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16. Abstract <p>This research report documents guidelines for the design of urban arterial interchanges in densely developed areas. It address the geometric issues, operational issues, benefits, and costs of three interchange configurations. The configurations investigated are the Tight Urban Diamond Interchange (TUDI), the Single Point Urban Interchange (SPUI), and the Left-Hand Exit Single Signal (LHESS).</p> <p>Despite the geometric differences between the TUDI and SPUI, they both require virtually the same amount of right-of-way. This contradicts the popular belief that the SPUI configuration minimizes right-of-way requirements. The LHESS configuration can be constructed within a narrower right-of-way due to the reduced control area.</p> <p>The SPUIs operational characteristics were found to be dependent on the relative proportion of simultaneous left turning volumes. By contrast, the TUDI was found to be efficient under a variety of demand volumes. The LHESS operated similar to an at-grade intersection.</p> <p>In general, the TUDI was found to be the best design alternative under urban conditions. It offers the greatest flexibility in operation and future expansion at a lower cost than the SPUI. The LHESS design is not recommended due to its violation of driver expectancy.</p> <p>A case study of two intersections in the state of Texas was performed to illustrate the application of the material presented in this report.</p>					
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**GUIDELINES FOR THE DESIGN OF URBAN ARTERIAL INTERCHANGES  
IN DENSELY DEVELOPED AREAS**

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Research Report 1237-2F

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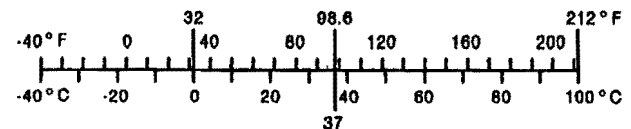
## METRIC (SI\*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS					APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>					<b>LENGTH</b>				
in	inches	2.54	centimeters	cm	mm	millimeters	0.039	inches	in
ft	feet	0.3048	meters	m	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	yd	meters	1.09	yards	yd
mi	miles	1.61	kilometers	km	km	kilometers	0.621	miles	mi
<b>AREA</b>					<b>AREA</b>				
in <sup>2</sup>	square inches	6.452	centimeters squared	cm <sup>2</sup>	mm <sup>2</sup>	millimeters squared	0.0016	square inches	in <sup>2</sup>
ft <sup>2</sup>	square feet	0.0929	meters squared	m <sup>2</sup>	m <sup>2</sup>	meters squared	10.764	square feet	ft <sup>2</sup>
yd <sup>2</sup>	square yards	0.836	meters squared	m <sup>2</sup>	yd <sup>2</sup>	kilometers squared	0.39	square miles	mi <sup>2</sup>
mi <sup>2</sup>	square miles	2.59	kilometers squared	km <sup>2</sup>	ha	hectares (10,000 m <sup>2</sup> )	2.53	acres	ac
ac	acres	0.395	hectares	ha					
<b>MASS (weight)</b>					<b>MASS (weight)</b>				
oz	ounces	28.35	grams	g	g	grams	0.0353	ounces	oz
lb	pounds	0.454	kilograms	kg	kg	kilograms	2.205	pounds	lb
T	short tons (2000 lb)	0.907	megagrams	Mg	Mg	megagrams (1000 kg)	1.103	short tons	T
<b>VOLUME</b>					<b>VOLUME</b>				
fl oz	fluid ounces	29.57	milliliters	mL	mL	milliliters	0.034	fluid ounces	fl oz
gal	gallons	3.785	liters	L	L	liters	0.264	gallons	gal
ft <sup>3</sup>	cubic feet	0.0328	meters cubed	m <sup>3</sup>	m <sup>3</sup>	meters cubed	35.315	cubic feet	ft <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.765	meters cubed	m <sup>3</sup>	m <sup>3</sup>	meters cubed	1.308	cubic yards	yd <sup>3</sup>
<b>TEMPERATURE (exact)</b>					<b>TEMPERATURE (exact)</b>				
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C	°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

Note: Volumes greater than 1000 L shall be shown in m<sup>3</sup>.

These factors conform to the requirement of FHWA Order 5190.1A

\*SI is the symbol for the International System of Measurements





## ABSTRACT

This research report documents guidelines for the design of urban arterial interchanges in densely developed areas. It addresses the geometric issues, operational issues, benefits, and costs of three interchange configurations. The configurations investigated are the Tight Urban Diamond Interchange (TUDI), the Single Point Urban Interchange (SPUI), and the Left-Hand Exit Single Signal (LHESS).

Despite the geometric differences between the TUDI and SPUI, they both require virtually the same amount of right-of-way. This contradicts the popular belief that the SPUI configuration minimizes right-of-way requirements. The LHESS configuration can be constructed within a narrower right-of-way due to the reduced control area.

The SPUIs operational characteristics were found to be dependent on the relative proportion of simultaneous left turning volumes. By contrast, the TUDI was found to be efficient under a variety of demand volumes. The LHESS operated similar to an at-grade intersection.

In general, the TUDI was found to be the best design alternative under urban conditions. It offers the greatest flexibility in operation and future expansion at a lower cost than the SPUI. The LHESS design is not recommended due to its violation of driver expectancy and relatively poor operation under high demand volumes.

A case study of two intersections in the state of Texas was performed to illustrate the application of the material presented in this report.





## **DISCLAIMER**

The content of this report reflects the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. This report is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was Dr. Vergil G. Stover, P.E. (26979).

## **IMPLEMENTATION**

This study was sponsored by the Texas Department of Transportation with the major objectives of documenting guidelines for the design of urban arterial interchanges in densely developed areas. The results of this research will aid engineers in addressing relevant design issues prior to the design of an interchange in a densely developed area which will lead to improved operations and safety for motorists on urban arterials.



## SUMMARY

The capacity of an urban arterial street is largely dependent on the number of high volume intersections over its length. Since these intersections are common in the urban setting, the mobility along an arterial street is significantly impeded. Grade separation offers significant improvement to the overall flow through a high volume arterial intersection. This research report documents guidelines for the design of urban arterial interchanges in densely developed areas. Special emphasis is placed on the geometric issues, operational issues, benefits, and costs of the Tight Urban Diamond Interchange (TUDI), Single Point Diamond Interchange (SPUI), and the Left-Hand Exit Single Signal (LHESS) configurations.

Right-of-way in densely developed urban areas is a valuable commodity. Consequently, each design was evaluated on its right-of-way requirements. Despite the claims that SPUIs minimize the required right-of-way, the SPUI offered no evident advantage over the TUDI. Anywhere a SPUI can be constructed, a TUDI can be constructed. The LHESS configuration, however, minimizes the amount of right-of-way required by reducing the overall control area.

An investigation into the length necessary to adequately provide overpass and underpass grade separation was conducted. The overpass SPUI configuration required 10 to 15 percent longer vertical curves than comparable overpass TUDIs. The disparity stems from the longer spans required for the SPUI design. Virtually no difference was found between the underpass designs. The required length to effect grade separation for the LHESS will approximate the TUDI if median bents are incorporated into the design or the SPUI if no bents are provided.

Operational analysis was performed on each design using TRANSYT 7F to determine which configuration minimized delay under various demand volumes. The efficient operation of the SPUI was found to be dependent on the ratio of simultaneous left turn volumes. The LHESS operated as a typical at-grade intersection. It provided large delays under high volumes. By contrast, the TUDI operated efficiently under a variety of volumes.

The direct costs associated with the each configuration were examined. The TUDI costs range from \$2.4 million to \$6.4 million. The SPUI costs were found to be from \$1 to \$4 million more than the TUDI. The cost of the LHESS is estimated to approximate the cost of a flyover. A range of \$2.2 million to \$5.7 million was calculated for the LHESS using available flyover data.

The results of this study indicate that, in general, the TUDI is the most appropriate configuration for urban arterial interchanges. It affords more efficient operation under a variety of demand volumes than the other designs. Moreover, the TUDI meets the driver's expectancy upon approaching and traveling through the interchange.

A simple benefit/cost analysis was performed on six study sites in the state of Texas. The four sites that appear to warrant grade separation are Site 1 -- FM 1960 and Kuykendahl (Houston), Site 2 -- Dairy Ashford and Westheimer (Houston), Site 3 -- Preston and Arapaho (Dallas), and Site 5 -- Preston and Beltline (Dallas). Site 4 -- FM 1960 and SH249 (Houston) was a marginal case and further study is warranted. Grade separation is not justifiable at Site 6 -- FM 731 and Altamesa. A case study was performed on Site 3 and 4, demonstrating the application of the material presented in this report.

Grade separation was justified at Site 4 using the methodology set out by Stover and Raza. They determined the cut-off flow rate for which grade separation is a viable option to be 5000 vph.

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## CHAPTER I INTRODUCTION

Congestion, without a doubt, has been the most significant transportation issue in our nations largest urban areas over the past decade. Nationally the figures are staggering. The Federal Highway Administration (FHWA), according to one study, anticipates increases in delay experienced on urban freeways of 360 percent over the next twenty years. However, this increase is not limited to freeway facilities. Non-freeway urban delay is expected to increase by approximately 200 percent (1).

The total annual cost of congestion in 1988 exceeded \$34 billion in 39 of the largest cities in the United States. Houston, Dallas, Austin, Fort Worth, San Antonio, El Paso, and Corpus Christi combined experienced a total congestion cost of \$3.3 billion dollars in 1988 (2). These costs are primarily attributed to the delays incurred by the motorists, increased fuel consumption, increased vehicle emissions, and additional maintenance costs associated with congestion.

The problem will continue to manifest itself due to an increasing number of people traveling by automobile to and from locations dispersed throughout the urban area. This view is supported by data from the National Personal Transportation Study (NPTS). The 1983 NPTS study shows that adult licensed drivers averaged approximately 30 miles of local personal travel per day, while those without a driver's license averaged approximately 10 miles of personal travel per day. Reno indicates that an average annual growth rate of 1.3 to 1.7 percent in personal vehicular travel should be anticipated between now and 2020. Furthermore, according to NPTS data, the percentage of households from 1969 to 1983 with no vehicles has steadily declined, while those with three or more vehicles has steadily increased (see Table 1) (3).

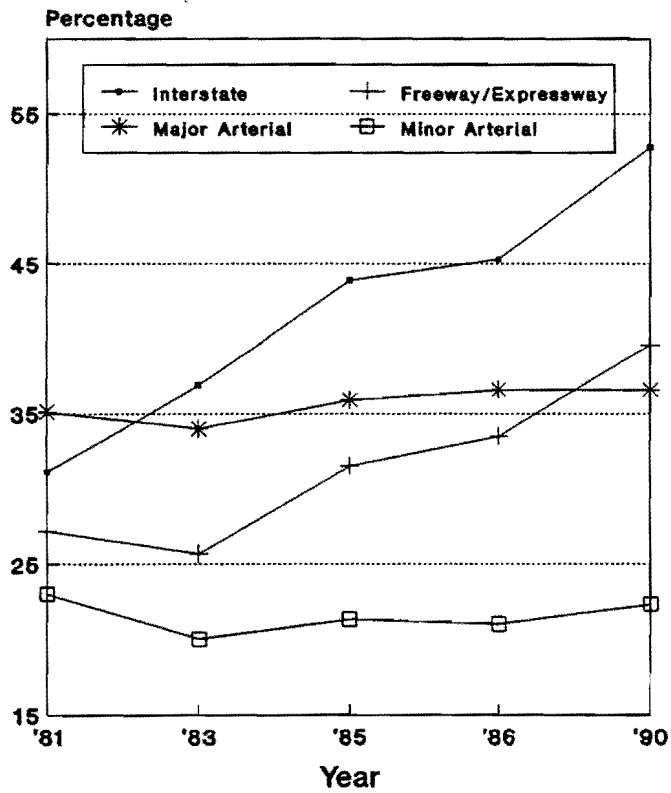
The FHWA Highway Statistics provide further evidence of the increasing pressures placed on the urban transportation system (4, 5, 6, 7). As shown in Figure 1, the percentage of urban interstates, freeways, and expressways with high volume to capacity ratios

**TABLE 1. Distribution of Households by Vehicle Ownership**

Number of Vehicles	Percentage of All Households		
	1969 <sup>1</sup>	1977	1983
None	20.6	15.3	13.5
One	48.4	34.7	33.7
Two	26.4	34.4	33.5
Three or more	4.6	15.6	19.3

<sup>1</sup> 1969 vehicle counts included automobiles and passenger vans only.  
 SOURCE: Reno, Personal Mobility in the United States

**FIGURE 1. Percentage of Road Miles at High<sup>1</sup> Volume/Capacity Ratios, 1981-1990**



<sup>1</sup> V/C Ratio > 0.70  
 SOURCE: Highway Statistics. FHWA, USDOT, 1981, 1983, 1985, 1986, 1990

has steadily increased through the 80s. Since this trend is expected to continue, the major arterial system in the urban area must be capable of providing mobility to the increasing vehicular demand being diverted from the congested urban freeways. Christiansen and Ward recognized the need for the arterial street network to play an increasing role in carrying daily vehicle travel, especially in Texas where arterial streets are not continuous (8).

Unfortunately, the capacity of an urban arterial street is largely dependent on the number of high volume intersections over its length. In view of the fact that these intersections are common in the urban setting, the mobility along the arterial street can be significantly impeded. In many instances, the disturbance in traffic flow caused by each high volume intersection not only drastically reduces the capacity of the arterial, but can affect the flow substantial distances from the intersection. Obviously, this can result in under utilization of a substantial portion of the arterial street.

Considering the problem presented, it is logical to ask what should be done. A strategic arterial network that is capable of carrying a sizable portion of the vehicle miles of travel (VMT) diverted from the congested freeway system is possible. This type of network would require "super arterials" that are designed to high geometric standards. Such arterials, according to Christiansen and Ward (8), would have limited access, be continuous for at least four miles, and utilize grade separations at the critical intersections. Unfortunately, in densely developed urban areas, grade separations are often difficult and expensive to implement due to limited right-of-way and the unlimited access typically afforded development at the intersection. As a result, there exists a need to develop and examine design guidelines for grade separations along arterial streets located in densely developed areas.

