39	25.78	30.95	10 - 11
	2.36	0.24	0.98	16.80	20.37	11pm - 12mid
TOTALS:	161	19	82	941	1203	

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