NCHRP-345

Traffic

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NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **345**

SINGLE POINT URBAN INTERCHANGE DESIGN AND OPERATIONS ANALYSIS

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TRANSPORTATION RESEARCH BOARD
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SINGLE POINT URBAN INTERCHANGE DESIGN AND OPERATIONS ANALYSIS

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

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WASHINGTON, D. C. DECEMBER 1991

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NCHRP REPORT 345

Project 3-40 FY '89

ISSN 0077-5614

ISBN 0-309-04868-0

L. C. Catalog Card No. 91-68525

Price $12.00

Areas of Interests
Facilities design
Operations and traffic control
Traffic flow, capacity, and measurements

Mode
Highway transportation

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report will be of interest to transportation officials involved in the planning, design, construction, and traffic operational aspects of the Single Point Urban Interchange. The SPUI essentially combines two separate diamond ramp intersections into one large at-grade interchange which accommodates all interchanging vehicular movements and the through traffic. A major application of the SPUI has been as a complete replacement for a major, at-grade signalized intersection of two principal urban arterials. In this application, the SPUI provides nearly twice the available traffic handling capacity as the intersection it replaces. Other significant applications include the upgrading of outdated signalized interchanges, new freeway construction, and reconstruction of urban arterials along a corridor to expressway standards.

Under NCHRP Project 3-40, research was undertaken by Texas A&M Research Foundation to (1) document current practice in design and traffic operations of existing SPUIs and (2) to develop and document guidelines for the design, traffic operational analysis, and cost-effectiveness of SPUIs.

To achieve the project objectives a nationwide survey of existing SPUIs was conducted in 1989 and some limited capacity data were obtained in 1990. An extensive database on planning issues, traffic operations and capacity, safety, geometric design, bridge design and construction costs was developed for 36 SPUIs in 17 states. Extensive analyses were performed, and summarized by the researchers, using interviews with officials responsible for existing SPUIs, on-site observations, traffic records, construction plans, video tapes, and data from plans and photographs of signing, signal control, and pavement markings.

Based on the information developed and through dialog with the NCHRP panel responsible for monitoring the project, the researchers prepared a series of general recommendations for planning and design of SPUIs. In addition, more specific, detailed application guidelines are provided in the report under the chapters on systems planning and design, interchange design, traffic control device applications, and cost-effectiveness analysis. A summary of SPUI implementation and application guidelines is provided in the closing chapter. Thus, this report will be an invaluable resource for transportation officials who are considering new or upgraded interchanges in response to urban traffic congestion.

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ACKNOWLEDGMENTS

Dr. Carroll J Messer, Research Engineer, served as the principal investigator for NCHRP Project 3-40. The primary authors for this report are Dr. Carroll J. Messer and Dr. James A. Bonneson, formerly an Assistant Research Engineer. Dr. Bonneson is now an Assistant Professor at the University of Nebraska—Lincoln. The cost-effectiveness chapter was written by Dr. Stuart D. Anderson, Assistant Research Engineer, and Dr. William F. McFarland, Research Economist at Texas Transportation Institute. Consultants to the project were Mr. Kenneth G. Cook, Chief Civil Engineer, and Mr. Merrill R. Cotten, Chief Cost Estimator, DeLeuw, Cather & Co., Chicago, Illinois, and Mr. Jim Clark, Traffic Systems Engineer, Barton-Aschman Associates, Inc., Washington D.C.

Special thanks are extended to all those state and city engineers who contributed in a significant way to this information gathering and synthesis effort. A special appreciation is expressed to those participants from Alabama, Arizona, Colorado, Florida, Georgia, Illinois, Kentucky, Kansas, Nebraska, New Mexico, Minnesota, Missouri, Tennessee, Utah, and Virginia. Considerable insight and guidance were provided by Mr. T. Wallace Hawkes, III, Executive Vice President, Greiner, Inc., Tampa, Florida.

SINGLE POINT URBAN INTERCHANGE DESIGN AND OPERATIONS ANALYSIS

SUMMARY

As stated in the research project statement, the Single Point Urban Interchange (SPUI) essentially combines two separate diamond ramp intersections into one large at-grade intersection which accommodates all interchanging vehicular movements and the through traffic. Signalization of the one major intersection simplifies coordination on the arterial. It has been reported that SPUIs can significantly increase traffic-carrying capacity compared with the conventional diamond interchange.

There were numerous uncertainties about the design and operation of SPUIs and, yet, many were being proposed for construction for addressing traffic congestion and safety problems. Some of these issues included: wrong-way movement potential; traffic signal, signing, and delineation requirements; sight distance; cost-effectiveness; increased capability to accommodate heavy traffic movements; safety problems; and driver behavior.

NCHRP Project 3-40 was initiated in response to the need for technical information on the SPUI in the form of a synthesis of current practices in the United States. The two main objectives of the research were specified as: (1) document current practice in design and traffic operations at existing SPUIs and (2) develop and document guidelines for the design, operation, analysis, and cost effectiveness of SPUIs.

This report addresses these broad technical objectives on the SPUI, information is provided on the historical development of SPUIs, typical geometric and bridge design, observed traffic operations, and general traffic engineering applications. In particular, the report presents a wide variety of statistical summaries of geometric and operations data collected for 36 SPUIs, most of which were observed during a field survey of SPUI operations in 13 states, which was conducted during the summer of 1989. Following a synthesis of these data, geometric and operational guidelines are provided for informational purposes, because there are no specific guidelines on the interchange in the 1990 AASHTO Green Book. Advantages and disadvantages of various SPUI design features are given. The report also contains guidelines for conducting cost-effectiveness studies of the SPUI as compared primarily to the Tight Urban Diamond Interchange (TUDI). A recommended methodology is presented and special SPUI features are identified for which local cost estimators might not be familiar. Updated operational user benefits estimation procedures are also given. The report concludes with a summary of research findings and recommended application guidelines for SPUIs as being a viable competitor to the TUDI for congestion relief in restricted urban conditions. In this context, the SPUI was found to be a safe and efficient interchange design alternative.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

BACKGROUND

Capacity Needs

An explosive growth in population and traffic demand occurred in the United States following World War II as many people moved to urban areas where good jobs were plentiful. Highway engineers responded to this forthcoming traffic snarl by constructing one of the largest interstate highway systems known. Much of the Interstate System was built on new rights-of-way at modest cost as plenty of open land, which is mostly developed today, was available.

Most metropolitan areas continue to experience increased traffic growth, population gains, and traffic congestion on their urban freeways. Highway travel demand in urban areas is increasing about 2 percent per year, with much higher growth rates occurring in some urban areas. Suburban traffic growth on principal arterial streets also has been significant. Capacity bottlenecks and safety problems are occurring at major intersections of principal arterials and are becoming more common. Many urban freeways not only have serious traffic congestion and related safety problems, but they are also experiencing structural deterioration as these older facilities near the end of their design life.

Transportation planners are faced with a difficult challenge to find solutions that can maintain a high level of urban mobility and safety in a cost-effective manner. No longer can an engineer just "move over" into a nearby open space and build a new highway. More efficient highway facilities have to be constructed to fit present alignments basically within existing rights-of-way. To further complicate matters, high volumes of traffic must be handled safely within construction work zones. Additionally, any new construction must be done as rapidly as possible to minimize traffic delays and impacts on the local business community.

Many traditional designs are not well suited for providing added capacity within restricted urban environs. Consequently, urban highway engineers have sought new and innovative solutions to relieve traffic bottlenecks and to meet current urban mobility needs. This report presents a new form of signalized urban diamond interchange that provides significant added capacity for addressing a wide variety of arterial and freeway congestion problems occurring at existing cross arterial facilities.

Single Point Urban Interchange

The new signalized interchange form being proposed for providing added capacity in restricted urban rights-of-way is the "Single Point Urban Interchange," or SPUI as it is called in this report. Several other names are used for the SPUI depending on the locale, including the Urban Interchange. The SPUI is a recent addition to the types of grade-separated interchanges that highway engineers can use to improve traffic flow along a major traffic artery. Considerable interest in the SPUI has been shown by many states in recent years because of its claimed efficient operation, although little information is presently available on its design features or operational performance. The SPUI was not mentioned in the 1990 AASHTO Green Book (*1*) even though one was shown on page 559 therein.

RESEARCH OBJECTIVES

The objectives of this project were to: (1) document current practice in design and traffic operations at existing SPUIs and (2) develop and document guidelines for the design, operation, analysis, and cost effectiveness of SPUIs.

To meet these objectives, the research concentrated on the following tasks: (1) Determine the state of the art and current practice through a review of the literature and contacts with highway agencies planning, designing, constructing, operating, and maintaining SPUIs. (2) Develop guidelines to assess the cost effectiveness of SPUIs in comparison with alternative design solutions on a life-cycle basis, including first costs, continuing agency costs, user costs, and environmental costs. (3) Develop guidelines for geometric design for use with the AASHTO "Green Book". (4) Develop guidelines to analyze the functional performance of SPUI traffic operations throughout its design life. Develop criteria for the optimum placement and operation of traffic control devices. (5) Define safety considerations and develop recommendations for dealing with the impact of these factors.

RESEARCH APPROACH

Field Survey

A nationwide field survey of single-point urban interchange design and practice was conducted during 1989 followed by limited capacity studies in 1990. The synthesis of current SPUI practice was taken primarily from these field observations and is therefore somewhat subjective. As of August 1, 1989, there were 27 SPUIs operational in 14 states in the United States and more than 50 others planned or under construction. At least 10 new SPUIs have become operational since the field survey was conducted. At least 10 states have built a subsequent SPUI following successful operational experience with their first one.

In general, traffic operations and safety experience with SPUIs have been good. Motorists appear to adapt well to the somewhat novel operating environment of the SPUI when good interchange-level signing is applied. Because SPUIs are very new, motorist's driving skills at SPUIs are expected to improve with time.

Design Features

There are several distinguishing design and operational features about a SPUI that give its name and nature. The SPUI is a grade-separated two-level diamond interchange well suited for restricted urban rights-of-way. A conventional overpass SPUI recently opened to traffic is shown in Figure 1. As an indication of scale, the single-span overpass bridge is about 220 ft long. The principal operational feature of the SPUI is that it has only one signalized intersection through which all four left turns operate. As found at a conventional high-type intersection, all opposing left turns operate inside one another.

Geometry is also greatly influenced by the high-speed single-signal operations of the left turns. To provide adequate visibility and efficient direct left-turn operations, the SPUI uses large left turning radii on the order of 150 to 300 ft for the off-ramps from the high-speed mainline facility. Consequently, a clear center span of 200 ft or more is needed over the intersection of a single-span overpass. Both single-span and multispan bridges are used in SPUI design with both overpass and underpass mainline grade separations. While most SPUIs do not have frontage roads, a few do. When the freeway goes under the cross street, a wide two-span platform bridge is normally used having a median supporting pier. The thinner two-span overpass promotes retaining the existing cross street grade line which minimizes difficulties in alignment design and reduces project costs. The total cost of a SPUI interchange may range from $8 million to 14 million, or more, depending on the size of the interchange, and on local right-of-way and construction costs.

The following section presents a brief history of the SPUI in the United States. Personal interviews, correspondence, and relevant literature were used as data sources for preparing this section.

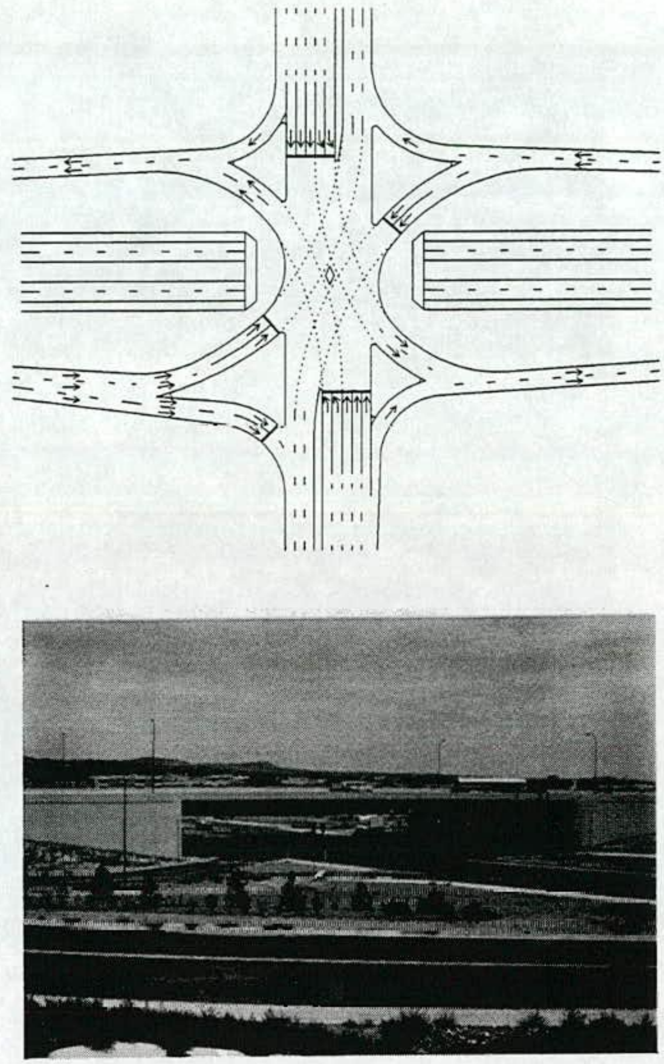


Figure 1. Overpass single-point urban interchange.

Historical Context

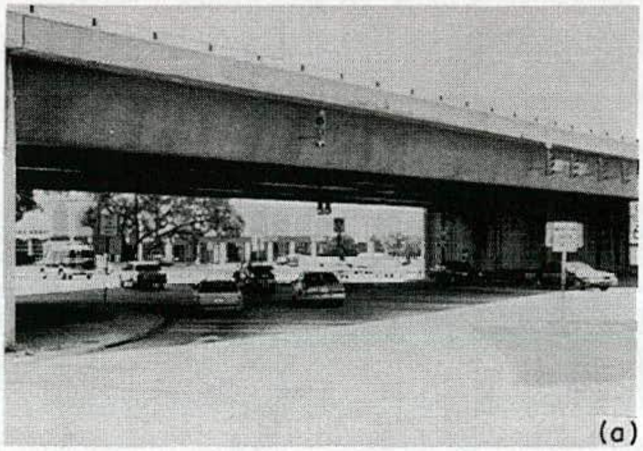
The field survey indicates that the SPUI was first proposed in the United States as a design alternative in the mid-1960s almost simultaneously, but independently, by at least two prominent American civil engineering firms (*2, 3*). Both of these proposed designs were later built in the 1970s. One SPUI was in Clearwater, Florida (*2, 4*) and the other was in Moline, Illinois (*3, 5*). In addition, an early commitment about this same time was also made to use several SPUIs along one major arterial in Huntsville, Alabama, which now has one-way frontage roads.

Other sources provide additional insight and perspective on early developments of the SPUI. One reference suggests that prior to the above actions, Caltrans had proposed an innovative "inside-left turn" interchange design for Palo Alto, California, in 1960 (*6*). Interchanges similar to the SPUI have been built in Germany (*7*) and Greece (*8*), to name a few. The German SPUI was built in Cologne near the Rhine River by 1975 (*7*). Several are programmed for construction in Edmonton, Alberta, Canada. As noted earlier, the 1990 AASHTO highway design policy does not mention the single-point urban interchange as being a viable interchange design type (*1*). Recent implementation and operational experience strongly suggest that the SPUI should be considered in future editions of the Green Book. This research should provide useful guidance to that effort.

The first SPUI built in the United States was completed in Clearwater, Florida, February 25, 1974. This interchange was designed by Greiner Engineering of Tampa, Florida. Greiner also proposed the added use of airport runway lights embedded in the pavement to clearly delineate the left-turn paths of vehicles at the Clearwater site. Greiner called this SPUI interchange an "Urban Interchange" (*2, 4*).