There are many strategies available to alleviate congestion and improve the overall flow through high volume arterial intersections. Some of the more common strategies implemented are: coordination of traffic signals along the arterial; improving signal timing at high volume intersections; widening the arterial to provide more lanes; providing dual left turn bays; providing exclusive right turn bays; turn prohibitions; and access restrictions (9). However,

these alternatives have been exhausted at many of the urban arterial intersections due to practical and financial limitations. Moreover, these strategies are ultimately limited by the time sharing of the intersection area. Typical high volume urban intersections only afford the through movements a maximum of 35 to 40 percent of the available green time during peak hours (10). Under these circumstances, queues will be unable to clear during each cycle, creating a queue spill over into the next cycle. As the increasing queues form, the delay incurred at the intersection grows exponentially until the demand is less than the capacity. Grade separation, in most instances, represents an effective method for reducing the delay and increasing the capacity of such high volume arterial intersections.

The primary purpose of this research project is to develop geometric guidelines and criteria for the replacement of congested urban arterial at-grade intersections with interchanges. The focus of this report is to present various grade separated configurations applicable to densely developed urban areas. Special attention will be given to the design elements, geometrics, operation, and costs and benefits of each configuration.



## CHAPTER II

### LITERATURE REVIEW

During the 1950s and 60s our nation embarked on the monumental task of providing a national system of interstate and defense highways. This system was supplemented by a secondary freeway system of urban and rural facilities. By the late 60s, the roadway building programs were in full swing and these facilities began to carry the majority of daily traffic. However, the environmental movement of the 70s brought an abrupt end to the majority of the freeway building programs. This, in conjunction with the increasing congestion on such facilities, made it apparent that the arterial system would need to carry a larger share of the ever-growing traffic demand.

Typical urban arterial corridors are interspersed with high volume signalized intersections which limit the overall capacity. These intersections, unable to provide the capacity necessary to maintain safe and efficient traffic movement, produce bottlenecks, long traffic queues, and generally retard the flow along the arterial. In order to maintain the integrity of the arterial, many jurisdictions are examining the possibility of creating a "super arterial." The concept, originally developed by James Brown in the 70s, is to provide a continuous flow along the arterial for a substantial distance, usually four miles or more. In terms of operation, the super arterial lies somewhere between a freeway and an arterial street (11). The increase in capacity along the arterial street is achieved by whatever means are practically available, from transportation system management (TSM) to grade separation. To the degree possible, major intersecting streets are grade separated in order to reduce the number of traffic signals.

Although the 1973 American Association of State Highways and Transportation Officials (AASHTO) *A Policy on Design of Urban Highways and Arterial Streets* proposes the use of grade separation to solve arterial congestion at critical intersections (12), it has traditionally been used exclusively in freeway design. Nevertheless, in recent years, grade separation has become a feasible alternative for reducing the congestion along arterial streets. Moreover, it represents an essential component of the "super arterial" concept. Some of the benefits grade separation



affords are: reduced delay incurred by the motorists, reduced number of conflict points at the intersection, increased capacity of the intersection, increased safety, and decreased amount of vehicular emissions.

Numerous studies have been conducted which investigate the benefits and costs of grade separating a congested arterial intersection. Some of the noteworthy research will be discussed in the following paragraphs.

### BENEFITS

In 1981 Stanley Byington (13) reported on the European experience with flyovers as a temporary measure to alleviate congestion at an isolated intersections. He addressed several issues including: (1) circumstances under which a flyover is feasible, (2) comparison of other traffic control improvements in terms of immediate rates of return, and (3) safety, aesthetic, and environmental issues. Despite the fact that other traffic control improvements may be more cost effective (see Table 2), the report clearly indicates that flyovers improve the overall performance at the intersection and thus provide a cost effective solution also.

**TABLE 2. Construction Costs and Immediate Rates of Return for Selected Traffic Control Improvements**

Traffic Control Improvement	Construction Cost in 1990 Dollars	Range of Immediate Rates of Return
Rural Flyover	\$64,000 - \$1,360,000	50 - 60%
Urban Flyover	\$1,120,000 - \$4,480,000	20 - 120%
Signal Timing Optimization	\$480 - \$640	10,800 - 14,000%
Signal Coordination	\$3,200 - \$16,000	900 - 4,500%

SOURCE: Byington, 1981.

JEF Engineering (11), in a study prepared for the Orange County Transportation Commission, reported that the installation of grade separation not only reduced vehicle delay at the improved intersection, but also for the intersections immediately downstream. The downstream intersection delay reduction is attributed to the dispersal of the platooned vehicles that form as a result of traffic signal control. Although the magnitude of the reduction is dependent on several factors, substantial reductions are not uncommon.

In addition, JEF Engineering, using the TRANSYT model, tested several high flow arterial strategies in order to evaluate the most effective scenario. They found that installation of a single grade separated structure in conjunction with coordinating the signalization along an arterial corridor resulted in reductions of 50 percent in delay, 16 percent in fuel consumption, and 29 percent in vehicle emissions. This scenario also afforded an increase of 43 percent in the average vehicular speed along the arterial (11).

A study was conducted by Parsons, Brinkerhoff, Quade, and Douglas which investigated the feasibility of a "super arterial" for Orange County, California. In the study grade separation was estimated to have an annual time savings of \$23 million and an annual gasoline savings of 1.6 million gallons. The report indicated that the total delay cost incurred during construction would be "repaid" in three to four years. Other benefits cited by the study are (14):

- Generation of a significant economic benefit to surrounding businesses and property owners due to the accommodation of increased traffic volumes.
- Positive effects on the surrounding property values due to increased capacity, ease in getting on site, less congestion, and better traffic control.
- Property becomes more attractive to the development of commercial buildings as well as retail and office space. The increased development activity would benefit many groups (i.e., property owners, developers, increase tax base, employment opportunities, etc.).

Van Dell and Associates concluded based on the results obtained from their divergent case study that grade separation (overpass or underpass) provided not only significant increases in the capacity for the through movement, but also increased the level-of-service of the at-grade

intersection (15).

## COSTS

The costs associated with grade separation can be broken down into two types: direct and indirect. Direct costs are those specifically related to the project, while indirect costs are "byproducts." The direct costs include the cost of the structure, at-grade roadway improvements, traffic control devices (i.e., signals, signs, markings, and illumination), utilities, and traffic handling. Indirect costs consist of the increases in travel time, fuel consumption, and vehicular emissions associated with the construction of the project. Any loss of business associated with reduced access to the surrounding properties is also an indirect cost. These costs are primarily the result of the construction delays and the diversion of motorists to alternate routes (18, 19).

Bonilla and Urbanik, in a 1987 Texas Transportation Institute (TTI) study, compared the costs, both direct and indirect, associated with conventional cast-in-place construction and prefabricated construction (18). Their case study consisted of seven intersections. They reported that the direct costs of the conventional construction grade separations ranged from a low of \$2.3 million to a high of \$7.0 million, while the corresponding prefabricated construction ranged from a low of \$4.6 million and a high of \$16.5 million. Although the prefabricated structures generally cost nearly twice that of the conventional structure, many feel that the higher capital costs are offset by the substantial decreases in the indirect costs thus making the prefabricated structure less costly overall. That is, the cast-in-place structure takes 18 to 24 months to complete, whereas the prefabricated structure can be operational in a matter of months, resulting in fewer delays, less fuel consumption, and lower emissions. Bonilla and Urbanik, however, concluded that the savings in indirect costs afforded the prefabricated flyover typically do not result in lower overall costs (18).

## IMPACTS ON ACCESS

Grade separation, by its very nature, requires restricting the access provided to surrounding development. Geometrically speaking, a physical barrier prohibiting left turns to and from the major arterial is created for a distance of 800 to 900 feet on either side of the signalized intersection (15). As a result, property access will be significantly limited, which may have an adverse affect on the local businesses. This is particularly true at congested intersections where dense development has occurred.

The Super Street Demonstration Project by Parsons, Brinckerhoff, Quade, and Douglas indicated that the economic impact on local businesses would not be negative when compared to the do-nothing alternative (14). Put simply, increasing congestion at the at-grade intersection would result in a substantially greater negative economic impact than grade separation. Moreover, the grade separation might actually stimulate the economic activity by increasing the carrying capacity of the roadway, lowering congestion levels in the immediate area, and providing better traffic control through the intersection. In turn, property values and development activity would increase.

By contrast, a 1966 study by Charles Walker on three flyovers built in Chicago indicated that the impact on the surrounding value of land depended on its use (20). He reported that the commercial areas surrounding the flyovers experienced a lower increase in property values than similar property located at nearby at-grade intersections. In fact, Walker shows two commercial properties actually experienced declines of 30 and 33 percent. However, industrial properties surrounding the flyovers experienced no adverse affects. The increase in property value for industrial land uses at the flyovers and at-grade intersections were virtually the same.

Although these studies seem to contradict each other, the economic impact (either positive or negative) of reducing the number of access points will generally depend on several factors. They are: (1) current levels of traffic using the at-grade intersection and access points, (2) number of access points within the functional influence of the flyover, (3) availability of

alternative access points, and (4) existing left turn restrictions via signs or medians (18).

## TYPES OF ARTERIAL INTERCHANGES

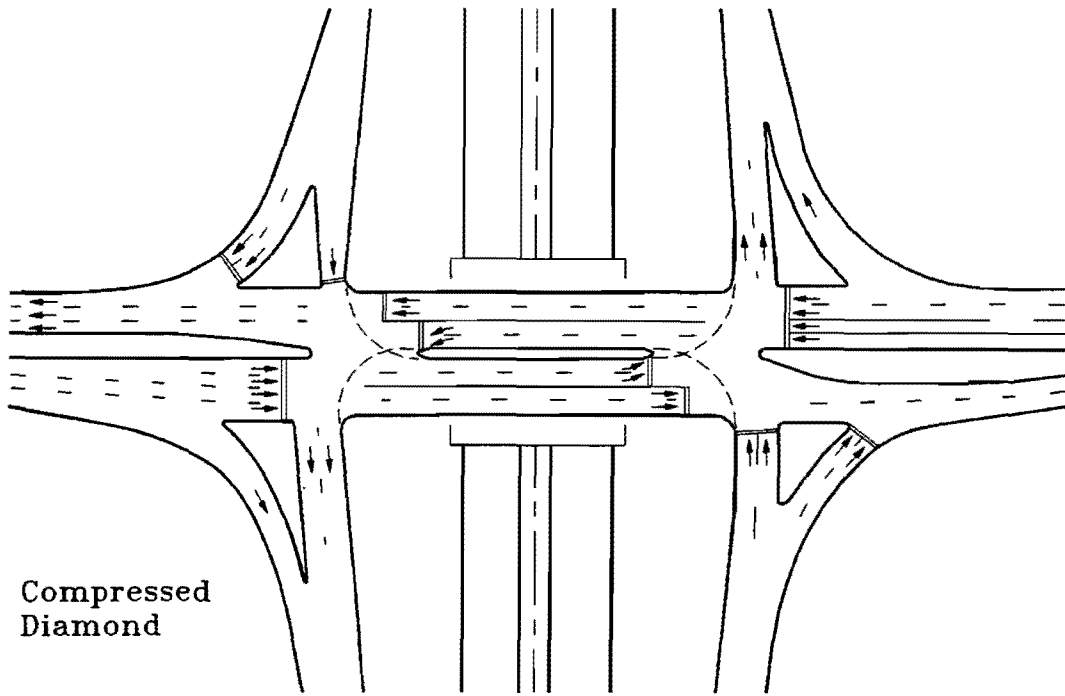
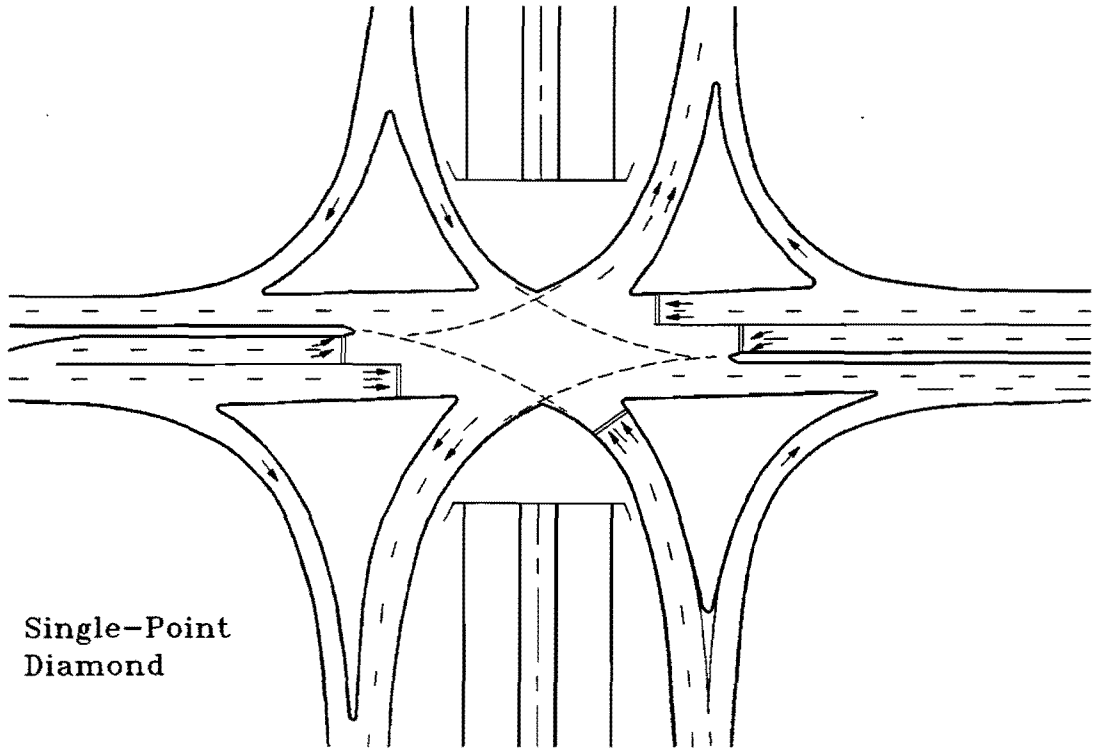
Arterial interchanges have the ability to accommodate higher volumes of traffic safely and efficiently through bottleneck intersections along the urban arterial corridor. Selection of the type of interchange is governed primarily by two factors in the urban setting - traffic demand and the availability of right-of-way (21). Since urban areas are typically densely developed, availability of right-of-way generally dictates.

Although many interchange types exist, diamond interchanges are usually the most desirable in locations where right-of-way is restricted. A variety of diamond interchanges exist such as conventional diamond, compressed/tight diamond, split diamond, single-point diamond, three level diamond, and three-level stacked diamond (22). Of these, only the tight diamond, also known as the tight urban diamond interchange (TUDI), and the single-point diamond, also known as the single-point urban interchange (SPUI), are practical in urban areas. (See Figure 2)

Since only a handful of SPUIs are in operation across the United States, recent research efforts have been focused on the geometric and operational characteristics of the SPUI (23, 24, 25, 26). Their principal advantage is that all movements can be accommodated at a single signalized intersection thus providing more capacity. The SPUI can offer 40 percent to 100 percent more capacity than an at-grade intersection and 10 percent to 50 percent more capacity than a TUDI, depending on the proportions of through and turning traffic (25).

The benefits associated with higher capacity of the SPUI are not achieved without some cost. Ben Martin notes that anywhere a SPUI can be constructed, a TUDI can be constructed less expensively (26). In the case of overpass SPUIs (the mainlanes elevated), the savings

**FIGURE 2. Common Types of Arterial Interchanges**

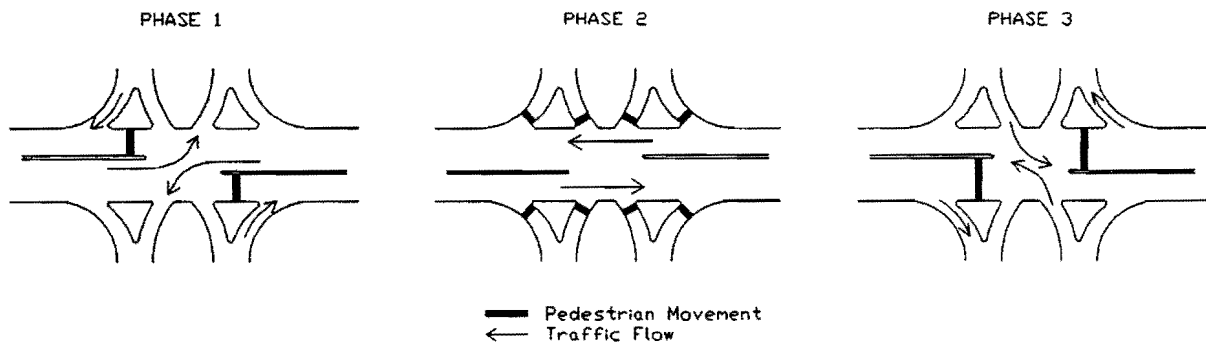


can be 150 percent or more. This is primarily due to the increased bridge length and depth. The depressed SPUI is typically more expensive as well due to design complexities. The complexities are a result of the left turn movements, which require wider sloping abutments and narrow central piers, much like an hourglass.

Pedestrian safety is another potential problem associated with SPUIs (24, 25, 26). Figure 2 shows the pedestrian phasing for a three phase SPUI. In order to cross the cross street, a full cycle is required. First the pedestrian must cross to the median during the cross street left turn phase. At this point, they must remain in the median during the cross street through phase while traffic is moving on both sides. The median, as a result, should be wide enough to safely store the pedestrians. The pedestrian completes the crossing movement during the off-ramp left turn phase. The pedestrian crossing movement parallel to the cross street is also shown in Figure 3.

To simplify the pedestrian movement and reduce conflicts, a pedestrian phase can be incorporated into the SPUI. However, the addition of another phase will reduce the overall capacity of the SPUI by some 30 percent thereby nullifying the primary benefit of the SPUI configuration (26). This is primarily due to the low ratio of pedestrian demand to the phase time required to service this demand.

**FIGURE 3. Pedestrian Movements at Three Phase SPUIs**



Additional problems with SPUIs are associated with the reduced amount of access to the surrounding developments (25). Vehicular traffic which exits the mainline cannot directly access development on the far side of the cross street. To alleviate this problem, a through movement must be provided. Providing such a movement requires an additional phase in the cycle which, similar to the pedestrian phase, will reduce the capacity afforded the remaining movements.

Poppe, Radwan, and Matthias examined the operational characteristics of three SPUIs in the Phoenix metropolitan area (23). Their primary objective was to determine the saturation flow rate of various movements at the SPUI. They concluded that saturation flow rates of 2,000 pcphgpl for the left turning movements could be achieved at SPUIs. They also suggest that left turn movements with radii larger than 300 feet may be able to achieve even higher saturation flow rates.

By far the most comprehensive study of SPUIs was conducted by Messer, Bonneson, Anderson, and McFarland in National Cooperative Highway Research Program (NCHRP) Report 3-40 (24). The report examines the historical development, geometric characteristics, traffic operations, design guidelines, and cost effective analysis for SPUIs. Their research indicates that higher saturation flow rates are achieved at SPUIs due to the large left turn radii which promote higher speeds and reduce the off-tracking of large vehicles. This supports the research of Poppe et al.

Some questions still remain about the effectiveness of a SPUI over other designs, due primarily to the large amount of uncontrolled pavement area in the center of the interchange. This area creates the need for additional driver guidance and controllability in the design. Moreover, the area creates substantially higher clearance intervals, which reduces the overall capacity of the intersection.

A need currently exists to provide more efficient high volume arterial intersections in our densely developed cities. These facilities must be constructed on existing alignments and, more-or-less, within existing right-of-way. Decision-makers are often confused over the various



advantages and disadvantages of designs. It is therefore the purpose of this paper to discuss the geometric issues, operational issues, and benefits and costs associated with the SPUI, TUDI, and basic flyover designs.



## CHAPTER III DESIGN ELEMENTS

This chapter will review and consolidate the design controls and criteria that affect an urban grade separated interchange. Special attention will be given to those issues that are peculiar and unique to the SPUI, TUDI, and Left-Hand Exit Single Signal (LHESS) configurations. The intent is to expose the design issues that need to be addressed for a typical high volume grade separated arterial interchange. Some intersections may have unique characteristics and site constraints which must be taken into consideration.

### DESIGN VEHICLE

Like pedestrians, design vehicles are easily overlooked in the design of an interchange. Since the 1982 Surface Transportation Assistant Act limited the restrictions the state can place on the size of tractor trailers, the physical characteristics of the trucks using our roadways have changed considerably (27). Moreover, there is constant political pressure from the trucking industry to allow larger and heavier tractor trailers on our roads. This is evidenced by the additional classes of design vehicles provided in the 1990 AASHTO *A Policy on Geometric Design of Highways and Streets* (hereafter referred to as the Green Book) over the 1984 edition.

Design vehicles are selected vehicle dimensions and operating characteristics that represent a large cross-section of vehicles that actually use the roadway. Design features such as lane width, rate of curvature, turning radius, clearance, and sight distance are directly related to the design vehicles' dimensions and operating characteristics.

Although most urban intersections are well suited for passenger cars, single unit trucks, and perhaps buses, few are designed with turning radii that can accommodate large combination tractor trailers. Small curb radii, narrow lane widths, and narrow overall street widths all contribute to the increased operational problems with at-grade intersections.

The large turning radii associated with the SPUI offers an advantage in terms of operation over other designs. Left turn radii at a SPUI generally range from 200 to 300 feet, while right turn radii range from 70 to 130 feet (28). By contrast, the left turning radii at TUDIs and at-grade intersections typically range from 50 to 75 feet while the right turn ranges from 25 to 50 feet. As a result, if large trucks represent a significant portion of the traffic stream, then the SPUI may prove the most operationally efficient alternative.

Special attention must be paid to the design vehicle if serious consideration is being given to a prefabricated structure similar to those used in Europe. European designs are focused on providing service to the glut of cars using the intersection (17). That is, only passenger cars are allowed to use the flyover while trucks, buses, recreational vehicles, and turning passenger cars are using the at-grade portion of the flyover. Therefore, the structures are designed for lighter loads, maximum grades, and minimum clearances.

## SIGHT DISTANCE

Sight distance is, without a doubt, one of the most important considerations that needs to be addressed in the design of roadways. Obviously, the ability to see ahead is of utmost importance in terms of efficient and safe traffic flow. Drivers must be able to see far enough ahead to detect and avoid unexpected and hazardous situations.

Of the five primary sight distances discussed in the 1990 AASHTO Green Book (stopping, decision, passing, intersection, and railroad grade crossing), only three apply to the design of an interchange. They are stopping sight distance, decision sight distance, and intersection sight distance. Each is briefly examined as they pertain to arterial interchanges in the following sections.

### *Stopping Sight Distance*

Stopping sight distance (SSD) is defined as the length of roadway ahead required to

enable a vehicle traveling at or below the design speed of the roadway to stop before reaching a stationary object in its path. SSD should be provided at every point along a roadway (29).

SSD is a function of speed, driver reaction time, coefficient of friction, and grade of the roadway. It is the algebraic sum of the distance traveled during perception-reaction time and the distance traveled during braking (29). The 1990 Green Book provides design stopping sight distances for various design speeds in Table III-A. Corrections for grade are provided in Table III-2 of the Green Book.

In terms of arterial interchange design, SSD requirements primarily controls the minimum lengths of vertical curves and the minimum radii for horizontal curves. The vertical curve cannot be so sharp that it limits the distance a driver can see ahead. Similarly, the horizontal curve cannot be so severe that it limits the ability to see objects on the roadway ahead.

#### *Decision Sight Distance*

SSD is typically adequate for ordinary conditions, where the situation requires only perception-reaction time. However, in arterial interchange design, the situation may be unexpected or may require a complex or unusual maneuver. In this environment, the work load placed on the driver is increased and additional distance must be provided to account for the increased time required for the driver to process and initiate an appropriate maneuver. The distance associated with this increased perception-reaction time is commonly referred to as decision sight distance (29).

Decision sight distance is recommended at locations of lane drops, left hand exits, toll plazas, and areas where traffic, advertisements, or traffic control devices compete for the drivers attention. AASHTO 1990 Green Book values for various maneuvers are shown in Table 3. For grade separated entrance and exit ramps, avoidance maneuver D or E is recommended.

**TABLE 3. AASHTO Decision Sight Distance**

Design Speed (MPH)	Decision Sight Distance for Avoidance Maneuver (ft)				
	A <sup>1</sup>	B <sup>2</sup>	C <sup>3</sup>	D <sup>4</sup>	E <sup>5</sup>
30	220	500	450	500	625
40	345	725	600	725	825
50	500	975	750	900	1025
60	680	1300	1000	1150	1275
70	900	1525	1100	1300	1450

<sup>1</sup>Stop on rural road.

<sup>2</sup>Stop on urban road.

<sup>3</sup>Speed/path/direction change on rural road.

<sup>4</sup>Speed/path/direction change on suburban road.

<sup>5</sup>Speed/path/direction change on urban road

SOURCE: AASHTO 1990 Green Book.

### *Intersection Sight Distance*

Intersection sight distance refers to the driver having an unobstructed view of the entire intersection. In the case of no intersection control or minor street yield control, this includes sufficient lengths of the intersecting roadway to avoid potential collisions. However, when the traffic at an intersection is controlled by stop signs or traffic signals, the unobstructed view may be limited to the area of control.

There are two basic types of intersection sight distance that need to be addressed in the design of an arterial interchange: approach and departure. Approach sight distance is the minimum distance that drivers can be from the intersection and still be afforded sufficient time to change speed, path, or direction as necessary. By contrast, departure sight distance is the minimum sight distance that a stopped vehicle must have of the conflicting movements in order

to provide sufficient time to safely enter the traffic stream.

Providing approach sight distance at an urban arterial interchange is difficult, if not impossible, due to the right-of-way limitations, vertical structures, and the surrounding dense development. Nevertheless, every attempt should be made to provide as much approach sight distance as feasibly possible, especially at unusual interchanges such as the SPUI or left hand exit configurations.

Departure sight distance should be provided at the intersection for all movements. This will reduce the potential for accidents when: (1) a violation of red occurs, (2) a malfunction of the signal occurs, or (3) the signal is operating in red/yellow flash mode. Additionally, departure sight distance is necessary at the intersection when right turn on red is permitted.

Messer et al. (24) note the importance of providing adequate sight distance with the SPUI design. Sight distance for the left turn movements is essential due to the increased driver work load and potential danger of crossing such a large conflict area. Special consideration must be given to the additional distance that the left turning vehicles occupy the oncoming traffic lane. At the intersection portion of typical grade separated interchanges, this distance is approximately equal to the width of the lanes being crossed. At SPUIs, however, it can measure 50 to 150 feet.

Although some engineers may argue that sight distance between intersecting traffic flows at a signalized intersection is not required due to the flows moving at different times, AASHTO policy is to provide adequate sight distance based on the Case III procedures. This is substantiated by the increased driver workload at intersections and the hazard involved when vehicles cross or merge with the minor roadway.

## HORIZONTAL AND VERTICAL ALIGNMENT

The combination of horizontal and vertical alignment on the approaches to and through the interchange must compliment one another. This will result in a safer and more efficient interchange. Unfortunately, the horizontal alignment in the urban setting is more or less fixed due to the high costs of right-of-way. As such, the remainder of this section will focus on the vertical alignment of the grade separation.

Perhaps the most important consideration that needs to be addressed in the design of a grade separated structure is whether the main lanes should go over or under the intersection portion. The choice can be dictated by the topography of the region or specific site constraints. Despite the locational factors, there are distinct advantages and disadvantages associated with each design.

### *Underpass Versus Overpass Design*

The underpass design offers several advantages over the overpass design. The depression of the through lanes leaves the grade separation nearly flush with surrounding properties, and obscures most of the retaining wall and abutment system. As such, the underpass design affords much better sight distance at the intersection than the overpass design. Moreover, main lane depression also eliminates the visual and auditory noise that is associated with an overpass in the urban setting. Another notable advantage of the underpass design is that the bridge structure is considerably shorter.

There are, however, several drawbacks associated with the underpass design that are costly and, as a result, quickly diminish its attractiveness. One major problem is maintaining traffic flow through the intersection during construction. Underpass construction is not very conducive to stage construction, and, more often than not, traffic must be detoured around the construction. Careful planning, construction scheduling, and the use of tiedback walls, however, can overcome this obstacle as evidenced at the intersection of Gallows Road and Arlington



Boulevard in Virginia's Fairfax County (30).