The term "urban interchange" is used by many engineers to describe the SPUI. This is mainly because of the national effort by Greiner to inform engineers about the features of the urban interchange. Other terms for the SPUI are used by state agencies, such as a "single-signal diamond," a "single-point diamond," and an "urban traffic interchange." Greiner gave a pioneering presentation on the features of their "urban interchange" to a Florida highway design conference held in Gainesville, Florida, on March 11, 1970. They also gave a similar presentation to a regional meeting of the AASHTO Committee on Highway Design held later that year in Houston, Texas, on November 11, 1970.

Attending the AASHTO design conference was the Illinois DOT district engineer from Dixon, Illinois, responsible for construction plans being prepared for the SPUI to be built in Moline,



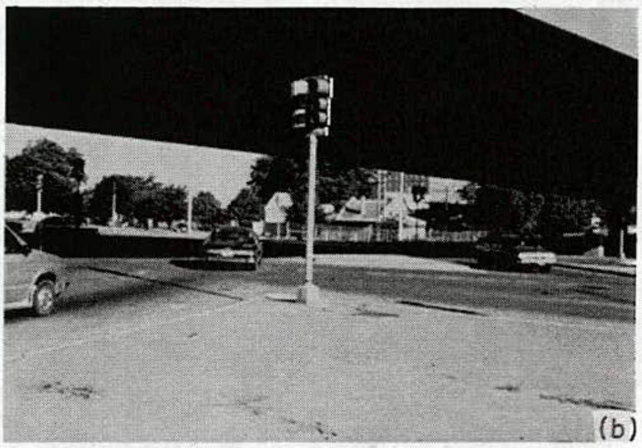


Figure 2. Pioneer SPUIs built in the United States: (a) Clearwater, Florida; (b) Moline, Illinois.

Illinois. After hearing the Greiner presentation, he had runway marking lights added to the Moline construction plans, which were completed May 31, 1972. The Moline contract was let May 22, 1973 with job completion on September 9, 1975. Eagle Signal, then located nearby in Davenport, Iowa, assisted Illinois DOT in the design and signal installation of the pavement marking lights. Thus, the Moline, Illinois, site was probably the second SPUI constructed in the United States. The preliminary engineering design study on this SPUI was done by DeLeuw, Cather & Co. of Chicago (*3*). One of the consultants to this NCHRP study was the engineer in charge of the Moline job (*5*). Figure 2 presents a photograph of these pioneer interchanges in Clearwater, Florida, and Moline, Illinois, respectively. Both SPUIs are still operational today.

Literature Review

The literature on the single-point urban interchange deals with two areas of interest. The first and earliest literature introduced the SPUI to the profession and described its basic features. This work has been previously referenced in the foregoing historical review together with other sources. Greiner Engineering was the apparent industry leader in spreading the word about the SPUI to the profession (*2, 4*).

The second area of literature generally provides introductory material about the basic features of the SPUI and, in a few cases, the sources also contain some preliminary operational experiences and assessments of the SPUI. Most of the evaluations have used computer analyses to infer SPUI operational quality. Assumptions were usually made regarding traffic behavior although little actual field data were obtained. Examples of this type of literature are noted (*9, 10*).

Three recent operational studies of SPUI headway characteristics have been published by university graduate students (*11, 12, 13*). These provide the only known SPUI traffic data collected and formally reviewed by others. These three studies were used in this research to provide updated capacity guidelines on SPUI traffic operations. An introductory cost-effectiveness paper is also noted (*14*). As more design and operational experience is gained with SPUIs, additional published reports on them are expected.

Diamond Interchange Alternatives

Field interviews suggest that some confusion may exist in identifying viable design alternatives to the SPUI in urban design. Some highway engineers and planners, well versed in rural highway and major interchange design, may not be as familiar with the efficiency of other signalized diamond interchange options in very tight right-of-way locations likely to arise in costly urban reconstruction work. The background for this possible confusion follows.

Rural Design

In the early stages of the Interstate highway program, state highway departments built many conventional diamond interchanges in rural areas. In these low-volume, low-cost rural settings, cross-road ramp terminals were often separated by 1,200 ft, or more, depending on the cross-road design speed, usually about 60 mph. These cross roads were narrow (two-lane) and any needed cross-road left-turn lanes to the on-ramps were provided totally off the bridge. Total transition and storage length per bay was about 500 ft at 60 mph. Thus, a minimum ramp separation of 1,200 ft would be required for an overpassing bridge 200 ft long. These designs were called "conventional diamond interchanges" by some state highway agencies.

As highway agencies continued building the freeway network into more developed areas having more expensive rights-of-way these agencies began to compress the size of the diamond interchange used to minimize land takings. This reduction was achieved by overlapping the tapers of the cross-road left-turn lanes on the bridge. The bridge had to be widened one lane to provide the overlapping left-turn tapers. In higher speed rural designs, ramp separation distances of 730 to 800 ft would result. In lower speed urban designs, separation distances of 600 ft could result. These overlapping taper designs were called "compressed diamonds" by some highway agencies.

The traffic signal control used at the compressed diamond interchanges is often provided by two independently operating traffic-actuated signals. Traffic signal coordination needed between the two actuated signals to provide smooth cross street flow and to minimize queueing backups is difficult to attain in this case. Signal coordination is seldom attempted with compressed diamonds. However, the need for coordination grows with increasing traffic volumes and reduction in the distance between the signals. Queue storage space becomes more critical as the separation distance is reduced.

Urban Design

A smaller "tight diamond" from a rural design viewpoint is sometimes used in urban areas where right-of-way costs have climbed dramatically. Traffic volumes have also risen but not to the level requiring a full freeway-to-freeway interchange. In earlier times when costs were lower, a parclo interchange may have been considered as a viable design alternative. A parclo requires extensive right-of-way on two quadrants of the interchange and is now usually not considered a viable design alternative for reconstructing an existing urban facility. On the other hand, a tight diamond can be used, which has ramp separations of 500 to 600 ft, sometimes less. Coordination of the two signals is required, but it has become much more complex to achieve than in the prior cases. Providing progressive flows across the bridge in both directions is extremely difficult using two traffic-actuated signal controllers. In addition, single controller operation is not usually used here because it does not work efficiently at ramp intersection separation distances of 500 to 600 ft. To say the least, operational experience with these tight diamonds has not always been good.

Tight Urban Diamond Interchange

Even shorter "tight urban diamond" interchanges are found in most large cities in the United States. They are the rule in Texas because of their routine use of continuous one-way frontage roads in tight urban freeway design. Ramp spacings of tight urban diamonds usually range from 250 to 350 ft with known cases from 150 to 400 ft. A tight urban diamond desirably has one continuous left-turn lane per direction between the signals. In some low-volume cases, no separate left-turn lanes are provided to minimize cost and yet satisfy the traffic demand. Whereas tight urban diamond interchanges are usually located on the state highway system, their operations are routinely handled by the local city or county traffic engineering departments in most states and not by state traffic planners. Considerable operational feedback and insight can be easily missed by traffic planners regarding the true capabilities and best applications for the tight urban diamond interchange.

It may appear ironic that tight urban diamond interchanges having spacings of 250 to 350 ft can operate better than wider compressed diamonds with spacings between signals of 500 to 600 ft. This reality is believed to be not well known by some highway engineers and planners. Two requirements must be met to achieve this level of operational performance, however. One is that the ramp spacing should be in the range of 250 to 400 ft. Second is that only one traffic-actuated signal controller should be used and it must be designed and timed properly to best satisfy existing traffic conditions. With these design specifications, the tight urban diamond interchange is a viable alternative to all other interchange forms in the two-level signalized urban interchange class.

In the remainder of this report, the "Tight Urban Diamond Interchange" is called a "TUDI" for short. The SPUI and TUDI should both be considered viable design options for many types of urban traffic congestion relief projects where signalized intersections are involved.

CHAPTER TWO

GEOMETRIC CHARACTERISTICS

The following is a summary of the various geometric design attributes of 36 operational, single point urban interchanges (SPUIs) found in the United States during the summer of 1989. Data forming the basis for these attributes were obtained primarily from two sources: (1) construction plans provided by local state highway offices, and (2) observations and measurements made during the field survey portion of this project. These collective attributes have been tabulated and categorized into various subgroups as a means of focusing on the individual geometric characteristics that comprise the SPUI.

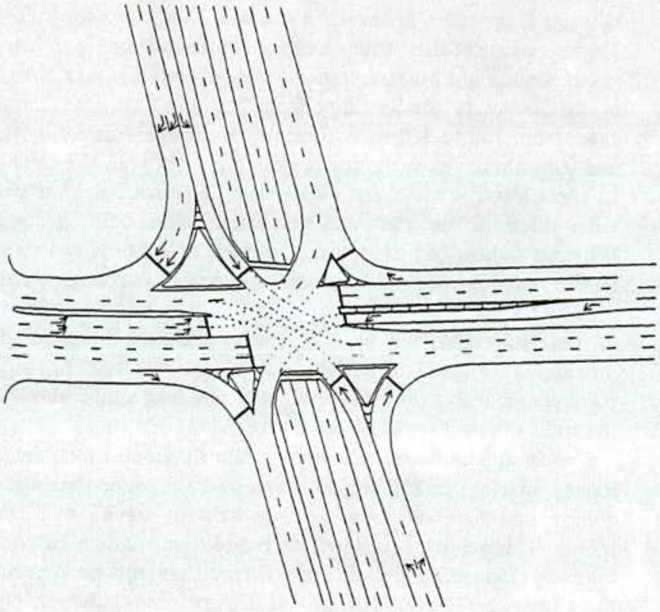
In the first section, the general features of the intersecting roadway systems are described. The thrust of this section is that there are two basic types of SPUI—those with the major road elevated over the ramp/cross-road intersection, i.e., a mainline overpass SPUI, and those with the major road depressed under the intersection area, i.e., an underpass SPUI. This fundamental difference tends to impose different design requirements and, as a result, will be discussed separately in the remainder of this chapter.

The second section focuses on the characteristics of the intersecting roadways. Specific characteristics include the cross-sectional components of the major and minor intersecting roadways. One outcome of this examination is a sense of the SPUI's right-of-way use.

The third section describes specific geometric characteristics of the SPUI. Particular attention is paid to the turning movements at SPUIs such as the number of lanes provided, the radii of the turn paths, and some operational features of these turning movements.

GENERAL FEATURES

This section describes several fundamental system characteristics of the interchange that often have a significant influence on its design and operation. These general characteristics include grade separation type, existence of frontage roads, and interchange skew in horizontal alignment. Each of these basic features is discussed more fully in the following paragraphs.



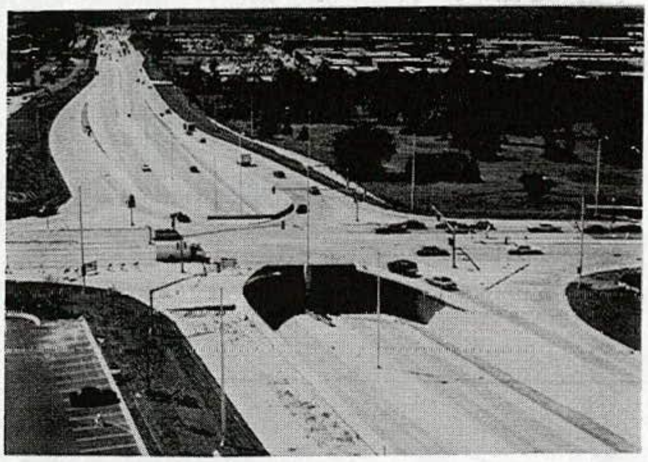


Figure 3. Underpass single-point urban interchange.

Grade Separation Type

As mentioned previously, SPUIs can be described as being one of two types with respect to grade separation of the intersecting roadways. One type of SPUI has the major roadway through movements passing above the ramp/cross-road intersection. This is termed an "overpass SPUI" or a SPUI with an overpass design as shown in Figures 1 and 2. In contrast, those SPUIs that have the major roadway through movement passing under the ramp/cross-road intersection are called "underpass SPUIs." An underpass SPUI is shown in Figure 3. There are major differences in many aspects of SPUI design based on this one attribute. Bridge design, in particular, is significantly different between the overpass and underpass interchanges.

On the basis of the survey of 36 SPUIs, the overpass design was found to be the more common type, outnumbering the underpass design by a ratio of 3 to 1, i.e., 27 overpasses and 9 underpasses. Moreover, most overpass SPUIs were found to have an elevated major road and an at-grade cross road. A review of the 36 SPUIs studied indicates that 25 of the 27 overpass SPUIs have this "fly-over" type design. Reasons for elevating the major road include less disruption to existing property and underground utilities, and simpler structural design. It should be noted that the vertical profile of the minor cross road remained essentially unchanged at over 90 percent of all SPUIs observed, thereby minimizing impacts to the adjacent property and local street system. The principal design trade-off for overpass designs is between bridge length and resulting horizontal curve sight distance for left turning lanes. Reasonably consistent turning speeds appropriate for the site should be maintained throughout the turning maneuver.

Although the flyover overpass SPUI is used more frequently, the major road at-grade overpass SPUI was used at two locations. Reasons for depressing the cross road may be based on topography, economics, aesthetics, or other reasons. For example, one SPUI is located on the top of a hill with relatively steep grades on all mainline approaches. Rather than increase the grade of the major roadway, the decision was made to lower the cross-road alignment. Field observations indicate that intersection visibility problems may arise if the design speed of the cross road is not maintained at a reasonably high level.

The underpass SPUI usually has the major road depressed and the cross road remaining at-grade. Of the 9 underpass designs observed, 7 basically retained the original grade line for the cross road and depressed the major road. This type of interchange design has the advantages of being removed from sight, attenuating traffic noise on the major road, and making ramp grades consistent with motorist acceleration-deceleration needs. In addition, visibility of the intersection conflict area is generally superior to the overpass design when the cross road is not on a crest vertical curve. Removal of excavation material, stormwater drainage, and utility relocation are apparent disadvantages of the underpass design. Moreover, they can not be easily modified once in place. Underpass designs are more likely to be found with depressed freeways or in the eastern United States where undulating terrain exists and many major crossing streets follow the natural crown lines between local drainage basins.

A combination of site-specific constraints may require that the major road remain at-grade and the ramp/cross-road intersection be elevated above the major roadway, i.e., a cross-road flyover. Two underpass SPUIs with a cross-road flyover were found to have railroad tracks parallel to the major road. The railroad tracks probably influenced the decision to build the flyover. The cross-road flyover design has the advantage of avoiding drainage problems associated with a depressed design. Some disadvantages are that the flyover requires a relatively long and complex bridge because of the need to elevate the on-and-off ramps. The length stems from the need to provide intersection sight distance as opposed to stopping sight distance along the cross road. This complex design is further exacerbated by the transverse ramp connections that produce significant horizontal dynamic loadings on the bridge. The type of grade separation has a direct influence on almost every major design element of the SPUI. This influence is particularly obvious in the SPUI's structural features such as bridge length, depth, number of spans, and abutment type. With an overpass design, for example, a single-span bridge would probably have a span length of about 220 ft and a depth of 9 ft; whereas, for the underpass design, the bridge would likely have two spans of about 70 ft and a depth of 3 ft. The type of structure may also influence the SPUI's ramp geometry, U-turn design, right-of-way requirements, turn path geometry, signal head placement, and lighting design.

Frontage Road Systems

Two types of frontage road systems were observed in the field survey: the combined frontage road and the offset frontage road. Both of these are shown in Figure 4. The combined frontage road design occurs where the ramps merge with one-way frontage roads prior to their intersection with the cross road. The offset frontage road design has two-way frontage roads laterally offset from the on-and-off ramp junctions to provide nominal separation along the cross road. Figure 5 shows the distribution of frontage roads found at the 36 SPUIs studied. As indicated, 11 SPUIs were found to have frontage roads—all of which had the overpass design. Of these 11 SPUIs, 8 were found to have combined frontage roads while 3 had offset frontage roads.

The combined frontage road systems are located in three southeastern states where high volume urban arterials were being upgraded to expressway standards along a considerable length of the facility. This design appears to be applicable to narrow right-of-way situations likely to be found in urban arterial upgrades. As shown in Figure 4, the basic SPUI design is modified slightly with this option to serve the frontage road through and mainline turnarounds. Multispan (generally three-span) bridges are used to provide for the turnarounds.