Utility relocation can prove to be a large and expensive problem in the underpass design. There are two basic types of utilities: those that are grade-dependent and those that are grade-independent (15). Grade-dependent utilities are primarily sewer and storm drainage while gas, water, telephone, and electric are grade-independent. Any utilities that run parallel with the grade separated major street can be relocated easily and inexpensively to the area of the at-grade lanes. On the other hand, utilities that run parallel with the at-grade cross street will require considerable relocation. The grade-independent utilities can easily be run along the bridge structure, but the grade-dependent utilities must be relocated outside the depressed underpass area. This can add significant cost to the project due to additional reconstruction beyond the depressed area to preserve the natural flow characteristics of these utilities.

Drainage is always an issue in transportation projects but especially troublesome in the design of an underpass. In some cases, the failure to provide positive, reliable drainage is sufficient reason for choosing to carry the main lanes over rather than under. Water removal from the underpass will typically require the construction of a sump and a pump station. Experience has shown that pump stations have a high initial costs, maintenance costs, and power costs (15). Moreover, the possibility of a power outage during storm may result in flooding of the underpass.

Although typically aesthetically unpleasing, the overpass alternative offers many advantages. The primary benefits stem from the fact the existing grade is hardly affected. This means that utility relocation is held to a minimum and maintaining traffic flow during construction is simplified due to minimum impairment of the existing facility. Moreover, drainage of the overpass is gravity-dependent which simplifies the design of the facilities required to carry the storm water.

The primary drawback of an overpass design is that it presents problems in the design of the intersection beneath it. The vertical structures (abutments and retaining walls) severely

limit the sight distance afforded the motorist. The problem is magnified if a unique design such as left hand exits or Single Point Urban Interchanges (SPUIs) are considered. Improving this condition means pulling the abutments further apart, which translates into substantial increases in cost.

### *Length of Grade Separation*

The distance required to adequately design an arterial grade separation is governed by the design speed of the facility, grade of the roadway, vertical clearance required, and the width over which this clearance must be maintained.

The overall length of the grade separation can be calculated using a series of vertical curves connected by tangents. (See Figure 4) By rearranging the equations for crest and sag vertical curves given in the Green Book and simple geometry, the following equation was derived:

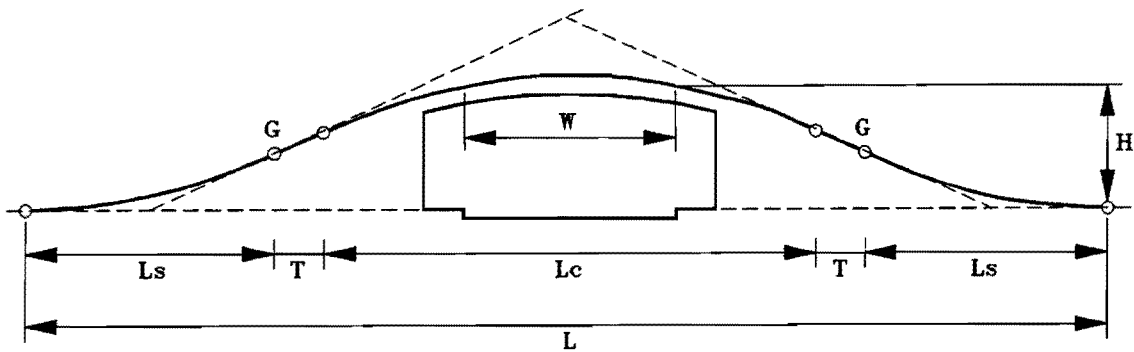
$$L = 2 [ G (K_c + K_s) + T ] \quad (1)$$

where:

$$T = \frac{100 H}{G} - G \left( \frac{K_c + K_s}{2} \right) + \frac{W^2}{8 G K_s} \geq 0 \quad (2)$$

- L = total horizontal length of grade separation (ft),
- T = horizontal length of tangent connecting consecutive vertical curves (ft),
- $K_c$  = rate of curvature for crest curve,
- $K_s$  = rate of curvature for sag curve,
- G = grade used for design of the grade separation (%),
- H = vertical clearance from ground level (ft), and
- W = width over which vertical clearance must be maintained (ft).

**FIGURE 4. Length Required to Effect Grade Separation**



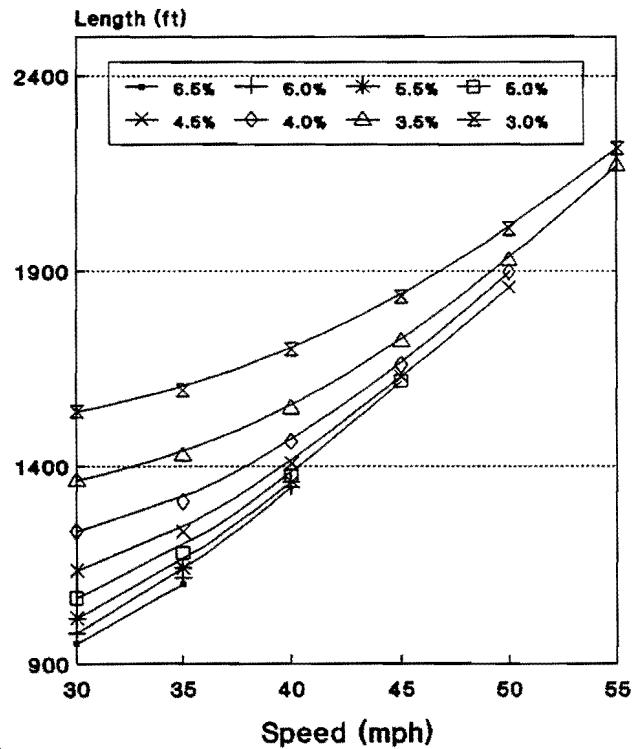
In Equation 2,  $T$  must always be greater than or equal to zero. A negative value for  $T$  represents the situation in which the vertical curves overlap with no tangent transition -- an unsafe and undesirable condition. These equations are valid for both the overpass and underpass configurations.

Using the above equations, an examination of the different types of grade separations can be analyzed. The general characteristics of urban arterials need to be identified such that the comparisons are equitable. According to AASHTO, the recommended design speed for an urban arterial ranges from 35 and 45 mph with 40 mph being preferred. This translates into a maximum grades of between six to seven percent. The 1990 AASHTO Green Book also recommends that vertical clearance from the lane to the underside of the structure be 16.5 feet but not less than 14.5 feet. Consideration must be given to future surface overlay projects which will decrease the vertical clearance. The width over which the vertical clearance must be maintained varies among the types of grade separation and intersection configuration. It will be assumed that: (1) the typical Tight Urban Diamond Interchange (TUDI) overpass situation requires 100 feet, (2) the typical (TUDI) underpass requires 80 feet, (3) the SPUI over pass requires 150 feet, and (4) the SPUI underpass requires 100 feet.

Vertical clearance is not only a function of the tallest design vehicle expected to use the facility, but also the distance the bridge spans. Typically, the longer the bridge structure spans, the deeper the beams required to support the roadway are. AASHTO provides minimum depth-to-length ratios for various types of beams. This analysis will assume a 1:25 (.04) ratio as a minimum. Additionally, it will be assumed that all designs except the SPUI overpass are supported by mid-span bents. This translates into vertical clearances of 20 feet, 19 feet, 23 feet, and 19 feet for the typical overpass, typical underpass, SPUI overpass, and SPUI underpass, respectively.

Figure 5 shows the minimum length of grade separation for a typical overpass as a function of speed for various grades. The curves are based on a vertical clearance of 20 feet and a clearance width of 100 feet. Similar curves are easily developed for other geometric conditions using Equations 1 and 2 and a simple spreadsheet program.

**FIGURE 5. Calculated Grade Separated Lengths for the Typical Overpass<sup>1</sup>**



<sup>1</sup> H = 20 ft., W = 100 ft.

**TABLE 4. Comparison of the Four Grade Separated Scenarios**

Design Speed (mph)	TUDI Overpass Length (ft)							SPUI Overpass Length (ft)						
	6%	5.5%	5%	4.5%	4%	3.5%	3%	6%	5.5%	5%	4.5%	4%	3.5%	3%
35	1116	1140	1177	1231	1307	1428	1597	1227	1262	1311	1379	1473	1618	1819
40	— <sup>1</sup>	1359	1376	1408	1463	1550	1702	1462	1476	1504	1550	1623	1733	1915
45	— <sup>1</sup>	— <sup>1</sup>	— <sup>1</sup>	1631	1661	1723	1836	— <sup>1</sup>	— <sup>1</sup>	1749	1770	1817	1901	2045
Design Speed (mph)	Typical Underpass Length (ft)							SPUI Underpass Length (ft)						
	6%	5.5%	5%	4.5%	4%	3.5%	3%	6%	5.5%	5%	4.5%	4%	3.5%	3%
35	1084	1105	1138	1181	1238	1316	1471	1089	1112	1145	1189	1247	1325	1482
40	— <sup>1</sup>	1326	1339	1367	1417	1490	1592	— <sup>1</sup>	1330	1344	1373	1424	1497	1601
45	— <sup>1</sup>	— <sup>1</sup>	— <sup>1</sup>	1590	1615	1670	1767	— <sup>1</sup>	— <sup>1</sup>	— <sup>1</sup>	1595	1620	1676	1773

<sup>1</sup> Indicates that the vertical curves overlap.

The results of the four scenarios are presented in Table 4. The SPUI overpass configuration requires 10 to 15 percent longer grade separations than the typical overpass. This is due to the longer span required by the SPUI. That is, the longer span translates into deeper beams and larger clearance widths which increase the overall length. There was virtually no difference between the typical underpass and SPUI underpass configurations. Small savings in overall length occur when a typical underpass is chosen over a typical overpass. This is primarily a result of the longer curve length required by a crest vertical curve to maintain safe stopping sight distance when compared that of a sag vertical curve. By contrast, the savings in overall length between the SPUI underpass and overpass can be substantial. The combination of reduced clearance height and clearance width affords the underpass significant savings in grade separation length. Again, this is the result of the long span required by the SPUI overpass configuration versus the SPUI underpass.

A sensitivity analysis was conducted on the grade separation length equations to determine the variables that are most influential on the overall length of the grade separation. The analysis indicated that the overall length is most sensitive to the factors of design speed and vertical clearance. Increases in design speed of 10 to 15 percent result in increases of 15 to 20

percent in length. Similarly, a 20 percent increase in vertical clearance will cause a 10 to 18 percent increase in length. This is evidenced by the substantial savings in length of the typical overpass versus the SPUI overpass ( $H_{typ} = 20$  ft. and  $H_{spui} = 23$  ft.).

Roadway grade can play a major role in the determination of the length of the grade separation, especially for small grades. However, within the feasible design limits for an urban arterial (assumed to be 3 to 6 percent), the decrease in grade is less influential. A 20 percent decrease in grade causes a 10 to 13 percent increase in length depending on the design speed.

The width over which a minimum vertical clearance must be maintained (clearance width) obviously influences the distance required to effect grade separation. Its effects, however, are not independent of the minimum vertical clearance. Put simply, an increase in clearance width requires a proportional increase in the bridge span length. The increased bridge span length creates the need for deeper support beams which, in turn, increases the vertical clearance required from the ground to the elevated roadway surface.

The results presented in this section indicate that any grade separated configurations that demand long bridge spans will also demand long grade separated lengths. These configurations include large intersection areas (such as a SPUI) or designs that necessitate long intersection sight distances.

### *Rate of Vertical Curvature*

Providing adequate drainage is an important factor in the design of urban interchanges. The ability to move water away from the driving surface will reduce accidents, reduce maintenance costs, and increase the efficiency of the roadway. This is reflected by the amount of money spent on drainage facilities which accounts for 20 to 25 percent of highway dollars spent (27).

In order to remove water from the driving surface, it is essential that the vertical

curvature is not too flat. Flat vertical curves are unable to move the water laterally and, as a result, are more prone to accidents caused by hydroplaning and loss of visibility from splash and spray. AASHTO has addressed this issue by assigning a maximum value for the rate of curvature. The value is based on the assumption that 0.3 percent grade should be achieved no less than 50 feet from the apex or nadir of the vertical curve. This corresponds to a maximum K value of 167. Vertical curves with K values higher than 167 will require special drainage considerations near the apex or nadir.

Typically, in the design of the urban interchange, the K values will not exceed 167. This is primarily the result of combining short vertical curves with large grades in order to minimize the impact of the grade separation on surrounding developments. Nevertheless, it is an important and noteworthy design consideration.

## TAPERS, MEDIANS, AND ISLANDS

This section will address design elements that are common to all urban grade separations regardless of configuration. A brief discussion of each design element is provided as way of review.

### *Tapers*

Tapers provide a smooth transition when redirection of vehicles is required. There are four basic types of tapers used in the design of an urban interchange. They are: (1) approach taper, (2) bay taper, (3) departure taper, and (4) lane drop taper.

The approach taper is used in advance of the at-grade intersection in order to provide separate left turn bays. It should provide a smooth lateral transition far enough to the right to shadow the left turn bay(s). These tapers can also be used to introduce a median on the grade separated through lanes. The approach taper is a function of the design speed and, in the urban setting, should approximate  $V^2/60:1$  where V is speed in mph (31).

The function of the bay taper is to direct turning traffic from the through lanes into the turning bay. The bay taper should not be so short as to force an abrupt entry maneuver; nor should it be so long as to confuse the through vehicles. Bay tapers are also a function of the design speed and are approximated by  $V/2.5:1$  where  $V$  is speed in mph (31).

The departure taper is used to narrow the widened at-grade intersection cross-section back down to the mid-block cross-section. This taper should be designed in concert with the left turn lane on the opposite approach. Although no calculation exists for approximating the departure taper, it should be such that it promotes smooth acceleration away from the intersection.

Lane drop tapers need to be such that they afford the driver the opportunity to merge safely into the adjacent traffic stream. Tapers between 30:1 and 40:1 are usually adequate for urban interchanges. A minimum of 20:1 is allowed for low design speeds but is not desirable.

### *Medians*

In the urban environment, medians are typically either raised or flushed. Raised medians offer better drainage and access control than flushed, however, they also increase the chance of the driver losing control of the vehicle during an errant maneuver. Since both types of medians offer adequate guidance for the motorist, local practice or site specific conditions will probably govern the design.

Median widths vary from a minimum of two feet to 15 feet or more along urban arterials. In densely developed areas, the medians are kept at a minimum due to the right-of-way restrictions. The two foot minimum is acceptable along the grade separated lanes and the at-grade intersection provided no pedestrians are expected to cross the intersection. The minimum median width for pedestrians is four feet, but six feet is recommended.



## *Islands*

Islands are not common at conventional limited right-of-way grade separated interchanges for obvious reasons. SPUIs, by contrast, require islands to separate the left turn and right turn off ramp movements. Therefore a brief discussion of islands is warranted.

Islands are used whenever the pavement area at the intersection becomes excessively large for the proper control of the various movements. As such, they should be of sufficient size to command attention. Islands that are too small are ineffective as a method of guidance and often pose maintenance problems. Small islands, less than 75 square feet, should be painted and flush due to their poor target value. Islands should only be curbed when they exceed 75 square feet (32).

Curbed islands should be designed with mountable curbs rather than barrier curbs. This will minimize the chance of the driver losing control of the vehicle should it hit the curb during an errant maneuver. Curbed islands should also incorporate landscaping to facilitate identification and delineation. The materials used to landscape the island must be carefully selected. That is, the landscaping material should be native to the area, require little maintenance, and, above all, should not obstruct sight distance (32).

## ACCESS CONTROL

During the development of our nations largest cities, little attention was given to the amount of access afforded private development from major arterials. The turbulent traffic flow created along the arterials as a result of this neglect has caused severe congestion in many urban areas. Moreover, political pressure and special interest groups are constantly trying to influence the amount of access provided to development along the urban arterial corridor.

Driveways located within the functional area of the arterial interchange tend to complicate an already complex driver work load by introducing additional conflict points. Since reducing

the number of conflict points through an urban interchange will promote safe and efficient operation, it is desirable to impose access control.

Unfortunately, controlling the number of driveways that lie within the functional area of the arterial interchange is difficult. The businesses surrounding the arterial tend to believe that the number of access points to their building is somehow directly proportional to the economic well-being of their establishment. The problem is compounded by the fact that unlimited access has most likely been afforded the businesses in the past. To be sure, the removal of existing driveways will meet significant opposition from the local businesses.

In reality, however, the improved traffic flow resulting from the reduction in conflict points will likely improve their economic condition. The economic benefits to the surrounding businesses stem from reduction of congestion, increased ease in getting on/off site, increased capacity attracting more traffic, and more efficient movements of goods through the area. Nevertheless, it will be a tough proposition to convince local business of these benefits.

## RIGHT-OF-WAY REQUIREMENTS

Right-of-way requirements at an urban interchange depend on the number of lanes on each arterial and the number of auxiliary lanes needed to accommodate the turning volumes. Moreover, the requirements will vary from the at-grade arterial to the grade separated arterial. The remainder of this section will discuss the general right-of-way requirements for high volume arterial-to-arterial intersections.

The typical high volume urban arterial has a mid-block right-of-way width of 100 to 120 feet. This cross-section will generally consist of six 12 foot lanes, a two to 12 foot median or a 14 foot continuous left-turn lane, two foot curb and gutter sections, and easements on both sides for utilities. At the intersection, however, additional right-of-way is required for auxiliary lanes which separate the turning movements from the through movements. The right-of-way required through the at-grade intersection varies from a minimum of 110 feet to 160 feet

depending on the number of through and auxiliary lanes. These increased widths are achieved by flaring the right-of-way prior to the intersection.

Although the grade separated arterial will require approximately the same amount of right-of-way for the mid-block areas, it will need more right-of-way through the intersection than the at-grade arterial. This is especially true for configurations in which the ramp terminals are both signalized intersections. Such configurations will require at least 250 feet of ramp terminal separation so that crossroad vehicles between the terminals can be stored without impeding any other movements. If ramp terminals are spaced less than 250 feet apart, storage cannot be accommodated, and a separate signal phase will be required to clear the vehicles located between the terminals. In this case, the signals must be coordinated so that the storage of turning vehicles does not occur between the ramp terminals. The result is substantial reductions in the capacity of the intersection.

Messer et al. report, based on data from 36 SPUIs, that 300 feet of ramp terminal separation is required for the SPUI configuration (24). This is comparable to the 250 to 350 feet required at TUDI. Thus, it does not appear that the SPUI results in a narrower design than the TUDI. Moreover, as Messer et al. indicate, it appears that TUDIs could easily be constructed in the rights-of-way provided the SPUIs studied. Claims that the SPUI minimizes the amount of right-of-way required are not validated by the Messer study.

## CONSIDERATIONS FOR UNIQUE CONFIGURATIONS

Since many unusual designs are being considered by various jurisdictions to improve the traffic flow and capacity of high volume intersections, special attention must be given to their unique geometrics. Three configurations will be examined in this section. They are the SPUI, left-hand exit configurations, and the vertically split diamond.

## *SPUIs*

The large left turn radii of the SPUI is one of its most unique design features. Earlier studies have indicated that as the left turn radius increases, the saturation flow rate increases (23, 24, 28). The increase in saturation flow rate, however, does not come without some penalty. That is, increasing the left turn radii also increases the lost time per cycle, the uncontrolled pavement area, and the length of bridge structure required to span the intersection (for the underpass case, increased bridge width). Therefore, a radius that provides adequate capacity and maintains safety without an inordinate amount of expenditure is desirable.

The survey of 36 SPUIs conducted by Messer et al. (24) found that left turn radii averaged 200 feet for the cross street to ramp maneuver for both the overpass and underpass designs. Similarly, the average left turn radii for the ramp to cross street maneuver was found to be 210 feet<sup>1</sup> for the overpass design. By contrast, the average left turn radius for the underpass design was found to be 300 feet. The discrepancy is thought to be due to the relatively low increases in the cost of the bridge structure associated with increases in the left turn radius.

The SPUI configuration typically has larger right turn radii than the TUDI. The average right turn radii for both overpass and underpass design on cross street was 100 feet while the off-ramp was found to be 120 feet for both designs. Again, this is considerably larger than the TUDI right turn radii which range from 35 to 75 feet. The need to separate the movements and maintain smooth flow is the rationale.

Due to the SPUIs large area of uncontrolled pavement, there is a need for positive driver guidance and controllability. In the intersection area, pavement markings often accomplished this. Runway lights embedded in the pavement and synchronized with the traffic signal are used at some SPUIs. The lights, however, are expensive and should only be used in special cases

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<sup>1</sup> This average was obtained by eliminating an outlier. The 1000' radius was removed.

(e.g., if a broken back curve is used).

Islands located in each corner of the SPUI also provide guidance. These islands are necessary for the operational efficiency of the merge and diverge maneuvers on the ramps and for the refuge of pedestrians crossing the intersection. The islands are typically very large ranging from 2,400 square feet to 33,000 square feet.

### *Left-Hand Exit Single Signal (LHESS) Configurations*

Conceptually these grade separated interchanges would operate similar to a single signalized intersection. Accordingly, they are presumed to have similar saturation flow rates, cycle lengths, and basic geometric characteristics as existing signalized intersections. The fact that drivers are unfamiliar with such a configuration will necessitate additional signing, pavement markings, sight distance, and roadside safety considerations.

The primary benefit of the left-hand exit design is that it offers a grade separated structure within a narrower right-of-way. That is, while the cross street right-of-way requirements are comparable to the other configurations, the grade separated street can be fit within 140 to 180 feet as shown in Figure 6. This required width is only slightly larger than the required width for an at-grade intersection.

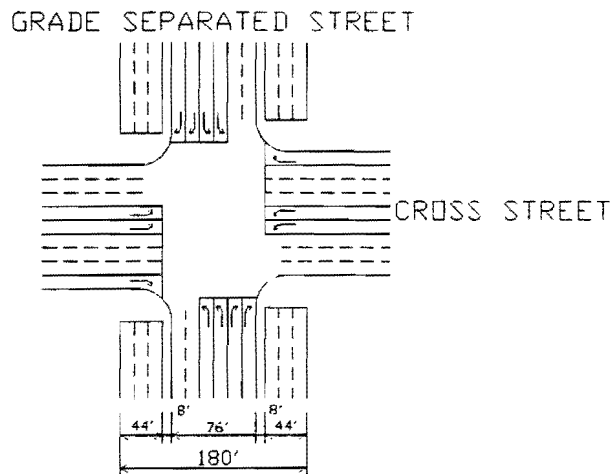
The required length of vertical curvature for the LHESS configuration depends on whether or not mid-span bents are provided. If bents are provided, the LHESS will require the same length as a TUDI. If bents are not provided, the LHESS will require lengths similar to those found in the SPUI design. The bents would be located in the median of the cross street (if provided) which will limit sight distance.

Unfortunately, LHESS configurations are contrary to the concept of driver expectancy. The driver's expectancy for is for right-hand exits. This is the result of intentionally consistent design of right-hand exit interchanges throughout the country. This consistency tends to create

a conditioned response to familiar situations and stimuli. Any design that violates this conditioned response will undoubtedly be a safety problem. The AASHTO Green Book recognizes the potential dangers of violating driver expectancy but also considers their use of left-hand exits satisfactory on low speed facilities.

"Extreme care should be exercised to avoid left-hand entrances and exits in the design of interchanges. .... Left-hand entrances and exits are considered satisfactory for collector-distributor roads; however, their use on high speed, free flow ramp terminals is not recommended." (29)

**FIGURE 6. Layout of the LHESS Configuration**



This reservation on left-side ramps is supported by the literature. Research (37) indicates that the accident potential of left-side on ramps is about 2.3 times that of diamond interchange (traditional right-side) ramps. The accident potential of left-side off-ramps is reported to be nearly 3.3 times that of diamond interchange off-ramps. This higher accident potential of off-ramps and on-ramps may well be due to the fact that left-side on and off-ramps are relatively rare and hence unexpected by the unfamiliar driver. Unfamiliar drivers expecting to exit the freeway may therefore tend to be in the right-lane and make an abrupt maneuver to a left-side ramp. Additionally, the left of a freeway commonly carries higher speed traffic than the right

lane, and hence a higher speed differential between the entering or existing vehicles and through traffic may result with left-side ramps than with right-side ramps. However, a recent TTI study of freeway-freeway interchanges (38) concluded that the accident analysis performed as part of this research was inconclusive in regard to associating left-hand ramps with higher interchange accident rates. While very different from arterial-arterial situations, this research might suggest that similar results are possible with interchanges on arterial streets.

Drivers anticipating a left-turn from an arterial street with at-grade intersections expect to make a left-turn from the left lane or a left-turn auxiliary lane. On an urban arterial with a number of at-grade intersections and access drives, an occasional interchange would be the "unusual" event. Thus, left-turning drivers, especially those not familiar with the roadway, may well be in the left lanes and hence a left-side off-ramp on such a street would not be the same unexpected situation as on a freeway. Thus, left-side ramps may not pose a problem, or as much of a problem as on freeways.

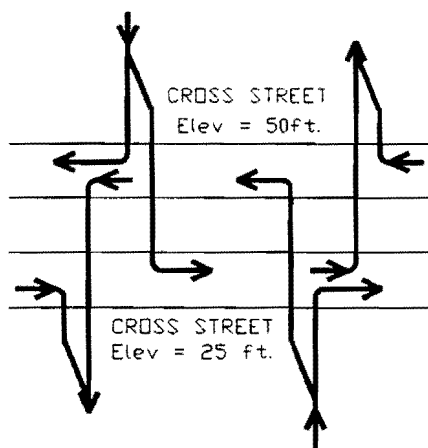
However, no arterial-arterial interchanges with left-side ramp are known to have been built and evaluated for operational accident characteristics. Consequently, no accident potential cannot be factually evaluated.

### *Vertically Split Diamonds*

The vertically split diamond represents a unique and expensive design solution. The configuration is formed by splitting the directional movements on the cross street and grade separating them individually. The elevation of each of the directional movements on the cross street is different forming a three-level interchange. The primary arterial traffic is carried on a third level, underneath the two levels of the cross street. All turning movements are free flowing with short radii at the ramp terminals. No signalized intersections are required on the cross street. A short weaving section is created on each directional overpass between the left-turning movements.

Although this configuration can be constructed within the right-of-way required by a TUDI, it is extremely expensive due to the excessive amount of elevated structures. In all, there are eight ramp structures and two cross street structures. The design approximates a low speed directional interchange (see Figure 6) and is therefore only feasible when the cost of acquiring additional right-of-way exceeds the cost of implementing other alternatives.

**FIGURE 7. Ramp Layout for Vertically Split Diamonds**





## CHAPTER IV

### OPERATIONAL CHARACTERISTICS

The operational characteristics of an arterial interchange are a function of geometrics, volumes, mixture of vehicle types, presence of pedestrians, and signal phasing. Since the operational characteristics along the arterial prior to the interchange are virtually identical between the various configurations, the principal differences will occur within the functional area of the intersection portion. This chapter will analyze the operations of three different arterial interchanges and compare the strengths and weaknesses of each design. The configurations are the TUDI, SPUI, and left-hand exit single signal, which were chosen for their very different at-grade geometrics through the intersection.

TRANSYT 7F was utilized to evaluate the three scenarios. TRANSYT 7F is a macroscopic deterministic traffic model that can be run on most microcomputers. The program assists traffic engineers in evaluating both individual signalized intersections and arterial networks. The program requires geometrics, signal phasing, saturation flow rates, and volumes as inputs. TRANSYT 7F was chosen to compare the three intersections because it affords an infinite number of combinations of the input variables thus allowing an equitable comparison. The SPUI and the left-hand exit single signal configurations each operate as individual intersections despite the differences in geometrics and saturation flow rates. By contrast, the TUDI has two coordinated signals, each with a limited number of phases, which operate as a single signal.