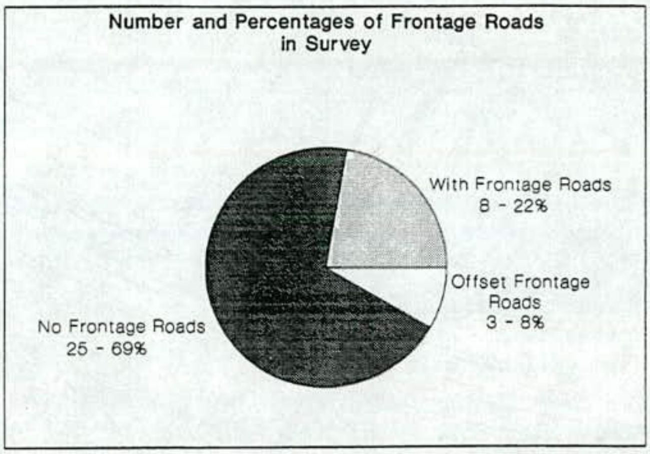


Figure 5. Distribution of frontage roads at 36 SPUIs in survey.

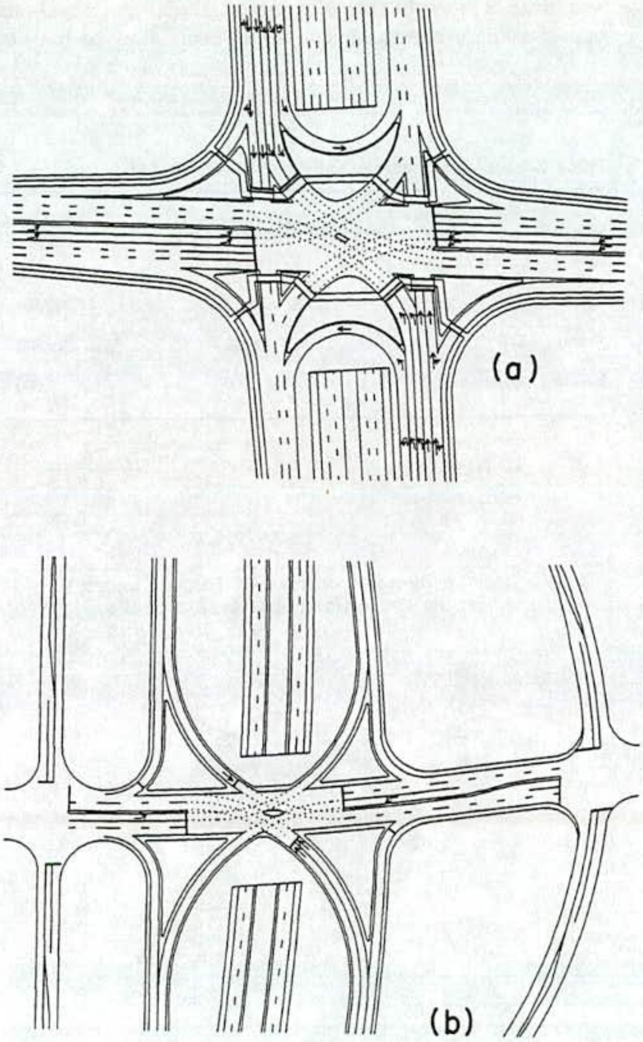


Figure 4. Frontage road treatments: (a) combined one-way frontage roads; (b) offset two-way frontage roads.

Alignment Skew

The intersection of two roadway alignments generally does not occur at a 90-deg angle. This deviation from 90 deg is called the alignment skew. Skew generally has an adverse effect on SPUIs because it increases clearance distances, decreases clearance speeds, and adversely affects sight distance by making it more difficult for some off-ramp drivers to see along the cross road. Severe skew in alignments may also increase the length of the bridge and widen the distance between cross-road stop lines, i.e., increase clearance distance and lost time.

The skew angle is defined in Figure 6 as the rotation of the cross road relative to the major roadway, with a clockwise rotation of the cross road from normal indicating a positive skew angle. A positive skew can make the off-ramp right-turn and the cross-road left-turn movements more difficult, because it often results in a smaller radius turn path and a more acute angle of entry with the intersecting roadway. A negative skew can make the off-ramp left-turn and cross-road right-turn movements more difficult. In some fortunate cases, however, a designer may be able to use a small skew to favor a major traffic movement at no expense to the minor movements.

The skew angles found at each of the 36 SPUIs studied are shown in Figure 6. Skew angles at these SPUIs were found to range from −30 deg to +28 deg. The distribution of skew angles suggests that there is no trend toward smaller skew angles as might be expected. The likelihood of finding any particular skew angle appears to be about equal to that of any other within the range observed.

ROADWAY CHARACTERISTICS

This section describes the general character of the roadways that intersect at the SPUI. Specific roadway characteristics in-

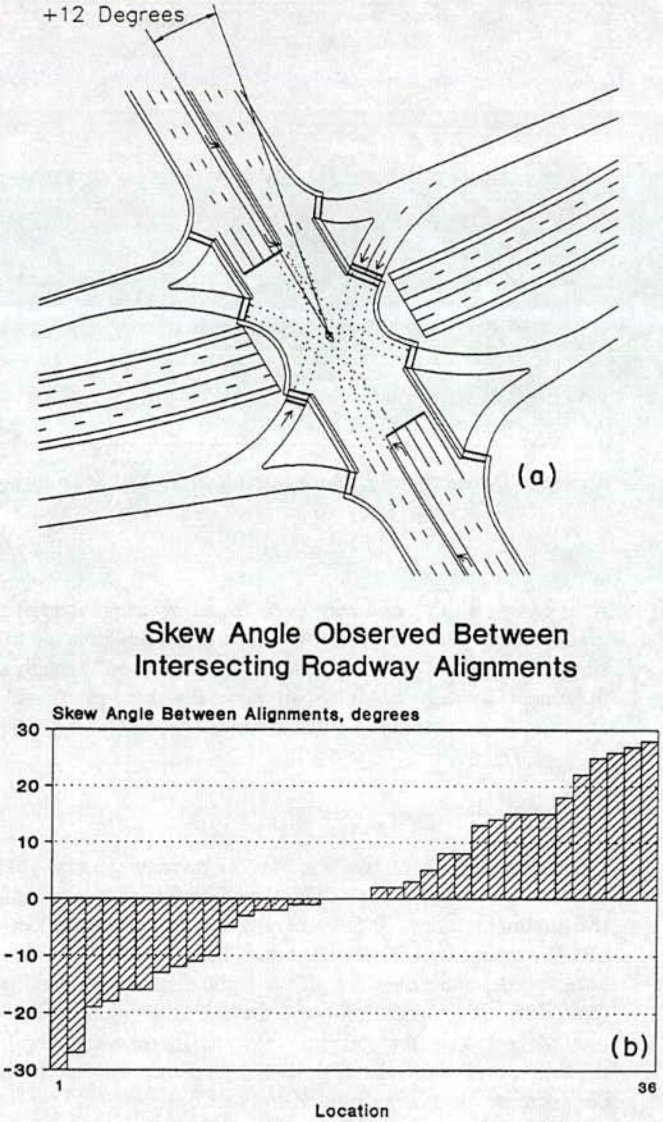


Figure 6. Skew angle observed between intersecting roadway alignments.

clude the cross sectional width, the number of through lanes, and the median width of the major and crossing roadways. The average daily traffic on the cross road is also included as an indicator of the range of traffic demands that the SPUI can serve. These characteristics have been selected because of their relevance to the SPUI design and their significant influence on the selection of critical design elements.

All characteristics presented in this section will be categorized by grade separation type, i.e., overpass or underpass design. The justification for this treatment relates to the inherent differences in the design and operation of these two design types. These differences are best illustrated by comparing the distributions of individual roadway design features collected during the field survey using both graphical and statistical measures. In some instances, the characteristic distribution of individual roadway elements for the two designs do not have the same physical design applications; however, their distributions will be included in this section for purposes of documentation, comparison, and contrast.

Cross Road Function

The cross-road average daily traffic recorded at 15 SPUIs is given in Table 1. These volumes suggest that the SPUI is adaptable to a wide range of cross-road traffic demands. The higher volumes generally exceed cross-road volumes found at high-type at-grade signalized intersections along major urban arterials.

To adequately serve cross-road traffic demands, SPUIs are usually designed with four or six through lanes on the cross road. The distributions of the number of through lanes found on the crossing arterial at the 36 SPUIs are shown in Figures 7 and 8. As these figures indicate, about 70 percent of the cross roads have four through lanes and the rest generally have six lanes. There does not appear to be a significant difference between the overpass and underpass designs in terms of the number of through lanes on the cross road.

Observations made during the field study, combined with data on daily traffic demands, were used to assess the functional classification of the cross road in the vicinity of the interchange. In general, the range of traffic volumes shown in Table 1 are consistent with the range of cross road demands found on urban expressways and major arterials. Observation of operating speeds and adjacent land access indicates that the cross road function was more frequently that of a major arterial and occasionally that of a minor arterial, but never an expressway or freeway.

Table 1. Cross road average daily traffic comparisons.

	Overpass	Underpass
Number of SPUI's	11	4
Range of ADT	9,000-52,000	28,000-30,000
Average ADT	30,500	29,500
Standard Deviation	13,600	1,000

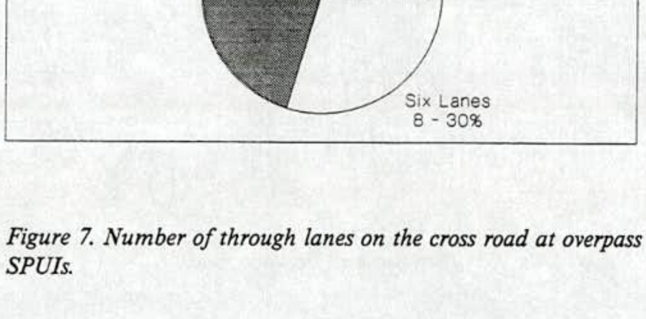


Figure 7. Number of through lanes on the cross road at overpass SPUIs.

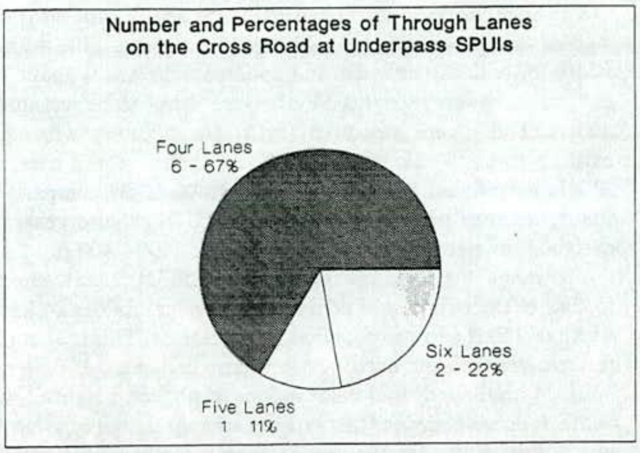


Figure 8. Number of through lanes on the cross road at underpass SPUIs.

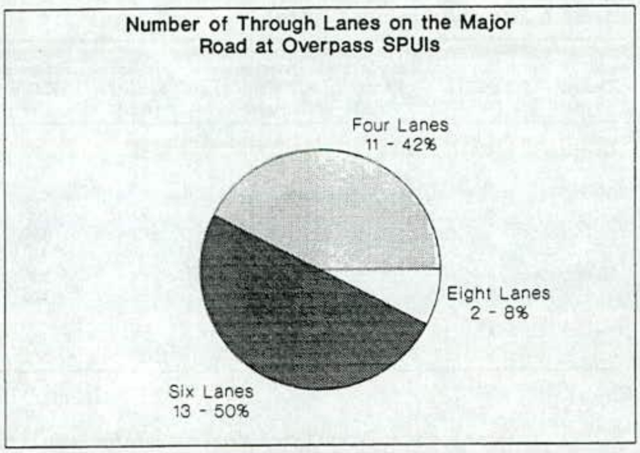


Figure 9. Number of through lanes on the major road at overpass SPUIs.

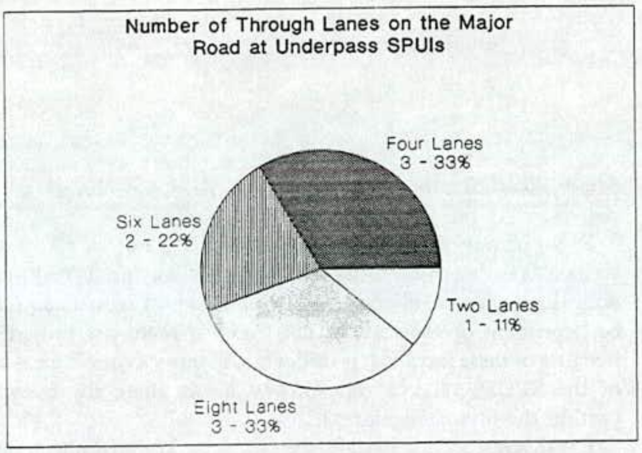


Figure 10. Number of through lanes on the major road at underpass SPUIs.

Major Road Function

The major road classification was determined using a slightly different approach from that used for the cross road. In addition to traffic demands, route numbering classification and basic number of lanes were used to assess the likely functional character of the major road. The classification of SPUIs by route numbering indicated that about 25 percent have major roads on the Interstate System, 25 percent on the U.S. Highway System, and 50 percent on the State Highway System.

Figures 9 and 10 illustrate the distribution of the number of through lanes on the major routes. As shown in these figures, the distributions are different for the underpass and overpass designs. In particular, it was found that 50 percent of the overpass SPUIs studied have six-lane major roads while about 40 percent have only four lanes. A few have eight lanes. In contrast, the underpass SPUI had major road lane allocations more evenly split among four, six, and eight lanes.

At those SPUIs being built to upgrade existing major arterial streets to urban expressway standards, major road overpasses were used in all cases. In each case, one-way frontage roads were first provided along the major artery to serve through traffic during construction and remained following completion of the project. U-turn lanes were provided to improve access and to gain public support for the project.

Right-of-Way Use

One of the reported benefits of the SPUI design is its minimal right-of-way needs when compared to conventional diamond interchanges. Because right-of-way takings at an interchange are influenced by many factors, e.g., cross section and boundaries of adjacent property, it would be neither practical nor prudent to use the actual right-of-way at specific SPUIs to make accurate assessments of the SPUI's nominal right-of-way needs. A better means of assessing the SPUI's right-of-way use would be to use a combination of the width of the cross road and the distance between the on-and-off ramp pairs. These two measures will describe the SPUI's nominal right-of-way requirement along the cross road and major road, respectively, in the vicinity of the interchange.

Cross Road Right-of-Way

In general, the right-of-way along an urban arterial includes the width of traveled way plus a 10-ft border area on each side of the roadway. This border area typically includes 2.5 ft for curb and gutter, 2.5 ft for utility poles, and 5 ft for a sidewalk. Thus, the nominal cross road right-of-way at a SPUI should measure about 15 ft (= 7.5 + 7.5) more than the cross road's back-of-curb to back-of-curb width. The back-of-curb widths and estimated cross road rights-of-way for 35 SPUIs have been categorized according to SPUI design type and cross section; the results are given in Table 2. Protected left-turn lanes are generally required on the arterial for safe and efficient SPUI operations. As indicated in Table 2, the cross road right-of-way at those SPUIs surveyed is generally consistent with the desirable right-of-way suggested by the Institute of Traffic Engineers (*15*) for similar urban arterials. Moreover, it would appear that the basic cross section within the SPUI has not been narrowed by the designer to minimize bridge expanse.

Table 2. Estimated cross road right-of-way.

SPUI Design Type	Thru Lanes (No.)	Left-Turn Lanes (No.)	Average[1] Width (feet)	Basic[2] Section (feet)	Estimated[3] R.O.W. (feet)
Overpass	4	1	77	69	92
		2	97	81	112
	6	1	91	93	106
		2	123	105	138
Underpass	4	1	89	69	104
		2	105	81	120
	6	1	na	93	na
		2	126	105	141

Notes:

1 - Average cross road cross section widths measured from curb to curb for the 36 SPUI's studied.

2 - Basic section widths calculated using 12-foot lanes, 2.5-foot curb and gutter, and a 4-foot median.

3 - Estimated right-of-way calculated by adding 15 feet to the average width.

na - Data not available.

Major Road Right-of-Way

Right-of-way use along the major road is not as easily described as is that of the cross road. Right-of-way needs may vary with the horizontal alignment of the on-and-off ramps and may be dependent on whether or not frontage roads are provided. Because of these factors, it is difficult to identify a simple measure of the SPUI's relative right-of-way needs along the freeway (within the interchange area).

For this research, a method was established for making quantitative assessments of the right-of-way needs of a SPUI. The objective of the method is to make comparative assessments among SPUIs and not to absolutely define the SPUI's actual right-of-way taking. This method is also general enough to be applicable to conventional diamond interchanges and, thereby, facilitates the comparison of different interchange design types, if so desired.

For diamond-type interchanges (such as the SPUI) the ramps are generally at their maximum separation just prior to their intersection with the cross road. The method used to estimate right-of-way measures the maximum separation distance between the SPUI's on-and-off ramps. This distance is measured between the off-ramp back-of-curb and the on-ramp back-of-curb for each ramp pair, perpendicular to the major road centerline. Four points are established in this procedure, one for each ramp, as shown in Figure 11. Any one point is found by visually offsetting the major road centerline to the most distant point on the inside curb of the ramp left-turn radius. Once the four points are located, a line is drawn through each point perpendicular to the major road centerline. Each line is then extended in both directions to the back-of-curb outside of each on-and-off ramp pair. The distance along each line is then measured between the ramp back-of-curbs with the largest distance found being defined as the maximum ramp separation. This width represents a conservative estimate of the nominal right-of-way needed by the SPUI in the vicinity of the cross road.

The mainline ramp-to-ramp widths measured for the 36 SPUIs studied are shown in Figures 12 and 13. In general, the average width for both the overpass and underpass designs is about 300 ft. The narrowest overpass SPUIs were found to be just under 200 ft. These were measured for a state highway where the existing right-of-way was 200 ft. In contrast, three overpass SPUIs were found to have widths over 400 ft. By comparison, the tight urban diamond interchange (TUDI) is also generally designed for rights-of-way in the range of 250 to 400 ft.

The range of widths for the underpass SPUI is quite similar to that of the overpass. The narrowest underpass design had a width of 160 ft and was found at two locations. This is about 30 ft narrower than the narrowest overpass design.

Based on these overall distributions of projected right-of-way width, it does not appear that one interchange design type results in a consistently narrower design than the other. Moreover, it appears that 90 percent of the SPUIs are constructed in rights-of-way that would also be adequate for a TUDI. This suggests that many SPUI designs may have been selected for providing adequate capacity and a desirable operating environment rather than to predominately minimize right-of-way.

Medians

All of 36 SPUIs surveyed were found to have some type of median separation on the major roadway. In contrast, only about 85 percent of the SPUIs have medians on the cross road. The field survey also indicated that about 75 percent of the overpass SPUIs with cross road median treatments had raised medians while only 25 percent of the underpass SPUIs with medians had raised medians. The type of median along the major road varied from narrow and flush with a concrete barrier to wide with a depressed, traversable median.

The distributions of cross road median widths for 33 SPUIs are shown in Figures 14 and 15. As these figures indicate, the average median width found for both overpass and underpass SPUIs was about 6 ft, as measured at the median nose. The most frequently found median width for both designs was 4 ft.

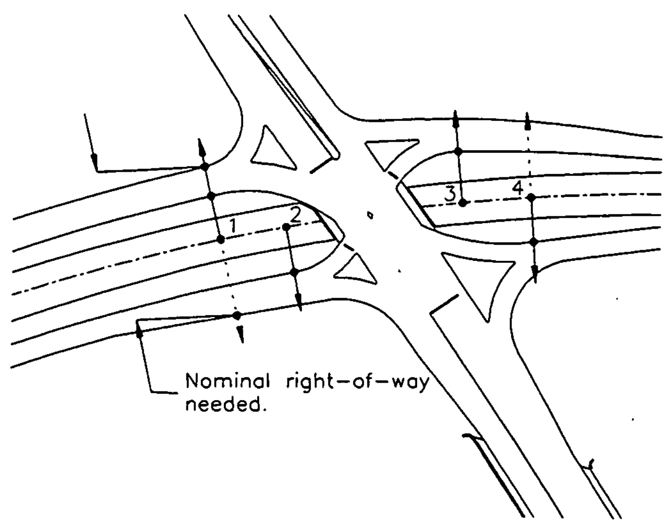


Figure 11. Major road right-of-way measurement.

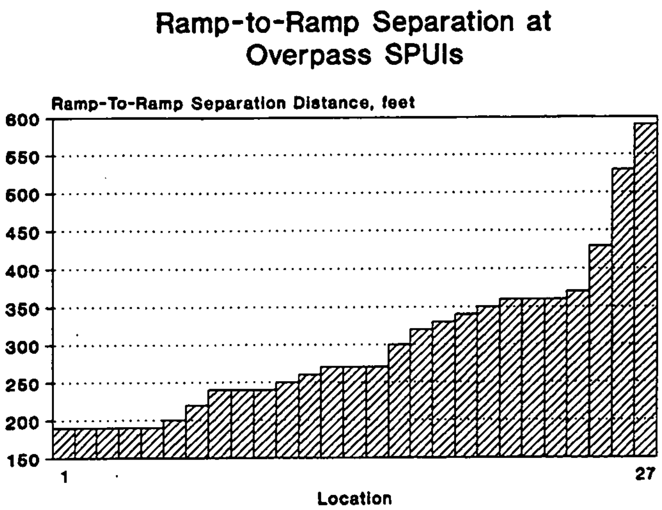


Figure 12. Ramp-to-ramp separation at overpass SPUIs.

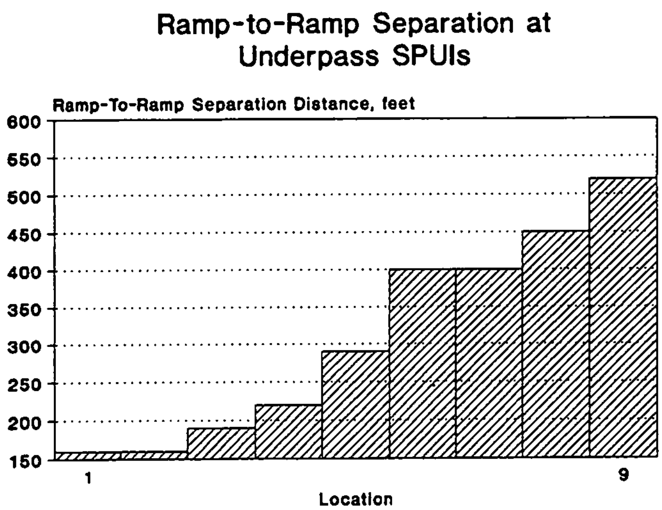


Figure 13. Ramp-to-ramp separation at underpass SPUIs.

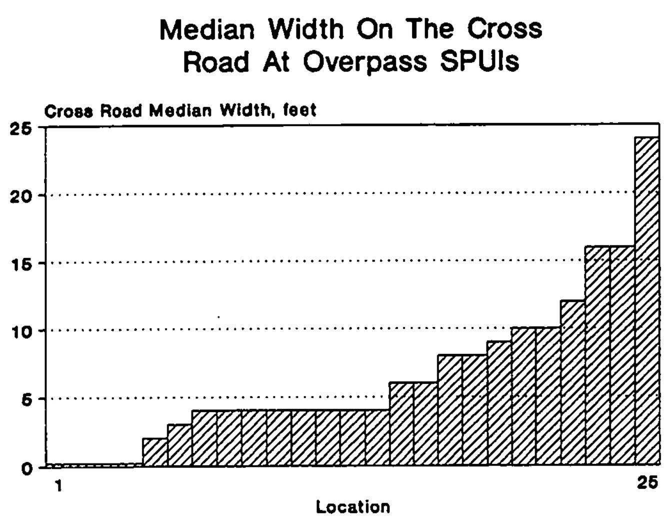


Figure 14. Median width on the cross road at overpass SPUIs.

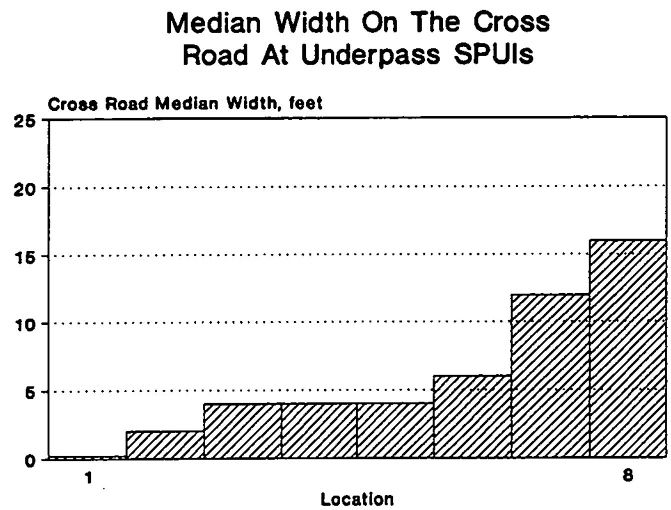


Figure 15. Median width on the cross road at underpass SPUIs.

Figures 16 and 17 illustrate the distributions of median widths for the major road. In contrast to the similarities noted in cross road median width, there are some differences in the average major road median width for overpass and underpass designs. The average overpass median width measured 24 ft compared to an average of 18 ft for the underpass design. Some of this difference can be attributed to one overpass SPUI with an extraordinarily wide median. If this median is removed from the sample, the average of the remaining overpass medians would be about 20 ft, which is closer to the underpass median width of 18 ft.

INTERCHANGE CHARACTERISTICS

The following sections describe the characteristics of selected geometric elements of the SPUI. These elements were selected because of their unusual dimension or treatment when applied to the SPUI design. They include grade separation type, left-turn radii, U-turn lanes, channelization, and bridge attributes.

Grade Separation Type

As noted previously, the type of grade separation used at a SPUI has a major effect on many design elements within the interchange. The structural requirements of either design type often place significant constraints on the dimensions that can be considered for other design elements. For example, the complexity and expanse of the structure require very specific bridge bent locations and, as a result, can pose severe constraints on the geometric design of the left-turn paths. Discussions with engineers having SPUI design experience indicate that design alternatives that minimize the structural size, complexity, or cost are usually given first priority. However, they also recognize that efficient traffic operations depend on the adequacy of the geometric design, particularly as it relates to the left-turn movements. Again, a critical design decision is the selection of the design speed profile and resulting left-turn radius for each of the on-and-off ramp left turns.

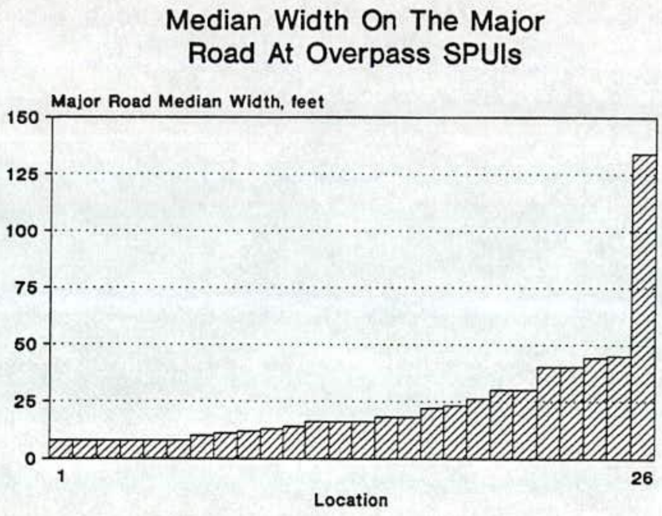


Figure 16. Median width on the major road at overpass SPUIs.

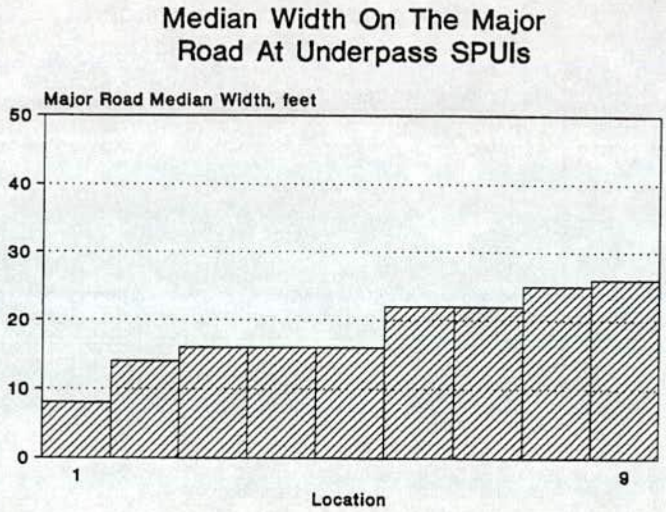


Figure 17. Median width on the major road at underpass SPUIs.

Turning Lanes

Of the ten traffic movements associated with the SPUI design, four are right-turn movements, four are left-turn movements, and two are through movements. If the SPUI is within a frontage road system and if U-turn lanes are provided, then four additional traffic movements exist. Obviously, the operational performance of the SPUI is highly dependent on the geometric design of the turning roadway associated with each of these turning movements.

Important design elements of a turning roadway include radius, superelevation, and width. Superelevating the left-turning roadways within the central intersection conflict area is not practical because of the common use of the pavement by other traffic movements. The lack of any superelevation is partially offset by the larger radii associated with the SPUI design. The large turning radii appear well suited for efficient truck traffic and dual-lane operation. The following sections describe these design elements as applied at existing SPUIs by type of turning operation.

Right-Turn Lanes

The operational efficiency of right turns at SPUIs depends on whether they are made from the crossing arterial or from the off ramps. Right turns from the cross road are more natural and have higher capacity per lane because of the three-phase signalization commonly used, as will be described later. Right turns from the off ramps are operationally more complex and typically have less capacity per lane.

Right turns from the cross road are operationally and physically similar to those found at conventional diamond interchanges. A common right-turn treatment on the cross road is to have right-turn traffic share the curb lane with through traffic up to a point 50 to 150 ft upstream of the stop line. At this point, the right-turn curve radii (or auxiliary lane taper) begins and right-turning traffic diverges from the shared lane. This type of treatment, shown in Figure 18, was found to occur at about one-half of the SPUIs studied; the other half have exclusive right-turn lanes.

An operational problem, cross street right turns may encounter, may occur when opposing left turning vehicles departing under signal control merge into the on-ramp turning lanes at relatively high speed made possible by the large left turning radius. Vehicle lane off-tracking may be a problem. Overpass designs make this merging operation more difficult, as compared with the normal tight urban diamond interchange (TUDI).

At about 80 percent of the SPUIs surveyed, the off-ramp right-turn traffic was provided a single exclusive lane on the ramp at its junction with the cross road, as shown in Figure 19. The other 20 percent of SPUIs provided dual exclusive lanes for the right-turn movement. Of those SPUIs with single lanes for the right-turn movement, 75 percent were designed to have right-turn traffic merge directly into the curb lane on the cross road, while 25 percent were designed with a separate acceleration lane along the cross road. The length of the exclusive off-ramp right-turn lane was found to vary from 50 to several hundred feet.

Observation of traffic operations on the off ramps indicates a strong correlation between ramp efficiency and the length of

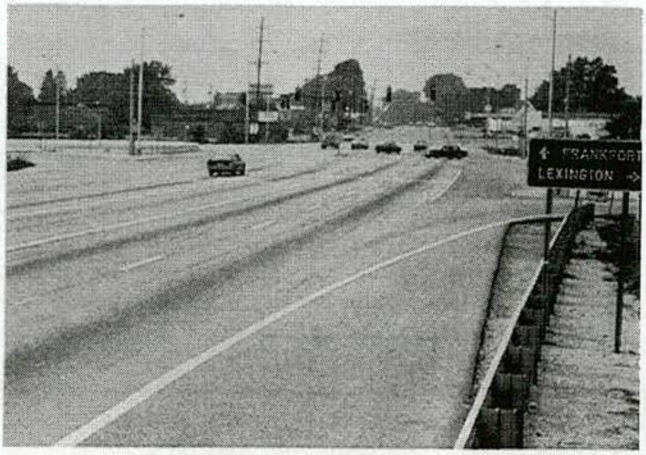


Figure 18. Cross-road right-turn lane.