In order to make equitable comparisons between the various configurations, several general assumptions concerning the geometrics and demand volumes are warranted. It will be assumed for each configuration all left-turn movements will be afforded dual left-turn auxiliary lanes. Similarly, an auxiliary right turn lane will be provided for each configuration from the cross street to the ramp. The SPUI and the left-hand exit single signal (LHESS) will have dual right turn lanes from the ramp terminal to the cross street, since no through movements are permitted due to the geometrics. The total approach volume to the interchange is assumed to

be 6000 vph for the low volume scenario and 8000 vph for the high volume scenario. The demand from the cross street is assumed to be 40% of the total approach volume while the remaining 60% utilize the grade separated approaches. Additional assumptions specific to each configuration will be discussed in the appropriate section.

Four scenarios will be run for each configuration; one with light turning traffic at low volumes, one with heavy turning traffic at low volumes, one with low turning traffic at high volumes, and one with high turning traffic at high volumes. Under the light turn scenarios it is assumed that 10% of the demand approach volume requires turning left, while another 10% requires turning right. The heavy turn scenarios will assume that 20% of the demand approach volume requires a left-turn, and 20% requires a right-turn. Table 5 shows the demand volumes used under each scenario.

**TABLE 5. Demand Volumes for the Four Scenarios**

		Low Approach Volume			High Approach Volume		
		LT	THRU	RT	LT	THRU	RT
Low Turning Volume	Ramp	180	200 <sup>1</sup>	180	240	300 <sup>1</sup>	240
	Cross	120	960	120	160	1280	160
All volumes shown are in vph							
High Turning Volume	Ramp	360	200 <sup>1</sup>	360	480	300 <sup>1</sup>	480
	Cross	240	720	240	320	960	320

<sup>1</sup> Volumes only pertain to the TUDI configuration. Not appropriate for the SPUI or the LHSS configurations due to geometric features.

## TIGHT URBAN DIAMOND INTERCHANGE (TUDI)

Of the three configurations discussed in this chapter, the TUDI is the most familiar to the traffic engineer and the motorists. Driver familiarity with the TUDI should not be taken lightly. In fact, it could be argued that this configuration affords a higher capacity for a given movement than the other configurations due to the driver's expectancy. These benefits would diminish as the frequent users became more familiar with the operation of the unique designs, but the infrequent users may still hamper the overall performance of the unique interchange.

Although TUDIs operate effectively in the urban setting, some disadvantages make them undesirable in certain situations. TUDIs are not designed to accommodate large vehicles. For this reason, a high percentage of large trucks in the traffic stream can seriously degrade the performance of the TUDI. If more than 10% of the traffic stream will be composed of large trucks, serious consideration should be given to an alternative design. Either modifying the TUDIs small turning radii or selecting a SPUI configuration are acceptable alternatives.

Another problem often associated with the TUDI configuration in the urban setting is that they are not conducive to two-way arterial progression. The signal sequence most utilized at TUDIs under high volumes is a four phase with overlap. As experience has shown, this type of phasing makes two-way progression along an arterial difficult (24).

The assumptions made for the operation of the TUDI are: (1) intersections are spaced 200 feet apart, (2) left-turn saturation flow rate is 1450 vphgpl, (3) right-turn saturation flow rate is 1500 vphgpl, and (4) four phase signal sequence with overlaps and no permitted left turns. The resulting delays were calculated using TRANSYT and are shown in Table 6 on page 43.

## SINGLE-POINT URBAN INTERCHANGE (SPUI)

The uniqueness of the SPUI stems from the large left-turn movements that geometrically turn inside of each other. This configuration requires a large amount of uncontrolled pavement

through the at-grade intersection. The result is larger clearance intervals and increased probability of erratic maneuvers which ultimately may reduce the overall intersection capacity. Large uncontrolled areas, which require long clearance intervals, may diminish any benefits associated with the operation of the SPUI.

The SPUI gains operational efficiency by utilizing a three phase signal operation. The increase in efficiency is achieved by eliminating the through movement from the ramp terminals. Eliminating the through movement, however, penalizes the pedestrians attempting to cross the cross street by requiring them to remain in the median during the through movements (see Figure 3 on page 12), obviously not a desirable position for the pedestrian to be in.

Right turn maneuvers operate differently at SPUIs than they do at most intersections. The right turn movement from the cross street to the ramp is afforded free flow conditions during two of the three phases (i.e., off-ramp left turns and cross street throughs). As a result, the efficiency of this maneuver may be quite high compared to other configurations, as long as a right turn auxiliary lane is provided. A right turn auxiliary lane is recommended on both cross street approaches.

By contrast, there is some concern about the right turn maneuver from the ramp terminal to the cross street. This maneuver is only afforded free flow movement during the cross street left turn phase. In the remaining time, right turns must yield to the cross street volumes, which proves to be a complex task for the driver. The combination of the sharp merge angle and the large conflict area make it difficult for the driver to locate the conflicting traffic and determine if a gap is adequate for merging. This problem is magnified if dual right turn lanes are used.

Ironically, dual right turn lanes are desirable at high volume SPUIs to reduce potential operational problems caused by the spill-over effects of queues. The dual right turn lanes increase the available storage for the movement thereby reducing the probability that the queues will spill back into the left turn lane(s). Serious reductions in the capacity of the phase will occur if queues are allowed to block the left turn lanes. To this end, shared lane operation on

the ramps is strongly discouraged. Experience has shown that ramps with exclusive right turning lanes of adequate storage length do not experience the adverse effects of vehicle interaction that are associated with shared lane operation (24).

The input variables used in the TRANSYT runs attempted to account for the various quirks associated with the SPUI. The assumptions made are: (1) clearance interval of seven seconds, (2) left turn saturation flow rate of 1650 vphgpl, (3) right turn saturation flow rate of 1500 vphgpl, and (4) a three phase signal sequence. The off-ramp right turns were coded to flow freely during the cross street left turn phase in order to reduce the effects associated with no through movement on the ramps. The delay results calculated using TRANSYT are shown in Table 6 on page 43.

#### LEFT-HAND EXIT SINGLE SIGNAL (LHESS)

This paper examines the LHESS configuration due to the potential right-of-way savings it has over the SPUI and TUDI. Bear in mind, however, that this configuration may increase the number of accidents due to the violation of driver expectancy. It should only be used in special situations as determined by the transportation engineers. The LHESS **must** be provided with ample signing in advance of and at the intersection. **Do not mistakenly assume that adequate signing will eliminate the majority of the accidents.** Driver expectancy is a conditioned response to a set of familiar stimuli and is difficult to compensate for. Providing adequate decision sight distance, good signing, and favorable geometrics are essential if the LHESS is to be implemented.

Operationally the LHESS behaves like an at-grade intersection with no throughs on two approaches (the off-ramps). The turning movements will typically govern the design of the off-ramp approaches, but it is desirable to provide both dual left and right turning lanes. The dual turning lanes will provide increased storage, which reduces the probability of vehicles spilling back on to the main through lanes of the major arterial.

The signal phasing of the LHESS is unique in that the off-ramp right turns might require a separate phase depending on the volumes. The off-ramp right turns can flow freely during the cross street left turn phase. During the remainder of the cycle they must attempt to find adequate gaps to merge into the cross street traffic. If the opposing volumes are heavy and the cross street phase is inadequate to serve the off-ramp right turns, excessive queues may develop. These vehicles can be better served by providing a special right turn phase. Such a phase, however, will reduce the performance of the remaining movements and perhaps reduce the overall capacity.

The assumptions made in the TRANSYT simulation of the LHESS configuration are: (1) left-turn saturation flow rate of 1450 vphgpl, (2) right-turn saturation flow rate of 1500 vphgpl, and (3) a four phase signal sequence with no permitted left turns. The resulting delays are calculated using TRANSYT and are shown in Table 6 on page 43.

## DISCUSSION OF RESULTS

The results of the TRANSYT simulations indicate that the SPUI operated with the least delay for each scenario. The predicted delay for the SPUI ranged from 19.06 veh-hr/hr under low volume and low turning movements to 44.37 veh-hr/hr for the high volume and high turning movements. The SPUI showed the most savings under the high volume and high turning movement scenario. Since the other two configurations operated under a four phase signal sequence, this result is to be expected.

The TUDI operated within 14 percent of the SPUI for the four scenarios. The predicted delays ranged from 21.74 veh-hr/hr to 50.45 veh-hr/hr for the TUDI. Under the high volume and low turning movement scenario the predicted delay for the TUDI was virtually equal to the SPUI. It appears that under high approach volumes and low turning movements, the TUDI is equally effective despite the additional phase.

**TABLE 6. Delay Results of the TRANSYT Analysis on the TUDI, SPUI, and LHESS Configurations (veh-hr/hr)**

Approach Volume/Turning Volume	CONFIGURATION		
	TUDI	SPUI	LHESS
Low/Low	21.74	19.06	20.94
Low/High	29.82	26.92	32.03
High/Low	30.88	30.56	34.92
High/High	50.45	44.37	65.80

The LHESS gave poor results compared to the TUDI and SPUI, except under the low approach volume and low turning movement scenario. Since the LHESS effectively operated as a four phase at-grade intersection, the delays predicted were 10 to 48 percent greater than the SPUI. Using a three phase LHESS would reduce the disparity. A three phase LHESS, however, would effectively operate as a SPUI with lower turning saturation flow rates. Therefore it would never operate as efficiently as a SPUI and was not examined. If relatively low turning volumes are expected and right-of-way is severely limited, the LHESS may prove to be a viable option.

Although it appears prima facie that the SPUI performs best under all scenarios, it is incorrect to assume that it is the optimal solution. Any operational advantage that the three phase SPUI has over the four phase TUDI will quickly diminish as the clearance interval is increased. Many other variables exist that have the potential to quickly reduce the performance of SPUI, such as the complex off-ramp right turn movement and pedestrian volumes.

The SPUI becomes less attractive if the volumes of the simultaneous left turn traffic differ greatly. In the simulation presented in this paper, the simultaneous left turn volumes were

perfectly balanced thereby making the best use of the SPUIs phasing. As a result, the SPUI performed well. The reduced performance is best illustrated by analyzing data presented by Leisch, Urbanik, and Oxley (33) in 1989. Table 7 shows the comparison of total system delay (veh-hr/hr) calculated using TRANSYT for the TUDI and the SPUI under various demand volumes. The table indicates that the larger the disparity between simultaneous left turning movements, the poorer the performance of the SPUI.

**TABLE 7. Comparison of Simultaneous Left Turn Movements and Delay for the TUDI and SPUI**

LOCATION	LEFT TURN FROM (Cross/Ramp)	RATIO (High/Low)	DELAY FOR SPUI (veh-hr/hr)	DELAY FOR TUDI (veh-hr/hr)
1	Ramp	1.17	60.8	67.2
	Cross	1.11		
2	Ramp	1.63	106.8	69.3
	Cross	3.29		
3	Ramp	2.68	69.2	64.2
	Cross	1.18		
4	Ramp	1.52	72.1	46.5
	Cross	3.27		
5	Ramp	1.17	72.0	61.2
	Cross	1.11		

The analysis presented in this paper and in the Leisch et al. paper both indicate that the TUDI is a more appropriate and efficient design in the urban setting. The TUDI is capable of accommodating a greater range of traffic demand more efficiently in most cases than the SPUI. Since travel patterns in the urban area can change significantly over time, the TUDI appears to be the optimal design configuration from the operations standpoint.



## CHAPTER V BENEFITS AND COSTS

The benefits associated with replacing an at-grade intersection with a grade separated structure are primarily derived from reductions in delay. Reducing the amount of time required to negotiate an intersection translates into travel time savings, savings in fuel consumption and environmental savings via reduced emissions. Other benefits such as reduced accident costs and decreased insurance costs have been associated with grade separations. This chapter will investigate the benefits and costs associated with grade separation based on data gathered from six congested intersections in the state of Texas (33). The six intersection locations are:

- Site 1: FM 1960 and Kuykendahl (Houston).
- Site 2: Dairy Ashford and Westheimer (Houston).
- Site 3: Preston and Arapaho (Dallas).
- Site 4: FM 1960 and SH 249 (Houston).
- Site 5: Preston and Beltline (Dallas).
- Site 6: Altamesa and FM 731 (Fort Worth).

### BENEFIT ANALYSIS

The analysis presented in this paper will focus on the time savings, fuel savings, accident reduction, and emission reduction associated with replacing the existing at-grade intersection with a grade separated structure. The methodology requires several key assumptions which are stated below (2).

(1) Average Vehicle Occupancy	---	1.2 persons
(2) Working Days per Year	---	250
(3) Average Cost of Time	---	\$9.75 <sup>1</sup>

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<sup>1</sup> Value obtained from Lomax (2) adjusted using the 1991 Consumer Price Index.

The reduction in delay afforded each study site was calculated using TRANSYT by Raza (33). Raza analyzed a period of six hours: 6:30 to 8:30 am, 11:30 am to 1:30 pm, and 4:30 to 6:30 pm. It will be assumed for these calculations that these six hours constitute 85% of the total delay experienced at the intersection during the day. Using this assumption, the annual time savings was calculated for each site based on the following equation:

$$A T S = Delay Reduc * Veh Occ * Avg Wage * WD/year * hr/day * 1.15 \quad (3)$$

where

A T S	=	Annual Time Savings in dollars,
D R	=	Delay Reduction due to grade separation in veh-hr/hr,
Veh Occ	=	Vehicle Occupancy (1.2 persons per vehicle),
Avg Wage	=	Average Wage (9.75 dollars/person),
WD/yr	=	Working Day per year (250),
hr/day	=	Hours/day of the study (6 hours/day), and
1.15	=	Adjustment to account for the remaining 15% delay.

The potential annual time savings are shown in Table 8 for grade separating in either direction. As shown, substantial annual time savings ranging from \$2.28 million to \$0.78 million can be achieved at five of the six sites. Site 6 would not substantially benefit from grade separation. Interestingly, it appears that from a time savings standpoint, direction of the grade separation makes little difference to these intersections. This implies that each direction is operating with high volume to capacity ratios.

The underlying basis for this type of analysis is that time not spent in travel can effectively be used for other activities. Although intuitively this assumption makes sense, it is difficult to imagine that this time is actually valuable. If an individual can sleep an extra 15 minutes in the morning or arrive at work/home 15 minutes earlier, will this time be effectively utilized? Surely time is money, but quantifying a specific dollar amount is difficult at best. Nevertheless, this report will assume that time can be converted to a dollar amount.

Less nebulous benefits are those resulting from the reduced cost of vehicle operation. These benefits are the result of fuel, oil, maintenance, and vehicle wear savings associated with the reduction in delay. This report will utilize the methodology used by Ismart (34) to determine the fuel savings. The calculations incorporate the incremental fuel consumption due

**TABLE 8. Annual Time Savings Due to Grade Separating at the Study Sites**

Location	Direction of Grade Separation	Total Delay Savings (veh-hr/hr)	Annual Time Savings (1000s Dollars)
Site 1	N/S	112.9	2,280
	E/W	108.2	2,180
Site 2	N/S	55.9	1,130
	E/W	54.9	1,110
Site 3	N/S	69.2	1,400
	E/W	73.4	1,480
Site 4	N/S	41.3	830
	E/W	38.9	780
Site 5	N/S	49.5	1,000
	E/W	72.1	1,460
Site 6	N/S	13.0	260
	E/W	13.4	270

to stopping, speed changes, and idling based on total number of vehicle entering the intersection and the stopped delay. The calculated annual gasoline savings are shown in Table 9. To convert the gallons of gasoline into a dollar amount, the average value of gasoline is assumed to be \$1.25 per gallon. Not surprisingly, the results mimic those found in the annual time savings. That is, Sites 1 through 5 show significant savings in annual fuel while Site 6 only shows marginal savings.

Ismart (34) also developed equations for calculating the amount of emissions caused by stopping, idling, and speed changes. The equations are based on the stopped delay and the total number of vehicles entering the intersection. Using this methodology, the potential emission savings for grade separating each intersection was determined. The results yield a similar pattern and are shown in Table 10.

**TABLE 9. Annual Fuel Savings Due to Grade Separation at the Study Sites**

Location	Gallons of Gasoline Saved Annually	Annual Fuel Savings (1000s Dollars)
Site 1	168,200	210.25
Site 2	82,300	102.88
Site 3	73,800	92.25
Site 4	66,800	83.50
Site 5	70,600	88.25
Site 6	16,600	20.75

**TABLE 10. Emission Savings Due to Grade Separation at the Study Sites**

Location	Annual CO <sup>1</sup> Reduction (lbs)	Annual HC <sup>2</sup> Reduction (lbs)	Annual NO <sub>x</sub> <sup>3</sup> Reduction (lbs)
Site 1	73,400	14,200	10,900
Site 2	31,800	6,600	5,200
Site 3	27,500	6,500	5,300
Site 4	24,500	5,100	4,000
Site 5	29,400	5,600	4,300
Site 6	6,000	900	600

<sup>1</sup> Carbon Monoxide emissions

<sup>2</sup> Hydrocarbon emissions

<sup>3</sup> Nitrogen Oxide emissions

The numbers presented in Tables 8 through 10 indicate that, of the study sites examined, Site 1 would benefit most from grade separation. The benefits are more than twice that of the other locations with an estimated total annual savings of \$2.49 million. Sites 2, 3, and 5 also achieve significant benefits from grade separating with estimated total annual savings of \$1.23 million, \$1.57 million, and \$1.55 million, respectively. Study Site 4 achieves only moderate benefits from grade separation with an estimated total annual savings of \$0.91 million. The least beneficial location from grade separating was Site 6. This site experiences an estimated total annual savings of \$0.29 million, which is three times lower than the next lowest site.

A simple benefit/cost ratio technique can be used to determine if the total annual savings offset the capital cost of building a grade separated structure. For comparison purposes, it is assumed that a grade separated structure built in the densely developed urban environment will cost approximately \$5 million and have a design life of 20 years. Using a capital recovery factor at 12% interest, the annual cost of the grade separated structure is approximately \$0.67 million. This yields a benefit/cost ratio of 3.7, 1.8, 2.3, 1.3, 2.3, and 0.4 for study Sites 1, 2, 3, 4, 5, and 6, respectively. From this simple analysis, grade separation is needed at study Sites 1, 2, 3, and 5. Study Site 4 is a marginal case, and, as a result, a more in depth study is warranted before a decision is made. By contrast, study Site 6 should not be considered for grade separation due to relatively insignificant benefits.

Recent research by Raza and Stover (33) indicates that the cut-off flow rate entering the intersection at which grade separation becomes a feasible option is 5000 vph. The study found that for approach volumes greater than 5000 vph, the delay savings are enough to justify the high cost of grade separation. Using this criteria, study Site 4 does warrant grade separation.

## COSTS

This section will address the direct costs associated with a grade separated structure from a general perspective. The intent is to give a "good feel" for the costs involved, not to provide

exact out-of-pocket costs.

The costs associated with a grade separated structure can be grouped into five categories. They are: (1) the cost of the structure, (2) at-grade roadway improvement costs, (3) the cost of traffic control devices (includes signs, signals, pavement markings, and illumination), (4) the cost of utility relocation, and (5) traffic handling costs (18). These costs typically contribute approximately the same percentage of overall cost from project to project. The majority of the overall cost is related to the structure which will account for 50 to 60 percent. Unusual site constraints, excessively long or short bridges, and clearance requirements can vary the cost of the structure. At-grade improvements, traffic handling, and utility costs are variable from project to project due to the existing site design. Nevertheless they can be approximated as a percentage of the overall cost. At-grade improvements and traffic handling will typically each account for two to eight percent of the final cost. Costs attributed to utilities are usually the most variable ranging from four to 24 percent of the final cost. By contrast, traffic control devices (TCDs) are the most consistent costs from project to project. They approximately account for four to five percent of the overall costs.

Review of the literature indicates that the average cost of a TUDI is approximately \$3.7 million in 1990 dollars. The data ranged from \$2.4 million to \$6.3 million 1990 dollars. The most recent data implies that a range from \$3.5 to \$5.5 million is appropriate for today's urban areas (11, 13, 14, 15, 18, 22, 36).

Experience has shown that the SPUI configuration typically costs between \$1 and \$2 million more than the TUDI due to differences associated with bridges, retaining walls, and earthwork. In one recent example the disparity between the two configurations was \$4 million (22, 26).

The LHESS configuration is basically a flyover with the ramps exiting on the left-hand side. As such, the cost of the LHESS design is estimated using cost figures from previous flyover data. The total direct cost of this design is estimated to range from \$2.2 million to \$5.7

million 1990 dollars (13, 16, 17, 18).

The costs associated with future reconstructing of a particular configuration are often overlooked. Adding lanes to TUDI is relatively inexpensive due to its simple geometric layout, however, these costs can be substantial in the case of the SPUI and the LHESS. The SPUI has a complex structure and layout relative to the TUDI thereby making expansion expensive. Although adding lanes to the cross street of the LHESS configuration can be achieved easily, additions to the ramp approaches are costly.

In summary, the TUDI appears to be the least expensive alternative in terms of both initial cost and future reconstruction costs. Bear in mind, however, that this section only addresses the general issues involved. Every site should be evaluated individually to determine the most feasible design alternative.





**CHAPTER VI  
CASE STUDIES**

The purpose of this chapter is to apply the issues addressed in this report to actual intersections. Two sites were chosen for analysis: one in Dallas (Site 3) and one in Houston (Site 4). Site 3 is the intersection of Preston and Arapaho. Site 4 is the intersection of FM 1960 and SH 249. Table 11 summarizes the pertinent geometric and operating characteristics of each site.

**TABLE 11. Geometric and Operating Characteristics of Sites 3 and 4**

	SITE 3 <sup>1</sup>	SITE 4 <sup>1</sup>
Mid-block R-O-W Width	100 ft (110 ft)	100 ft (120 ft)
Flared Width at the Intersection	110 ft (120 ft)	120 ft (140 ft)
Estimated ADT	52,300 (34,700)	35,000 (39,000)
Design Speed	40 mph (40 mph)	40 mph (40 mph)
Number of Thru Lanes	3 (3)	3 (3)
Number of Left Turn Lanes	1 (2)	2 (1)
Auxiliary Right Turn Lane	SB approach only	NB and SB approaches
Nearest Signalized Intersection to the North	3500 ft	1600 ft
Nearest Signalized Intersection to the South	1500 ft	2200 ft
Nearest Signalized Intersection to the East	2000 ft	1100 ft
Nearest Signalized Intersection to the West	2200 ft	3100 ft

<sup>1</sup>E/W street data is in parenthesis

### SITE 3

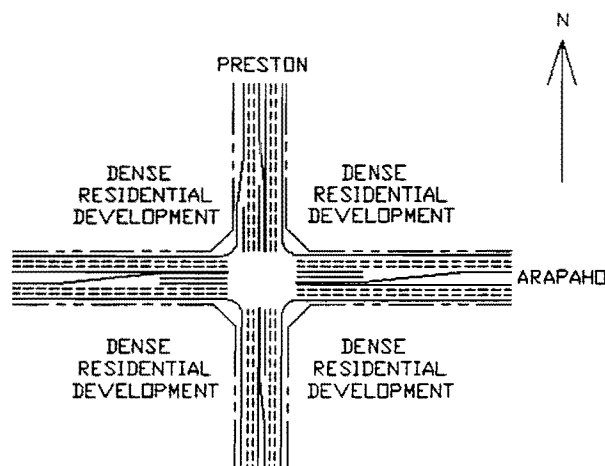
The intersection geometry for the intersection of Preston and Arapaho is shown in Figure 6. Since the previous analysis indicated that no substantial benefit was achieved by grade separating one direction over the other, the first design decision is direction of grade separation. Two factors were examined to determine the appropriate direction. They were: (1) existing geometrics and site constraints, and (2) volume criteria. Geometric and site constraints should be carefully examined to determine if efficient and safe movement can be maintained at a reasonable cost. The volume criteria should examine the relative proportion of through movements on each approach that will be removed from the intersection.

Careful evaluation of the site indicates that Preston is the better choice for grade separating. The decision is based on the compound horizontal curvature on the west approach of Arapaho and the high through volumes present on Preston. Another consideration that favored Preston is the proximity of Beltline and Preston (Site 5) to the intersection. Site 5 is only 1500 feet south of the study site. This intersection is a high volume arterial intersection and would benefit from the free flow along Preston. Delay reductions at Site 5 will occur as the result of the dispersal of platooned vehicles formed by the traffic control at Site 3.

An overpass design was chosen for Preston. This is primarily due to the narrow right-of-way, which is not conducive to efficient traffic handling during construction. Local interest groups, however, might push for an underpass design to reduce the visual and auditory noise impacts on the surrounding residential developments.

Access to Preston will be limited to right turn only or removed altogether for 800 feet either side of the intersection as a result of the overpass structure. Removing and limiting the access points within this length should pose little problem since alternate routes are available throughout the residential developments.

**FIGURE 8. Existing Layout of Site 3**



Selecting the design configuration for this site is difficult due to its narrow right-of-way. In terms of reducing the amount of additional right-of-way required at the site, the best design would be a LHESS. A low type LHESS would only require 140 feet of right-of-way. Despite this advantage, the LHESS will not be used due to the violation of driver expectancy. Instead, a TUDI will be used, which requires the purchase of additional right-of-way. In order to reduce the residential impacts, the ramp terminals will only be spaced 160 feet apart. This reduced ramp terminal spacing will reduce the operational efficiency of the TUDI at-grade intersection, but not significantly.

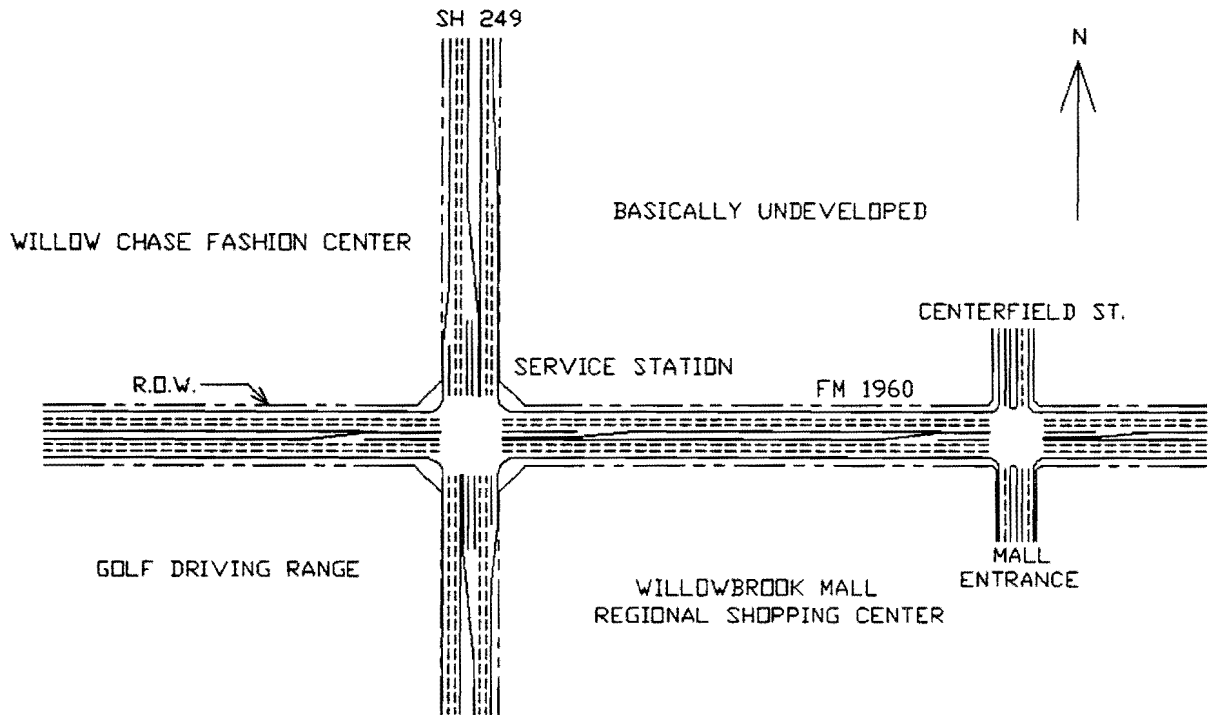
Six lanes are warranted on the overpass due to the volume of through traffic along Preston. The ramp terminals will be provided with one left turn lane and a shared right and through lane. The cross-section of Arapaho will basically remain unchanged.