Figure 19. Off-ramp right-turn lane.

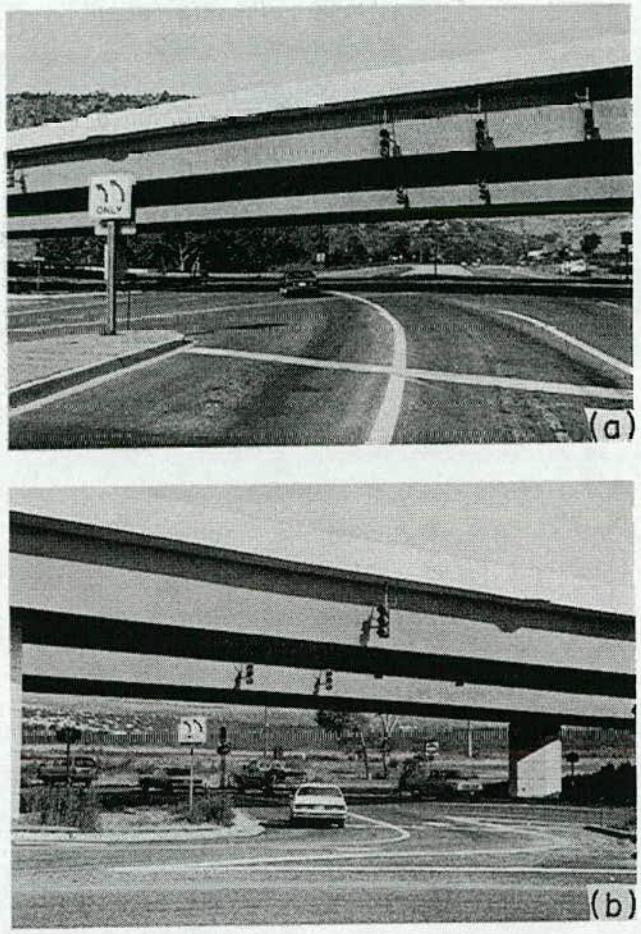


Figure 20. Left-turn lane treatments at an overpass SPUI: (a) cross-road left-turn lanes; (b) off-ramp left-turn lanes.

separation provided between left- and-right-turning vehicles. This relationship is due to the distinctly different types of traffic control regulating the left- and right-turn movements, i.e., traffic signal vs. YIELD sign, and the times during the signal cycle when each movement enters the cross road. Of the three SPUI signal phases (the cross road left turn, cross road through and off ramp left turn), the best opportunity for off-ramp right-turn entry is during the cross road left turn phase. The yielding right turn is generally blocked by departing platoons during the other two phases. Blockage of right-turn entry during the off-ramp left-turn phase was observed to create operational problems at some small SPUIs. During this phase, blocked off-ramp right-turn vehicles would queue up and impede the left-turn vehicles sharing the off-ramp from departing. This problem was more evident at those sites wherein the left-turn and right-turn vehicles did not have exclusive lanes of sufficient length to physically separate the two movements. The off-ramp right-turn vehicles were similarly impeded by stopped off-ramp left-turn vehicles during the cross road, left turn phase. The cross road, right turn movement does not experience this impedance as much because they move simultaneously with the through movement during the thorough-movement phase.

Left-Turn Lanes

One desirable geometric feature of the SPUI is its naturally large radii for ramp and cross-road left-turn paths. These long radii are generally two to three times larger in magnitude than similar radii found at at-grade intersections. Large radius curves appear to have several operational advantages, including higher turning speeds, higher saturation flow rates, and reduced off-tracking of lengthy vehicles (*11*). Typical left-turn lane treatments at a SPUI are shown in Figure 20.

Radius of Left-Turn Lanes. The longer left-turn radius at SPUIs provides increased design speed of the turn; conversely, a higher design speed is likely. For a given design speed and required sight distance, reducing the clear span distance between the bridge supports requires increasing the left-turn radius to flatten the curve with increasing lateral separation between the ramp and the major roadway.

The distribution of cross road left turn radii is shown in Figures 21 and 22 for the overpass and underpass designs, respectively. These radii represent the actual radius of the curvature of the inner edge of the inside of the turn lane. In those instances where three-centered compound curves are used or where geometric design information was unavailable, an average radius was determined graphically.

Figures 21 and 22 indicate that the average left-turn radii for both the overpass and the underpass design is about 200 ft on the cross road. The difference in design types becomes apparent when considering the variability in radii—the overpass has a much wider range of radii. This variability is likely due to a greater flexibility in the design of left-turn paths at overpass SPUIs because of fewer physical constraints imposed by the bridge structure.

Three-centered compound left-turn curvature was found at about 30 percent of the SPUIs surveyed. At these locations, the curvature along the turn path transitions from a small radius to a large radius and then back to a small radius. This approach has the advantage of being better able to "fit" the left-turn path between the constraining bridge supports. On the other hand, this design is contrary to traditional horizontal curve design which transitions from tangent, i.e., infinitely large radius, to a relatively small radius and then back to tangent. This large-small-large design transition is normally accomplished by using

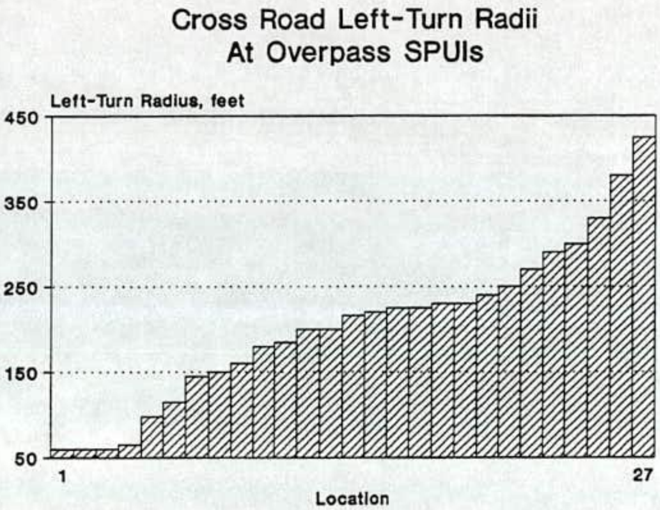


Figure 21. Cross-road left-turn radii at overpass SPUIs.

3-centered curves at intersections and by using spirals on high-speed roadways. Because the SPUI's small-large-small radius transition is not consistent with the design of other horizontal curves, it is not likely to be consistent with driver expectancy.

At a few SPUIs observed in the field survey, the compound curvature was so extreme that the curve was actually a broken-back curve. For these SPUIs, it was impossible to find an average radius that would be tangent at the beginning and end of the curve and yet not conflict with the opposing left-turn path. Under these circumstances, the smaller of the entry and exit radii was recorded instead of an average radius.

Those radii shown in Figure 23 that are smaller than 100 ft represent SPUIs that have broken-back left-turn paths. This situation represents the extreme application of the 3-centered compound curve treatment wherein the central radius is infinitely large. The result is a short tangent section on the opposing "turning" vehicles pass to the *right* of one another. This broken-back design violates driver expectancy in that oncoming vehicles normally meet one another on their left sides and, as a result, may give drivers the feeling that they are driving on the wrong side of the road.

Figures 23 and 24 also illustrate the distribution of radii for the off-ramp left-turn movements. As these figures show, the average off-ramp radius for the overpass is 237 ft while the corresponding average radius for the underpass is nearly 300 ft. The distribution of radii for the overpass design is noticeably skewed by one SPUI with a radius of 1,000 ft. If this "outlier" is removed, the average radius for the overpass design would be about 210 ft. This value is only slightly larger than the average cross-road left-turn radius of 204 ft (see Figure 21).

The off-ramp radii is slightly larger than the cross-road radii for both the underpass and overpass designs. One explanation for this is based on the physical constraints imposed on each type of turn. The cross-road left-turn path has its outside edge, tangent-to-curve point (PC), near the centerline. In contrast, the off-ramp path has its outside edge, curve-to-tangent point (PT),

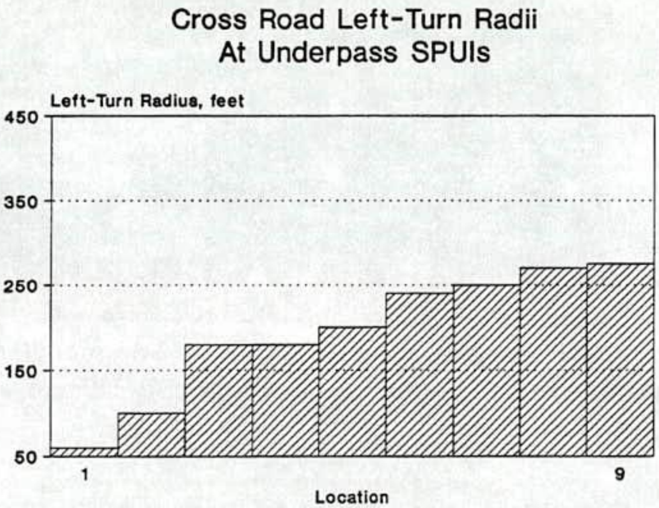


Figure 22. Cross-road left-turn radii at underpass SPUIs.

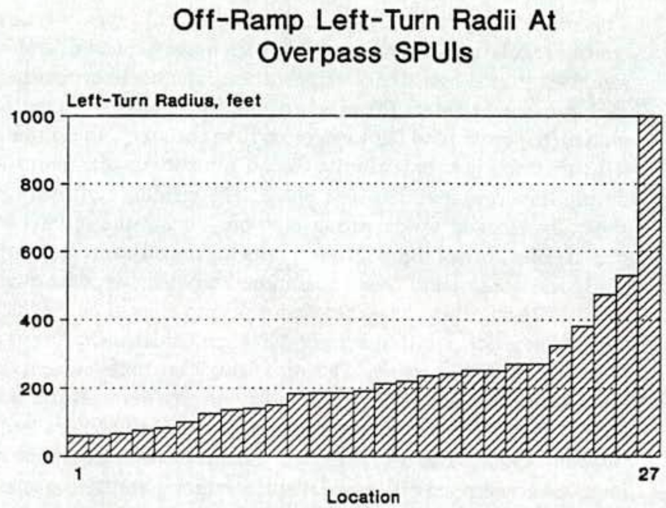


Figure 23. Off-ramp left-turn radii at overpass SPUIs.

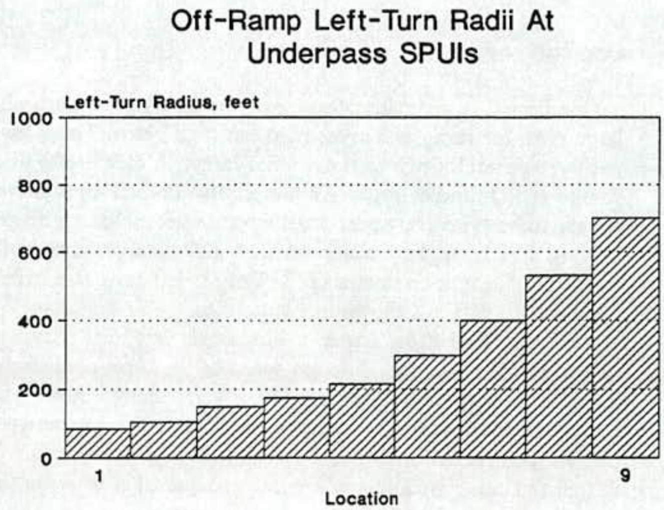


Figure 24. Off-ramp left-turn radii at underpass SPUIs.

on the far side of the cross road, one or two lane widths beyond the median. As a result, the off-ramp path must be physically larger than the cross-road path, thereby necessitating a slightly larger radii to fit the desired turning path.

Number of Left-Turn Lanes. Discussions with engineers during the field survey indicated that traditional methods were used to determine the number of left-turn lanes at a SPUI. The methods used in determining the number of lanes were based primarily on forecasted traffic demands, although there is also a predisposition among traffic engineers to provide dual lanes at all high-type intersections and interchanges whenever possible. This attitude is even more applicable to the SPUI because of the difficulty in modifying its design elements once constructed.

Figures 25 and 26 illustrate the distribution in number of cross-road left-turn lanes at overpass and underpass SPUIs, respectively. The most common treatment for the overpass was the use of dual left-turn lanes. In contrast, the "single- and dual-lane" treatment, i.e., a combination of single-lane bay on one cross road approach and dual-lane bay on the other approach, was found most frequently at underpass SPUIs.

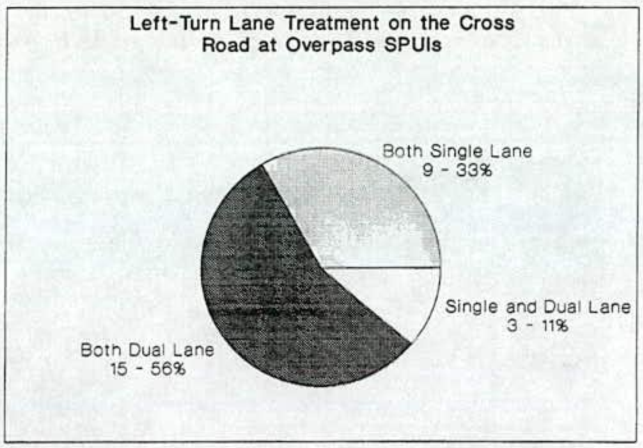


Figure 25. Left-turn lane treatment on the cross road at overpass SPUIs.

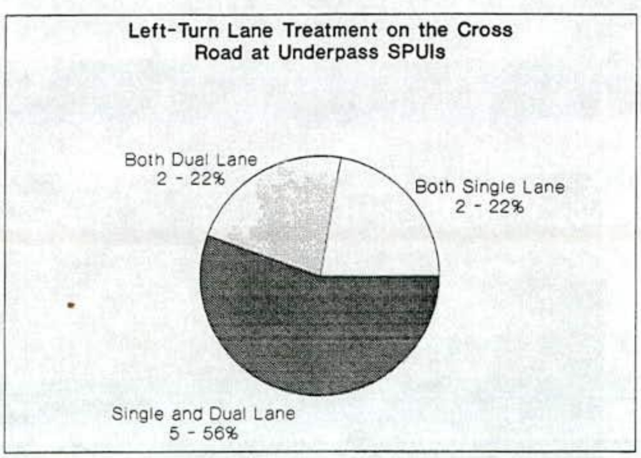


Figure 26. Left-turn lane treatment on the cross road at underpass SPUIs.

In most cases where both single and dual lanes are used in combination, provision in the cross road section is made for ultimate widening of the single-lane bay to dual lanes. However, at some underpass SPUIs it was noted that this provision was not extended to the throat of the on ramp. Given the complexity of the structural design of the on-and-off ramps for the underpass design, it is important that future widening of the ramp to dual-lane left-turn operation be considered early in the design process design whenever future widening is a possibility.

As shown in Figures 27 and 28, the off-ramp left-turn movement has dual lanes at two-thirds of the SPUIs studied (regardless of whether the SPUI is an overpass or underpass type). This proportion is also consistent with that found for the cross-road left-turn movement of the overpass SPUI (see Figure 25).

In summary, several design controls are noted. First, traffic operations at the SPUI require that single or dual left turn lanes be used on the cross street approaches prior to the intersection.

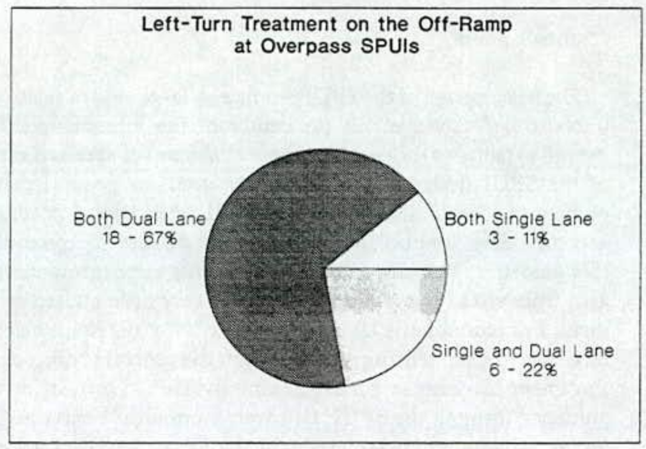


Figure 27. Left-turn treatment on the off ramp at overpass SPUIs.