#### SITE 4

The existing geometrics and surrounding land uses for the FM 1960 and SH 249 intersection are shown in Figure 9. Again, the first step in the design will be determining which roadway to grade separate. At this location the geometric and site constraints do not favor one direction over another. As a result, analysis of the relative through volumes to benefit from the

grade separation will be the criteria used. The turning volumes on SH 249 were much larger than those on FM 1960, at times accounting for 17.5 percent of the total approach volume. Consequently, FM 1960 was found to be the better choice for grade separation.

**FIGURE 9. Existing Layout of Site 4**



Due to the proximity of the intersection of Centerfield and FM 1960 (1100 feet) and minimal utility relocation, an underpass design was chosen. The underpass design can be constructed within a 700 foot distance either side of the intersection, which will ensure efficient operation at Centerfield. Traffic handling is not anticipated to be a problem due to the sparsely developed northeast and southwest quadrants. Land around the golf driving range and the service station are likely to be purchased at a reasonable price. Moreover, the underpass will minimize the visual and auditory noise in the area. Special care should be taken to avoid problems associated with drainage in the underpass.

The impacts on access to the surrounding businesses will be minimal due to the site

designs of Willowbrook Mall and the Willow Chase Fashion Center. Both sites are afforded access points well in advance of the interchange. Additionally, both sites have ring roads that provide ample access to the satellite developments in the area.

The design configuration chosen for this site is the TUDI. The availability of right-of-way in the northeast and the southwest quadrants allows a ramp terminal separation of 250 feet or more. As indicated previously, the operation of a TUDI with adequate ramp terminal spacing is well-suited for the urban environment. The TUDI will also afford more flexibility for future improvements.

The volume of through traffic on FM 1960 warrants a six lane underpass. The ramp terminals are provided with an exclusive left turn lane, a shared through and right turn lane, and an exclusive right turn lane. SH 249 will have the same lane configuration as shown in Figure 9.



## CHAPTER VII

### CONCLUSIONS

Congestion, without a doubt, has been the most significant transportation issue in our nations largest urban areas over the past decade. In these areas, increasing demands are being placed on the arterial networks due to the inordinate delays being experienced on the urban freeways. Since this trend is expected to continue, the major arterial system must be capable of providing mobility to the diverted traffic. The problem is providing more efficient urban facilities within existing right-of-way and on existing alignments.

This research has documented guidelines for the design of urban arterial interchanges in densely developed areas. It addresses the geometric issues, operational issues, and benefits and costs of various interchange configurations. The primary focus is on the Tight Urban Diamond Interchange (TUDI), Single Point Diamond Interchange (SPUI), and the Left-Hand Exit Single Signal (LHESS), but other possible configurations are mentioned where appropriate.

In terms of right-of-way, there is no evident advantage of the SPUI over the TUDI. It appears that anywhere a SPUI can be constructed, a TUDI can be constructed less expensively. Therefore, the assumption that SPUIs minimize the required right-of-way is not valid. By contrast, the LHESS can be constructed within minimal amounts of right-of-way. The decreased right-of-way requirements result from the small conflict area at the at-grade intersection of the LHESS.

In addition to requiring less right-of-way, the TUDI has an advantage over the SPUI when an accident occurs between the ramp gores; traffic can be diverted through the at-grade intersection using the off and on-ramps. This advantage could be a significant benefit in implementing and operating an incident management program.

Vertical alignment is an important consideration in the design of urban interchanges due to its direct relationship to overall cost. Long vertical curves will not only add to the cost but

will also increase the distance over which left turns are restricted. Analysis on the required length for grade separation for the overpass and underpass SPUI and TUDI configurations were performed. The analysis revealed that overpass SPUIs require a 10 to 15 percent longer vertical curve than comparable overpass TUDIs. This is primarily a result of the longer span length required by the SPUI. The longer span requires deeper support beams for the bridge deck, which in turn increases the required vertical clearance over the cross street. The required length of a vertical curve is sensitive to the vertical clearance required. Virtually no differences were observed between the underpass TUDI and SPUI. The required length for grade separation for the LHESS configuration depends on whether or not median bents are utilized. If bents are provided, the LHESS will require the same length as a TUDI. Otherwise, the LHESS will require lengths similar to those found in the SPUI design.

Underpass design offers savings in the required length of vertical curve over the overpass design. This is attributed to the longer sight distance afforded the driver when traversing the sag vertical curve in the underpass. However, the savings are not so significant that they should dictate the type of design. Instead attention should be given to utility relocation, drainage issues, esthetics, sight distance, and traffic handling as justification for overpass/underpass design.

Operational characteristics of an arterial interchange are a function of geometrics, demand volumes, mixture of vehicle types, presence of pedestrians, and signal phasing. The operation of the TUDI, SPUI, and LHESS was investigated using TRANSYT 7F. The results indicate that the TUDI is the most efficient design in the urban setting. The benefits derived from the three phase operation of the SPUI diminish quickly due to long clearance intervals per phase. Moreover, if the simultaneous left turn volumes differ greatly, the performance of the SPUI is greatly reduced. The LHESS configuration operated much like a typical at-grade intersection. Its performance was poor under high demand volumes.

The total direct costs associated with the various configurations are broken down into five categories. They are structure, at-grade improvements, traffic control devices, utility relocation, and traffic handling. The TUDI has an average cost of \$3.7 million (1990 dollars) and can



range from \$2.4 million to \$6+ million. In general, the SPUI is \$1 to \$2 million dollars more expensive due to differences associated with the bridge structure, retaining walls, and earthwork. The total cost of a LHESS design is estimated to range from \$2.2 million to \$5.7 million. Costs associated with the future expansion of the SPUI and LHESS are much higher than those for the TUDI.

Table 12 subjectively compares the results of this report for the TUDI, SPUI, and LHESS configurations. This report indicates that, in general, the TUDI is the most appropriate design in densely developed urban areas. The familiar geometrics and efficient operation in a variety of traffic patterns of the TUDI make it an attractive alternative.

**TABLE 12. Comparison of the Results for the TUDI, SPUI, and LHESS**

CHARACTERISTIC	TUDI	SPUI	LHESS
ROW Requirement	Moderate	Moderate	Low
Costs	Moderate	High	Moderate
Sight Distance Requirements	Low	Moderate	High
Length of Vertical Curves	Low	Moderate	Low
Driver Expectancy	Meets	Violates Slightly	Violates
Accommodation of Pedestrians	Good	Poor	Good
Accommodation of Heavy Vehicles	Poor	Good	Poor
Operation Under Varying High Volume Scenarios	Good	Fair	Poor

The benefits associated with grade separation considered in this report are time savings, fuel savings, and emission reduction. By assigning dollar values to time and fuel savings, a

benefit/cost analysis was performed on six study sites in the state of Texas. Only four of the sites appear to decisively warrant grade separation based on the methodology employed. They are: Site 1 -- FM 1960 and Kuykendahl (Houston), Site 2 -- Dairy Ashford and Westheimer (Houston), Site 3 -- Preston and Arapaho (Dallas), and Site 5 -- Preston and Beltline (Dallas). Site 4 -- FM 1960 and SH 249 in Houston was found to be a marginal case and requires further study. At Site 6 -- FM 731 and Altamesa in Dallas, grade separation was not justified.

Site 4 was justified as warranting a grade separation (33). This report indicates that the cut-off flow rate entering the intersection at which grade separation becomes a feasible option is 5000 vph. The study found that for approach volumes greater than 5000 vph, the delay savings are enough to justify the high cost of grade separation.

## REFERENCES

1. M.D. Meyer. Chapter 1 -- Traffic Congestion: An Overview. A Toolbox for Alleviating Traffic Congestion. Institute of Transportation Engineers, Washington, D.C. 1989. pp. 1-9.
2. J.L. Hanks, Jr. and T.L. Lomax. Roadway Congestion in Major Urban Areas 1982 to 1987. Research Report 1131-2, Texas Transportation Institute, October 1989.
3. A.T. Reno. Personal Mobility in the United States. Special Report 220, Transportation Research Board, Washington, D.C. 1988. pp. 369-393.
4. J.R. Stowers. Organizing and Funding Transportation for 2020. Special Report 220, Transportation Research Board, Washington, D.C. 1988. pp. 477-506.
5. Highway Statistics 1981. FHWA, U.S. Department of Transportation, Washington, D.C. 1982.
6. Highway Statistics 1986. FHWA, U.S. Department of Transportation, Washington, D.C. 1987.
7. Highway Statistics 1990. FHWA, U.S. Department of Transportation, Washington, D.C. 1991.
8. D.L. Christiansen and W.V. Ward. An Enhanced Role for the Arterial Street System in Texas Cities. Research Report 1107-1, Texas Transportation Institute, November 1988.
9. T.F. Humphrey. Chapter 3 -- Highways - Getting the Most Out of the Existing System. A Toolbox for Alleviating Traffic Congestion. Institute of Transportation Engineers, Washington, D.C. 1989. pp. 19-68.
10. R.P. Kramer. New Combinations of Old Techniques to Rejuvenate Suburban Arterials. Strategies to Alleviate Traffic Congestion. Proceedings of the Institute of Transportation Engineers 1987 National Conference, Washington, D.C. 1987.
11. JEF Engineering. High Flow Arterial Concept: Feasibility Study and Evaluation of Case Studies for the Orange County Transportation Commission. Santa Ana, California. May 1982.
12. AASHTO. A Policy on Design of Urban Highways and Arterial Streets 1973. American Association of State Highway and Transportation Officials. 1973.

13. S.R. Byington. The Fly-Over: A View from Both Sides. Transportation Engineering Journal, American Society of Civil Engineers. November 1981. pp. 667-680.
14. Parsons, Brinckerhoff, Quade, and Douglas. Super Streets Demonstration Project: Beach Boulevard Corridor, Final Report. Prepared for the Orange County Transportation Commission, California. March 1986.
15. Van Dell and Associates. Super Streets Program for Orange County. Prepared for the Orange County Transportation Commission, California. June 1984.
16. W.W. Recker, M.G. McNally, and G.S. Root. Flyovers and High Flow Arterial Concept. Transportation Engineering Journal, American Society of Civil Engineers. March 1985. pp. 139-154.
17. W.W. Pleasants. The Fly-Over: It Unclogs Urban Traffic in a Hurry. Civil Engineering, American Society of Civil Engineers. May 1980. pp. 71-75.
18. C. Bonilla and T. Urbanik II. Increased Capacity of Highways and Arterials Through the Use of Flyovers and Grade Separated Ramps. Research Report 376-1, Texas Transportation Institute. June 1987.
19. M.D. Meyer and E.J. Miller. Urban Transportation Planning: A Decision-Oriented Approach. McGraw-Hill Publishing Company. New York, New York. 1984.
20. C.R. Walker. Investigation of the Through Lane Overpass: Warrants for Highway-Highway Grade Separations. Illinois Cooperative Highway Research Program, Project IHR 55. Northwestern University, Evanston, Illinois. September 1966.
21. A.J. Horn. Diamond Interchange Applications in Constricted Rights-Of-Way. Technical Views. Howard, Needles, Tammen, and Bergendoff, New York, New York. Winter 1989. pp. 30-34.
22. J.P. Leisch, T. Urbanik II, and J.P. Oxley. A Comparison of Two Diamond Interchange Forms in Urban Areas. ITE Journal. Institute of Transportation Engineers, Washington D.C. May 1989. pp. 21-27.
23. M.J. Poppe, A.E. Radwan, and J.S. Matthias. Some Traffic Parameters for the Evaluation of the Single-Point Diamond Interchange. Paper No. 910414. Presented at the 70<sup>th</sup> Annual Meeting of the Transportation Research Board. Washington, D.C. January 1991.
24. C.J. Messer, J.A. Bonneson, S.D. Anderson, and W.F. McFarland. Single Point Urban Interchange Design and Operations Analysis. National Cooperative Highway Research Program (NCHRP) Report 3-40. October 1991.

25. S.J. Brown and G. Walters. The Single-Signal Interchange. Institute of Transportation Engineers 1988 Compendium of Technical Papers. Institute of Transportation Engineers, Washington, D.C. 1988. pp. 180-184.
26. B. Martin. Urban Diamonds -- Justified or Not? Technical Views. Howard, Needles, Tammen, and Bergendoff, New York, New York. Winter 1989. pp. 22-30.
27. N.J. Garber and L.A. Hoel. Traffic and Highway Engineering. West Publishing Company. St. Paul, Minnesota. 1988.
28. J.A. Bonneson and C.J. Messer. A National Survey of Single Point Urban Interchanges. Research Report 1148-1, Texas Transportation Institute. March 1989.
29. AASHTO. A Policy on Geometric Design of Highways and Streets 1990. American Association of State Highway and Transportation Officials. 1990.
30. R. Robison. Tiedback Walls Untie Intersection. Civil Engineering, American Society of Civil Engineers. January 1986. pp. 44-45.
31. T.R. Neuman. Intersection Channelization Design Guide. National Cooperative Highway Research Program (NCHRP) Report 279, Transportation Research Board. November 1985.
32. V.G. Stover and F.J. Koepke. Transportation and Land Development. Institute of Transportation Engineers (ITE), Washington D.C. 1988.
33. S.S.Raza and V.G. Stover. An Analysis of Expected Delay Reduction By Replacing A Congested At-Grade Intersection with an Interchange. Research Report 1237-1, Texas Transportation Institute. July 1991.
34. D. Ismart. Mobile Source Emissions and Energy Analysis at an Isolated Intersection. TRR 842, Transportation Research Board, Washington, D.C. 1982. pp. 5-10.
35. Consumer Price Index (CPI) Detailed Report, October 1991. U.S. Department of Labor and Bureau of Labor Statistics. pp. 67.
36. J.M. Witkowski. Benefit Analysis for Urban Grade Separated Interchanges. Transportation Engineering Journal, American Society of Civil Engineers. January 1988. pp. 93-109.
37. J.E. Leisch, J.W. Hess, and M.J. Rosenbaun. Chapter 6 -- Interchanges, Synthesis of Safety Research Relative to Traffic Control and Roadway Elements. Volume 1, Report FHWA-TS-82-232. Federal Highway Administration, December 1992.

38. J.W. Hanks, Jr., W.L. Gister, S.J. Taylor, and J.M. Mounce. Operational Evaluation of Effects Resulting from Freeway - Freeway Interchange Geometrics. Research Report 1232-4, Texas Transportation Institute, February 1992.