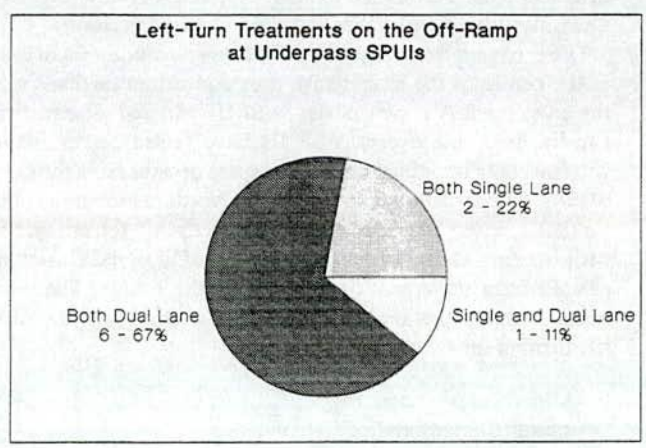


Figure 28. Left-turn treatment on the off ramp at underpass SPUIs.

Second, off-ramp left-turn lanes should be fully divided from the right-turn lanes to provide separate queue storage space. Third, provision of dual left turn lanes on all approaches is suggested by practice.

U-Turn Lanes

U-turn (or turnaround) lanes were observed at seven of the 36 SPUIs studied; all were overpass SPUIs. In each case, U-turn lanes were used at SPUIs where one-way frontage roads were present along the mainline facility. One benefit of U-turn lanes is improved access to businesses located along the frontage road. Another benefit is diversion of traffic from the SPUI's signal-controlled movements. The one-way frontage roads provide a convenient bypass detour during mainline reconstruction. In almost all cases, the U-turn lanes were conveniently accommodated by adopting a three-span bridge design with one U-turn lane being located under each of the two outer spans. A typical SPUI with U-turn lanes is shown in Figure 4(a).

Channelization

The basic design of the SPUI produces a large area of relatively uncontrolled pavement in the center of the interchange. The overall expanse of the central intersection conflict area and rarity of the SPUI design suggest a greater need for positive driver guidance and visibility. However, the SPUI design does not easily accommodate traditional traffic control devices for providing this guidance. Vehicles entering and exiting the central conflict area from virtually every direction leave very little unused pavement. For economic SPUI designs, few locations exist for placing large raised channelizing islands within the central conflict area. Pavement markings are most commonly used to provide driver guidance through the SPUI; however, the multiple entry angles and high volumes characteristic of the SPUI may lead to relatively quick obliteration of even the most durable marking materials.

SPUIs typically have four large refuge islands—one at each ramp and cross-road junction—outside of the central conflict area. Based on the field study data, the surface area of these refuge islands varies from 2,400 to 33,000 sq ft. In almost every instance, these islands have raised concrete curbing. Figure 29 illustrates island channelization used at two SPUIs.

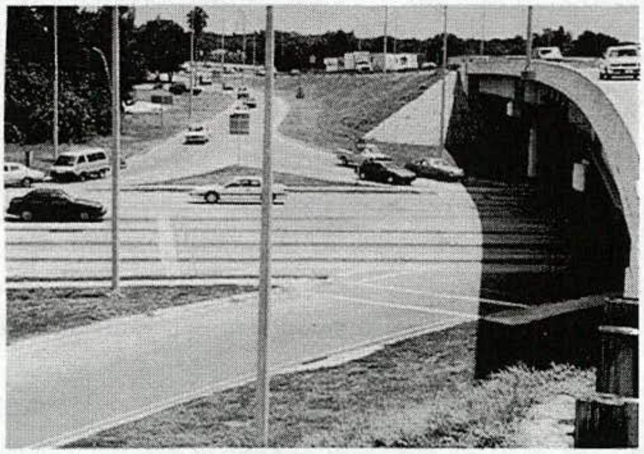
Other channelization used at SPUIs may include a small island at the center of the interchange area and raised medians along the cross road. A review of the 36 SPUIs studied indicated that two-thirds of the overpass SPUIs have raised center islands; whereas, only one-third of the underpass designs have the center islands. One explanation for this difference is based on the premise that a raised center island may be desirable for another use with overpasses—that of protecting a traffic signal cluster suspended from underneath the center of the bridge. The center island is also sometimes used to mount traffic signs that direct left turning movements.

Structural Characteristics

One of the more unique, and certainly more expensive, components of the SPUI is the bridge and its support system. Safe traffic operation requires the intersection conflict area to be clear of objects that might obstruct the driver's view of traffic conditions on the crossing street. This requirement is significant at the SPUI because it creates a relatively large open area directly above or below the bridge deck. For the overpass design, the bridge must be designed to span the conflict area. This requirement usually leads to a relatively long bridge with a clear span length of 200 ft to 240 ft. For the underpass design, the conflict area must be supported on a bridge deck which can be relatively wide, typically measuring about 100 ft wide by 150 ft long. The underpass design normally has the distinct advantage of having a median pier to support the center spans. For either design type, however, the atypical size and shape of the superstructure usually results in added design complexity and construction cost.

The structural characteristics recorded during the survey include overall bridge length, center span length, total number of spans, and cross-sectional shape and composition of the beams. Based on this tabulation, it was found that about 80 percent of the bridges used steel, while the remainder used a combination of precast and cast-in-place concrete. About 70 percent of the SPUIs used plant-fabricated I-beam plate girders and the other 30 percent used trapezoidal box girders.

Figure 30 illustrates the distribution of total span length found at the 22 overpass SPUIs surveyed for this research. As this



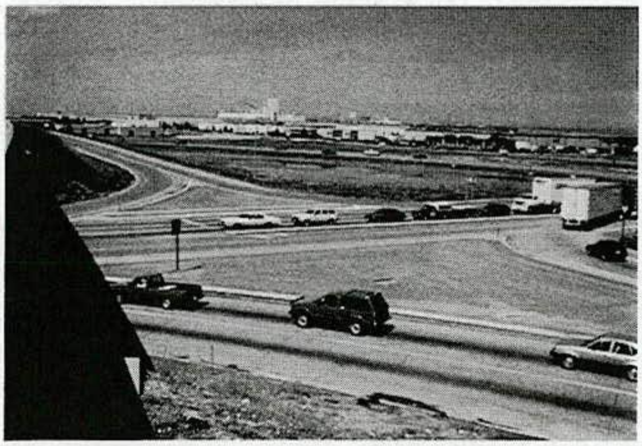


Figure 29. Typical island channelization treatments.

figure indicates, bridge lengths were found to range from 180 ft to nearly 600 ft, depending on the total number of spans of the bridge. The average length of the 7 single-span bridges was found to be 223 ft. By comparison, the average length of the 12 three-span bridges was 401 ft.

The average center span length of the three-span overpass bridges was 183 ft; 40 ft less than the 223-ft average for the single-span overpass bridges. This difference can be partially attributed to the 3 multispan bridges in one state that have span lengths of 160 ft or less. Because these three SPUIs also have broken-back left-turn paths, it would appear that the circular curvature of the left-turn paths was compromised to achieve the relatively short span lengths. As mentioned previously, circular turn paths are more nearly consistent with driver expectancy on intersection curves and should represent a desirable design goal.

The distribution of span lengths for the underpass SPUIs is shown in Figure 31. As indicated, the total bridge length for the underpass design ranges from 90 ft to nearly 200 ft. In most cases, the bridge has two spans and averages a total length of 143 ft. Those SPUIs with two spans used the median of the major roadway to locate the center bridge support and thereby reduce the bridge's clear span length to two sections of about 70 ft each. The reduction in span length was accompanied by a reduction in bridge depth; however, the cost of the median bridge bent may exceed the incremental cost of a single span bridge when reconstructing the bridge within an existing narrow freeway median under heavy traffic. Bridge designers will want all approach leg stopping to be done off the overpass bridge, if possible, to minimize transverse dynamic loading on the bridge and will not want traffic detector loops cut in the bridge deck to detect vehicle presence at the stop line.

In summary, both single span and multispan bridges are used to build SPUIs. For overpass designs, simple span and three-span bridges are commonly used. If frontage roads with U-turn lanes are used, three-span overpass bridges are always used. Otherwise, local topography, aesthetics, or cost may impact the choice of bridges. General openness and sight distance requirements should also be considered. For underpass designs, two-span bridges supported by a median pier is the likely design. Simple box platform designs with flared ramp framing seem to be the likely choice, although cast-in-place concrete slabs have been used in special situations were this type of construction is more economical.

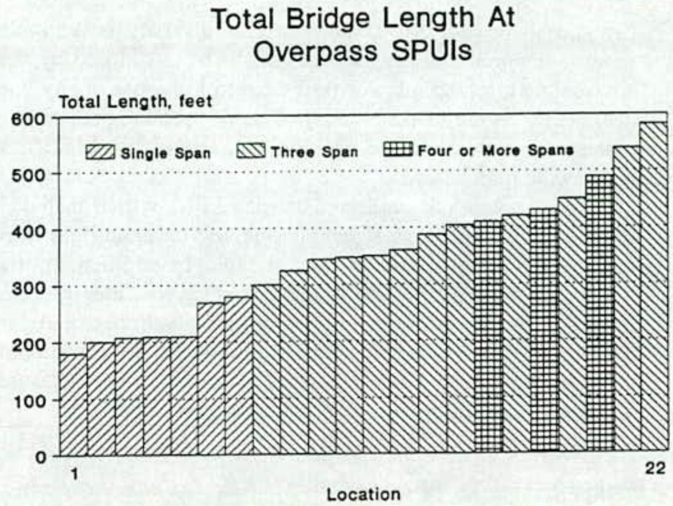


Figure 30. Total bridge length at overpass SPUIs.

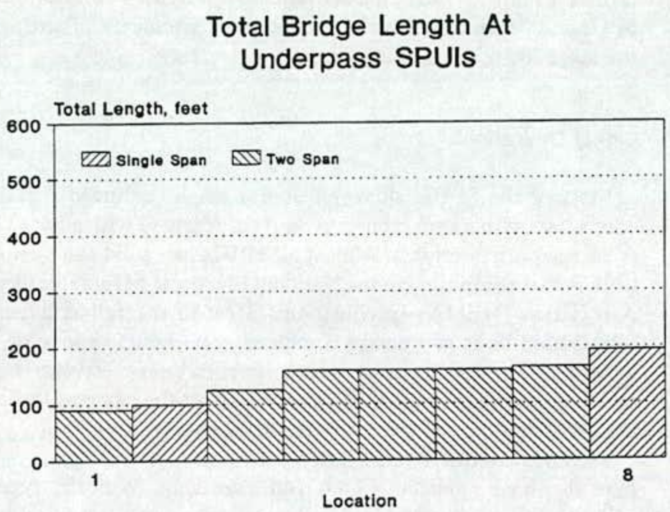


Figure 31. Total bridge length at underpass SPUIs.

CHAPTER THREE

TRAFFIC OPERATIONS

Provision of safe and efficient traffic operations at a SPUI is highly dependent on the quality of its traffic control plan. This plan includes all elements of the SPUI that regulate, warn, or guide motorists traveling through the interchange area. One of the main traffic control devices is the traffic signalization, which includes the controller, signal heads, signal phasing, and signal timing. Other equally important elements of the traffic control plan include signing, marking, and lighting elements. Because of the SPUI's somewhat unique operation, the safety and efficiency of its operation depend on the coordinated design and application of its traffic control devices. As some motorists will be unfamiliar with the physical layout and operation of a SPUI, generous visibility of all physical features should be provided. Enhanced illumination of the interchange area likewise is to be encouraged.

This review of traffic operations at SPUIs is based on the signal timing data and operational experience obtained during the field survey portion of this project. This field study took place during the summer of 1989 and included visits by project team members to 23 operational SPUIs in the United States. The data base is composed of a diverse collection of SPUIs. For example, it includes SPUIs from 14 states, SPUIs that have been in operation for over 15 years, and SPUIs with cross-road traffic demands in excess of 50,000 vpd. This diversity is desirable because it minimizes the bias that might be introduced by the environment, topography, or traffic control practice of any one geographic region of the country. This perspective permits the assessment of local traffic operations practice compared to a nationwide data base.

Traffic control data collected at each SPUI visited included signal controller settings, signal phasing, left-turn and right-turn control, pavement markings, and signing. In addition, traffic operations were recorded on videotape, which was later used for obtaining average cycle lengths and traffic volumes, and for making general qualitative assessments of the SPUI's operational efficiency and safety. Some nighttime illumination surveys were also conducted using a modern digital photometer.

TRAFFIC SIGNAL CONTROL

The following discussion pertains to the basic operational features of the SPUIs observed for this study. Initially, some of the traffic control elements that influence SPUI operation will be described. Then, a brief comparison of the SPUI to other more traditional designs will be made to illustrate some of their similarities. Finally, some of the operational differences between the SPUI and other designs will be mentioned as a means of setting the stage for the remaining sections.

Signal Operations

Most of the SPUIs surveyed used a single, actuated signal controller (with a background cycle if coordinated with adjacent cross-road intersections). Almost all SPUIs had solid-state controllers that conformed to the National Electrical Manufacturers Association (NEMA) specifications. Most of the full-actuated SPUIs used basic gap timing combined with inductive loop detection in advance of and at the stop line. In many respects, the techniques used to control SPUIs were about the same as those used for high-type at-grade intersections (AGIs).

Pretimed control is commonly used in coordinated signal systems that have predictably high traffic demands. With this type of control the phase (and interval) durations are preset to values that are representative of traffic demands over a relatively long period of time, e.g., the peak hour. Only one SPUI found during the field survey operated under pretimed control; the controller at this SPUI was electromechanical and was interconnected into an arterial signal system in a large city where progressive flow was a priority.

As with any signalized arterial, the benefit of interconnecting signals along the arterial is improved coordination of the through movements and, thereby, more efficient performance. The need for coordination of adjacent signals increases as the signals are placed nearer to one another. For example, a signalized offset frontage road would likely require some type of coordination with the interchange's controller to preclude the buildup of traffic queues between the two conflict areas. Of the three SPUIs with offset frontage roads, the two that were signalized were also interconnected with the SPUI controller to improve their combined operation. Another agency added the SPUI to their computerized signal system, using a NEMA eight-phase controller for local control.

In general, only a few of the SPUIs studied are interconnected with the adjacent signals along the cross road. However, where interconnection between signals is used, the delay-reducing benefits of coordination were obvious. Discussions with various operating agencies indicated that the SPUI's three-phase operation and single signal control made it well suited for providing two-way traffic progression along the cross road unlike that experienced for most TUDIs.

The choice of signal phasing sequence for a particular intersection is dependent on many factors. The basic signal phasing at the intersection of two streets consists of two phases — one for each street. However, when left-turn volume or left-turn-related accidents are relatively high, protected left-turn phases are often added at high-type intersections. The number and sequencing of left-turn phases depend on the desired left-turn operation and potential impact that a protected left-turn phase may have on other movements.

If a protected left-turn phase is included, it can occur before or after the through movement. If the left-turn phase precedes the through movement on the same approach, it is called a "leading" left-turn phase. In contrast, when the left-turn phase follows the adjacent through movement it is called a "lagging" left-turn phase. In some situations, one left-turn movement leads its adjacent through movement and the opposing left-turn movement lags its through movement; this is commonly called "lead-lag" phasing. Both leading left-turn phasing and lagging left-turn phasing have advantages and disadvantages, as described in leading traffic signal design manuals (*16*).

The following sections describe the application of the above signal phasing principles to the SPUI. Initially, the number and sequence of signal phases used for SPUIs will be described. Then, the type of left-turn phasing and its operation will be discussed. Finally, selected signal timing parameters, such as cycle length and change interval duration, will be briefly examined.

Signal Phasing Without Frontage Roads

The signal phasing sequence used at almost all SPUIs without frontage roads is shown in Figure 32. Figure 32 is a basic three-phase sequence with a leading left-turn phase on the cross arterial and an overlap phase between phases 1 and 2. Only the controlled movements provided protected operation are shown in Figure 32 and subsequent figures. Related permissive right-turn movements may be present in a phase, but are not shown. When a one-way frontage road system is present, a fourth phase is added following phase 3. This phase may be overlapped with phase 3. Frontage road phasing will be more thoroughly described in the next section.

Each phase of the three-phase sequence (shown in Figure 32) serves two nonconflicting movements simultaneously. First, phase 1 serves both cross-road left-turn movements. If one of these left-turn movements is heavier than the other, an overlap phase is called to serve the additional left-turn vehicles and the adjacent through movement simultaneously. Next, phase 2 serves cross-road through traffic in both directions. Finally, phase 3 is called to serve both off-ramp left-turn movements. It should be noted that the right-turn movements are not normally signalized, although this may be an issue if pedestrian signals are used.

The phase sequence shown in Figure 32 has the cross-road left-turn movement leading its adjacent through movement. This also is the most common treatment at an AGI, which is probably the reason why it is used almost exclusively at the SPUIs studied. A leading left-turn phase is desirable because it is more commonly found and, thus, more consistent with driver expectancy. On the other hand, a lagging left-turn phase or lead-lag left-turn phase combination may be better able to provide arterial through movement progression. In fact, the one SPUI with pretimed control was observed to have lead-lag phasing for the stated purpose of providing better arterial coordination.

Signal Phasing With Frontage Roads

Three types of signal phasing were observed at SPUIs with combined one-way frontage roads. These three types vary in the sequence of the phases and in the combination of movements permitted during each phase. The advantages and disadvantages of each type, when applied to a SPUI, will be more thoroughly discussed in the following paragraphs.

The first type of phasing is similar to that described in the preceding section and is shown in Figure 33. As described previously, this type of phasing is called "lead-lead" phasing because all left-turn movements precede their adjacent through movements. Two SPUIs with frontage roads were observed to have the lead-lead left-turn phase sequence.

As mentioned previously, leading left-turn phasing has both advantages and disadvantages. The advantage of leading left-turn phasing is that it is consistent with left-turn phasing at most other signalized intersections. On the other hand, two disadvantages of this type of phasing emerged during the field studies. First, it was claimed that leading left-turn phasing would require longer red clearance intervals than would a lagging phase sequence. The implication being that a leading left-turn phasing makes less efficient use of the signal cycle time. The second disadvantage pertains to the simultaneous operation of the left-turn movements, the degree of separation between moving left-turn vehicles and driver comfort. The concern found here relates to driver apprehension about meeting and passing by opposing left-turning vehicles on the right with minimal left-turn separation distance.

A second type of phasing is similar to that shown in Figure 33, with the difference being in the treatment of the off-ramp and frontage-road movements. For this second type, shown in Figure 34, the frontage-road left turn and through movements on each approach are assigned to the same phase. Thus, that portion of the signal cycle serving the frontage road traffic is split between the two approaches, e.g., northbound left-turn and through movements in phase 3, southbound left turn and through movements in phase 4. This type of split phasing is often called "directional separation" because only one traffic movement from one direction (approach) can enter per phase (*16*).

Directional separation phasing was used at four small SPUIs in one city. Discussions with the city traffic engineer indicated

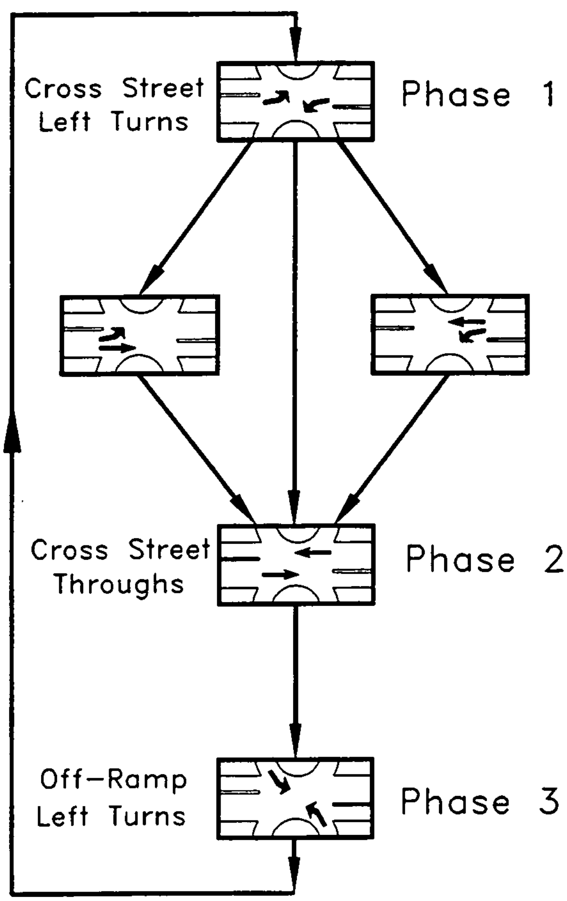


Figure 32. Typical SPUI 3-phase sequence.

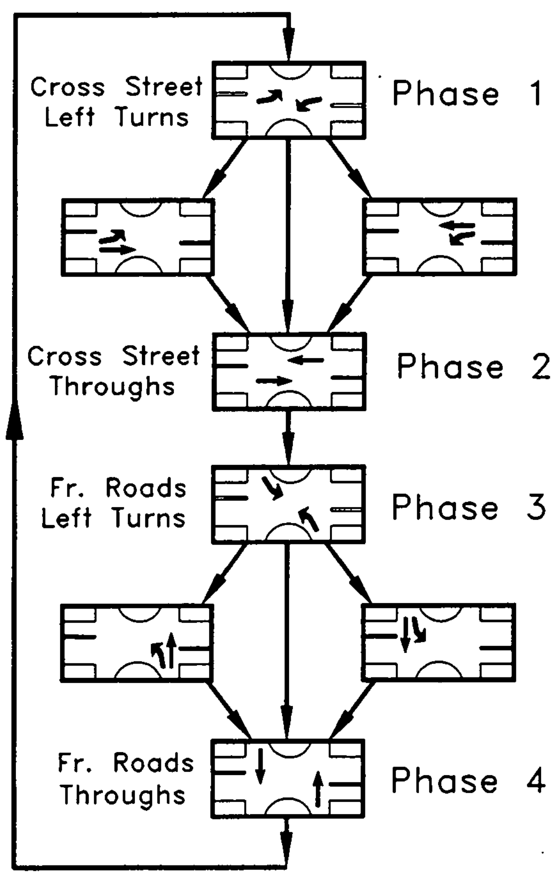


Figure 33. SPUI "lead-lead" 4-phase sequence.

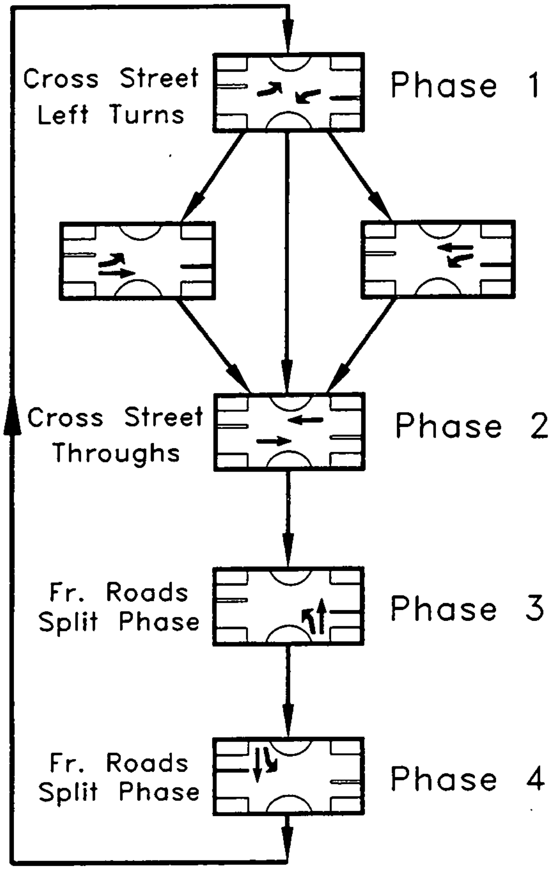


Figure 34. SPUI "directional separation" 4-phase sequence.

that the original signal design called for more conventional leading left-turn movements on the off ramps; however, this phasing was changed after operational experience indicated two types of problems. One problem was that left-turning off-ramp motorists were generally uncomfortable passing so close to opposing off-ramp left-turn vehicles. A second problem was encountered during the change interval wherein high-speed left-turning vehicles from the off ramps were sometimes unable to stay within their lanes.

Examination of the geometric design of the above SPUIs (built in a 200-ft right-of-way) indicated that the tight horizontal alignment of the off-ramp left-turn paths contributed to the poor operation of the leading left-turn phase. In particular, the left-turn path included a "broken-back" design (presumably done to minimize bridge length given the tight right-of-way). The tangent portions of the broken-back curve design provided only 5 ft of separation between opposing off-ramp left-turn vehicles. Although 5 ft of separation is usually acceptable on two-lane highways where vehicles meet on their left sides, this was apparently not enough separation for drivers in the process of negotiating a relatively high-speed turn where they met on their right side and on the outside of converging curves without superelevation. To eliminate this problem, directional separation phasing was used to separate the off-ramp left-turn vehicles in time, thereby permitting only one left-turn movement per phase. Interchange capacity was obviously sacrificed for improved safety.

The third type of phasing found at a SPUI with frontage roads is the lag-lag left-turn sequence shown in Figure 35. This phase sequence is also similar to that shown in Figure 33; however, the order of the through and left-turn phases has been switched such that the left-turn movements lag the through movements. This lag-lag left-turn treatment was observed at one SPUI. One potential advantage of the lag-lag phasing sequence is that it may permit shorter clearance times than the other sequences. In fact, the agency that uses lag-lag phasing at their SPUI claims that the total red clearance time (for all phases) is 8.3 sec for lag-lag phasing compared to 13.4 sec for lead-lead phasing. The benefit of shorter red clearance times is the more efficient use of the cycle time which leads to less motorist delay. This issue will be further discussed in the next section on signal change intervals.

Signal Cycle

The duration of the signal cycle and the intervals that comprise it are established by different criteria; however, the goal in all cases is the safe and efficient operation of the intersection. For example, the green interval should be long enough to serve the traffic demand for its respective phase. A typical range for the green interval is 20 sec to 40 sec in high-volume conditions. In contrast, the duration of the yellow interval is based on the need to inform approaching drivers of the impending transfer of right-of-way to the next phase.

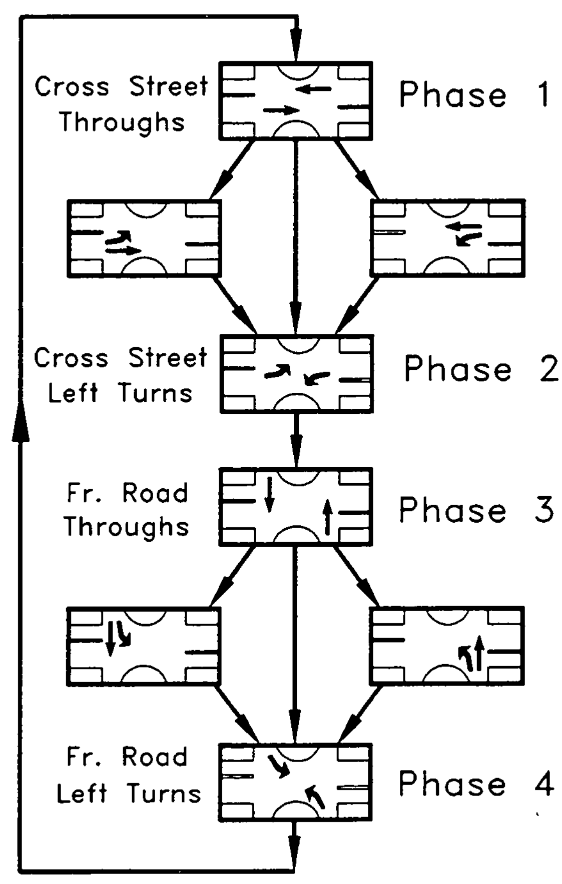


Figure 35. SPUI "lag-lag" 4-phase sequence.

Although a few SPUIs are under pretimed control or have background cycles for coordination, most use vehicle-actuated control in the local mode. In this mode, the cycle length is allowed to vary with demand and is constrained only by the minimum and maximum green settings on the controller. As a result, the cycle length must be manually measured in the field if its length is desired. Moreover, because of its variability, several cycles must be measured and averaged in order to obtain the best estimate of typical cycle length during the specific time period.

The duration of the cycle and signal intervals were obtained for several SPUIs during the field survey. In some cases, the signal timing data were provided by the operating agency; however, in all cases, a videotape record was made of each SPUI's operation during the peak traffic demand periods. These recordings were replayed in slow motion to measure the cycle lengths at which the actuated SPUIs were operating during peak traffic demands. Of the 23 operational SPUIs studied, 16 SPUIs were found to have three-phase operation and 7 SPUIs were found to have four-phase operation. The SPUIs with four-phase operation also have combined frontage roads. Of the 16 conventional three-phase SPUIs, cycle lengths were obtained for eight. The average cycle length observed was 90 sec; the range was between 70 and 104 sec.

Of the 7 SPUIs with frontage roads, cycle lengths were observed for five. The average cycle length of these 5 SPUIs was 133 sec with a range between 100 and 150 sec. Comparing the three- and four-phase cycle lengths observed during the peak hours, it would appear that the four-phase sequence requires an additional 43 sec of cycle time. Alternatively, the average phase duration of the four-phase sequence is about 3 sec greater than that of the three-phase sequence (33 sec vs. 30 sec).

Observation of these SPUIs does not suggest that significantly longer or shorter cycle lengths are necessary to improve operations. However, it should be noted that the longer cycle lengths associated with the four-phase sequence may cause lengthy delays for some movements. In general, it appears that most agencies responsible for the SPUI's signal timing approach it as they would an AGI. This is reflected in the fact that the observed cycle lengths are typical of those commonly found at high-type AGIs. As discussed below, red clearances average about 2 sec longer.

Signal Change Interval

The signal change interval includes a yellow change interval to warn approaching traffic of impending loss of right-of-way and also usually includes a red clearance interval to permit late-arriving traffic time to clear the intersection conflict area before the phase ends. Thus, the "change interval" is composed of two separate signal intervals:

$$CI = YL + RC \tag{1}$$

where CI = signal change interval, sec; YL = yellow change interval, sec; and RC = red clearance interval, sec.

This section will initially describe the method used to determine the duration of these two intervals and then focus on their application to SPUIs. The length of the yellow interval is a function of the approach speed of the traffic stream that is losing the right-of-way. The yellow interval typically ranges from 3 sec to 5 sec. The red clearance interval is usually included at intersections with a relatively large conflict area to provide additional time for vehicles to safely clear the intersection. If a red clearance interval is used, it generally ranges from 1 sec to 2 sec at an AGI, but may be larger at SPUIs or other facilities as needed.

ITE Recommended Procedure

In general, the yellow indication, YL, is intended to give drivers advance warning of an impending loss of right-of-way and thereby allow them sufficient distance to stop or adequate time to safely proceed through the intersection if unable to stop. The formula recommended by the Institute of Transportation Engineers (ITE) (*17*) for determining the length of the yellow change interval is:

$$YL = T + V/(2d + 2gG) \tag{2}$$

where YL = yellow change interval, sec; T = driver perception/reaction time, use 1.0 sec; V = velocity of approaching vehicle, fps; d = deceleration rate, use 10 fpss; g = gravitational acceleration, 32.2 fpss; and G = grade of approach, ft/ft.

The red clearance interval, RC, if included, follows the yellow indication and is intended to provide sufficient time for those vehicles entering during the yellow to clear the intersection conflict area. One of the following equations is recommended for determining the length of the red clearance interval, depending on the amount of pedestrian activity crossing the clearing vehicle's path (*17*):

Amount of Pedestrian Activity	Red Clearance Interval Equation, RC
If none, use:	$(W + L)/V$
If some, use	larger of P/V or $(W + L)/V$
If significant, use	$(P + L)/V$

where RC = red clearance interval, sec; W = clearance path length measured from the near-side stop line to the far edge of the conflicting traffic lane along the actual vehicle path, ft.; P = clearance path length measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path, ft.; L = length of vehicle, use 20 ft; and V = speed of vehicle through the intersection, fps.

The recommended method for applying the foregoing equations is specific to the type of maneuver the approaching vehicle will make. For a through movement, the total change interval is calculated twice: once for the 15th percentile speed and once for the 85th percentile speed. The longer of these two change intervals is then used. The purpose of these two calculations is to ensure that the change interval is adequate for both the slow and fast drivers. The following procedure can be used to estimate the individual yellow and red interval durations of the change interval:

Change Interval (CI) Calculation for Through Movements

$$CI_{85} = YL\,(V_{85}) + RC\,(V_{85})$$
$$CI_{15} = YL\,(V_{15}) + RC\,(V_{15})$$

CI = larger of CI_{85} or CI_{15})
$YL = YL\ (V_{85})$
$RC = CI - YL\ (V_{85})$

where CI_{85} = change interval calculated using the 85th percentile speed, sec; CI_{15} = change interval calculated using the 15th percentile speed, sec; CI = change interval retained for use, sec; V_{85} = 85th percentile speed, fps; and V_{15} = 15th percentile speed, fps.

Note that if a speed sample is not available, the 85th percentile speed can be assumed equal to the posted speed limit with the 15th percentile speed being 10 mph lower.

The change interval duration for protected left-turn movements is slightly more complicated for two reasons. The first complication stems from the generally lower speed at which the turn is made relative to the approach speed. In recognition of this, ITE recommends using the average turn execution speed to determine the length of the red clearance. The second complication involves the manner the left-turn is approaching the conflict area. In one case, a left-turning vehicle could be approaching at a speed higher than the turn speed. On the other hand, the left-turn vehicle could be in queue and approaching the intersection at a crawl speed. To accommodate this latter complication, the average of the 85th percentile approach speed and the turn speed is used to determine the length of the yellow interval. The following technique is recommended by ITE for turning movements (*17*):

<u>Change Interval (CI) Calculation for Left-Turn Movements</u>
$V_y = (V_{85} + V_{lt})/2$
$YL = YL\ (V_y)$
$RC = RC\ (V_{lt})$

where V_y = average of approach and turn execution speed, fps; and V_{lt} = average turn execution speed, fps.

As the foregoing procedure indicates, determining the change interval duration for each movement is not a trivial matter. The distribution of approach speeds, average turn speed, and the clearance path length are not normally available or easily obtained, particularly for new SPUIs. However, as a minimum, good estimates of each variable are needed to obtain an initial estimate of the change intervals. Improved change intervals may be derived from field data taken several months after the SPUI has been open to traffic and operating speeds have stabilized.

Current Practice

Because the change interval represents time during the signal cycle that traffic is stopping or stopped, a portion of it is "lost time" for moving traffic. Hence, traffic engineers may consider avoiding the use of a red clearance in an attempt to maximize intersection capacity. Although this practice occurs for AGIs, all agencies contacted during the survey recognized the need for red clearance intervals at SPUIs.

The ITE recommended practice for determining the length of the change interval is not widely adopted. Thus, one of the goals of this research was to determine what methods or procedures were being used for SPUIs. Therefore, as part of the field survey conducted for this study, the duration of the signal change intervals at 12 SPUIs was obtained from the operating agency or extracted from the videotape records taken during the site visits. Analysis of these data indicated a wide range of change interval durations—particularly for the red clearance interval. Table 3 summarizes the duration and variability of the change intervals found.

As indicated in Table 3, the average duration of the observed yellow warning intervals fell within the range commonly found at at-grade signalized intersections, i.e., 3 sec to 5 sec. The relatively low variability (as indicated by the standard deviation) of the yellow warning interval suggests that most agencies do not use yellow intervals outside this range. The methods used for determining the yellow interval duration ranged from a constant amount (applied at all intersections in the area) to a straightforward use of the ITE procedure. None of the agencies surveyed precisely followed the entire ITE procedure, particularly for left turns.

In contrast to the yellow warning interval, the red clearance intervals found at the SPUIs studied were much more varied. The red intervals were found to range from 1 sec to 10 sec per phase. The average red clearance interval of 3.4 sec is about 2.0 sec longer than typically used at AGIs and suggests that most agencies recognize the need for longer clearance times at SPUIs. However, the wide variability suggests that some operating agencies may be concerned about the SPUI design and are cautiously using long red clearance intervals. Other agencies treat the SPUI like any other high-type AGI and use a nominal 1.0-sec red clearance per phase. Discussions with several operating agencies indicated that most do not explicitly use the above procedure recommended by ITE for determining the length of the red clearance interval at SPUIs.

One agency has used the signal phase sequence to justify a reduction in the length of the red clearance interval. This method takes advantage of the SPUI's special geometrics in recognizing that the clearance interval needs only to be long enough (discounting pedestrian conflicts) for the exiting vehicle to cross the furthest conflicting path of the entering vehicle. In application,

Table 3. Signal change interval statistics.

	Number of Samples	Average Yellow Interval (sec)	Average Red Clearance (sec)	Total Change Interval (sec)	Calculated Red Clearance[a] (sec)
3-Phase SPUIs					
Average/Phase	8	4.0 (0.8)[b]	3.4 (2.9)	7.4	3.5 (0.6)
Total/Cycle	8	12.0 (2.3)	10.2 (8.8)	22.2	10.6 (1.7)
4-Phase SPUIs					
Average/Phase	4	3.9 (0.3)	3.3 (1.0)	7.2	4.5 (0.7)
Total/Cycle	4	15.8 (1.0)	13.4 (4.2)	29.2	18.1 (2.8)
Combined 3 & 4-Phase SPUIs					
Average/Phase	12	4.0 (0.6)	3.4 (2.4)	7.4	———

Notes:

a - Red clearance intervals calculated using ITE methodology (*17*).
b - Numbers in parenthesis represent the sample standard deviation.

it was found that when the left-turn phase lagged the through phase, the unique combination of entering and exiting vehicles reduced the clearance length for each phase by one to 2 sec. One drawback of this method is that it may not work in areas with pedestrian activity because the reduced clearance interval will probably not be long enough for the exiting vehicle to clear any far side crosswalk. Another drawback to this method is that it requires every phase in the sequence to be presented every cycle, i.e., pretimed control or all phases on recall, unless special external clearance logic is added to the controller. This operation has the potential to be inefficient because it will bring up each phase regardless of whether or not traffic is present.

Another method for reducing the clearance interval follows the approach taken in Germany where the "entrance time" of vehicles in the next phase is used to reduce the red clearance time interval (*18*). This method has particular merit at SPUIs because of the lengthy entrance distances associated with its design. To date, it is believed that this method is not being explicitly used because of potential safety implications. However, it is likely that entrance time is being indirectly used to some degree at those SPUIs that have clearance intervals that are shorter than those that would be found using the ITE procedure.

Another means of examining the red clearance intervals in Table 3 is based on the variability of their duration among sites. For this examination, the duration of the red interval for each phase was calculated for 9 of the 12 SPUIs using the existing phase sequence and the methodology proposed by ITE. The results of these calculations are given in the last column of Table 3. Based on these data, it would appear that only a small part of the variability in existing red intervals can be attributed to differences in geometry. In fact, much of the variability can only be attributed to the result of differences in local practice. It should also be noted that the red clearance intervals calculated for the four-phase sequence were found to average about 1 sec more than those for the three-phase sequence.

Based on this survey, methods used by local agencies for determining the duration of the change interval may need to be reevaluated. Use of minimal durations may not be prudent when considering the SPUI's relatively new and unusual geometry as well as its significant physical size. It is possible that intervals, perhaps 2 sec longer than typically used at AGIs, may be needed to provide an added margin of safety; at least until the SPUI design becomes more familiar to local motorists.

The total signal change interval is significantly impacted by the size of the signalized intersection control area and the resulting length of the clearance path, *W*. Normally, right-turn conflicts from the off ramps are ignored if they are YIELD-controlled. However, if the right turns are signalized, say because of pedestrian crossings, then the intersection control area would be significantly increased. Right-turn signalization may add 2-sec additional red clearance time per phase.

LEFT-TURN OPERATION

The signal phase sequence used at almost all three-phase SPUIs includes two left-turn movements and one through movement. In this case the efficiency of the signalized portion of the interchange is highly dependent on the efficiency of the left-turn movements. This section will examine techniques used to improve left-turn operation such as allowing protected/permitted operation and the use and number of exclusive left-turn lanes.

One method used to improve left-turn operation at AGIs is to allow permissive left-turn operation in combination with the protected-only (or simply "protected") left-turn phase. This type of "protected/permitted" operation permits left-turn drivers to turn during the adjacent through movement's phase after yielding the right-of-way to any through traffic. As a result, that portion of the through phase unused by opposing through traffic can be used for left-turn maneuvers. This can effectively increase the capacity of the left-turn movement and, thereby, lower motorist delay.

In spite of the operational benefits of protected/permitted phasing, there are several safety-related considerations that limit its applicability. One recent study has shown that a protected/permitted phase creates an increased left-turn acccident potential approximately equal to that of an unprotected left-turn movement (*19*). This study also identified the conditions under which protected/permitted phasing should not be considered; those unfavorable conditions that might be applicable to a SPUI are through movement speeds on the cross road that are over 45 mph; more than two opposing through lanes to cross (because of the large gap between through vehicles needed by the left-turn driver to clear the extended conflict area); and dual-lane left-turn movements (because of adverse interaction between left-turn vehicles in adjacent lanes and restricted sight distance).

In recognition of these conditions, the agencies contacted during the field survey indicated that protected/permitted left-turn phasing was not a feasible option. In fact, all of the SPUIs surveyed had protected-only left-turn phasing for both the cross road and off-ramp left-turn movements.

Two agencies offered some additional insight regarding the application of protected/permitted phasing to the cross-road left-turn movement at SPUIs. One agency that had considered using protected/permitted phasing noted that the left-turn driver attempting a permitted maneuver would not be able to see the left-turn signals if their mounting location was on the bridge facia relatively near the stop line (which they typically are at SPUIs). Thus, a supplemental left-turn signal would be needed on the far side of the conflict area to keep the left-turning driver informed of the status of the signal indication. Another agency stated that it had used protected/permitted phasing for a brief period, but reverted to protected-only phasing after experiencing a significant number of left-turn accidents.

All of the SPUIs surveyed have either one or two exclusive lanes for the cross-road and off-ramp left-turn movements. About two-thirds of these SPUIs have dual-lane left-turn bays for the off-ramp movements and the other one-third have single-lane bays. Similar to the off-ramp left turn, single-lane left-turn bays are provided for the cross-road left-turn movement at about one-third of the SPUIs and the other two-thirds have at least one dual-lane left-turn bay on the cross road. The dual left leading signalization requires that left-turn bays be provided in advance of the overpassing structure. In general, the survey indicates that dual-lane left turns are preferred by most SPUI designers for the traffic conditions served. The added turn lanes provide additional signal flexibility and a significant increase in interchange capacity at some modest increase in signal clearance times.

RIGHT-TURN OPERATION

The right-turn maneuver at a SPUI can be categorized as one of two types, depending on the approach from which it departs. In particular, there are cross-road right-turn maneuvers and off-ramp right-turn maneuvers. The characteristics of each maneuver define the nature of its operation and can give some insight as to its capacity and safety. These characteristics include the geometry of the turn path, complexity of the entrance maneuver, capacity of the maneuver (as affected by the SPUI traffic signal), and type of traffic control used to regulate the entry into the downstream roadway.

Examination of the characteristics of the right-turn maneuver make it apparent that each type of right-turn maneuver at a SPUI is operationally different from the other. In particular, each right-turn maneuver is significantly affected by the type of traffic control, e.g., STOP, YIELD, etc., the number of conflicting signalized movements, and the signal timing of the conflicting movements. The right-turn movements also differ in terms of the sight angle and distance to the conflicting movements. The following sections will more fully discuss the characteristics of the right-turn maneuvers and their application to the cross road and off-ramp right-turn movements at a SPUI.

Characteristics of the Right-Turn Maneuver

The geometric characteristics of the right-turn path include the magnitude of the turn radius, the length of right-turn bay (if any), and the use of an auxiliary acceleration lane at the end of the turn. In general, the right-turn maneuver will operate more safely and efficiently if a right-turn bay and auxiliary lane are provided. These geometric features have the benefit of physically separating the right-turn vehicles from other vehicles so that all can proceed to their best advantage. Right-turn operation can also benefit from larger radii turn paths; however, very large radii can increase the complexity of the entrance maneuver.

The safety and efficiency of the right-turn maneuver are highly dependent on the complexity of the entrance maneuver into the conflicting traffic stream. The complexity evolves from several sources. First, the SPUI traffic signal phasing regulates traffic flow past the entrance point such that the signals form platoons of highly concentrated traffic. The alternating sequence of high and low flows past the entrance point are not unique to the SPUI; however, the distances within the SPUI conflict area are so large that it may be more difficult for the right turn driver to visually locate the sources of conflicting traffic and predict their arrival at the merge.

A second complexity associated with the right turn entrance maneuver stems from the angle of entry and the physical requirements necessary to verify the safety of entering the conflicting stream. In general, the angle of intersection of the right-turn movement and the conflicting stream can degrade the right turn driver's ability to verify the availability of an acceptable gap in the approaching traffic. As the angle of entry becomes sharper, the driver must turn his or her head further to the left to see approaching traffic and yet safely negotiate a turn to the right. The angle of entry typically increases with the skew angle of the interchange and radius of the right turn.

The capacity of the right-turn maneuver is dependent on the type of traffic control used. If STOP, YIELD, or MERGE control is used, the right-turn capacity is dependent on the availability of gaps in the conflicting stream. On the other hand, if the right turn is controlled by the SPUI traffic signal, the right-turn capacity is a function of the amount of cycle time provided to it. In the case of STOP, YIELD, or MERGE control at a SPUI, the gaps in the conflicting stream are created artificially by the traffic signal. Thus, during some signal phases, there will be dense platoons of traffic flowing past the right turn entrance point, with almost no entrance opportunities. During other phases, there may be no conflicting flow and the right-turn movement can depart at its maximum possible rate.

Cross-Road Right-Turn Characteristics

The cross-road right-turn maneuver at a SPUI has some similarities and some critical differences with the corresponding maneuver at a high-type at-grade intersection. In each case, the right-turn vehicles are removed from the cross road just upstream of the stop line. In addition, the right-turn maneuver is completed without control or, perhaps, with YIELD control at the point of completion of the turn, i.e., at the junction with the on ramp. On the other hand, the most significant difference is the lack of a crossing through movement (except at the few SPUIs with frontage roads). Because of this fact, the cross-road right-turn maneuver has only one conflicting movement—the opposing, cross road left turn. As a result, the cross road right turn at a SPUI generally flows freely during two of the three signal phases; having to yield only when the opposing cross road left turn is moving. The basic relationship between the cross-road right-turn conflict and signal phasing is shown in Figure 36.

The efficiency of the right-turn maneuver at a high-type intersection or SPUI is somewhat dependent on whether an exclusive turn bay is provided. A turn bay has the operational benefit of separating right turn traffic from through traffic, thereby allowing both movements to proceed through the intersection when their individual opportunities arise. This benefit can be significant for right turning and through traffic at a SPUI because the through movement is provided with only one signal phase each cycle, while the right-turn movement can move during two phases.

A general recognition of the operational benefits of right turn bays (or lanes) for the cross-road right-turn movements is suggested by their use at most of the SPUIs studied for this research. In particular, of the 36 SPUIs studied, 21 (58 percent) were found to have exclusive right-turn lanes on one or both of the cross road approaches to the SPUI.

A MERGE or YIELD control is typically used at the end of the cross-road right-turn maneuver at the point where the right-turn path intersects with the on ramp. At most of the SPUIs studied, the right-turn maneuver was YIELD controlled. However, at a few SPUIs, MERGE control was used to efficiently join the two on-ramp movements. With MERGE control, the on-ramp geometry included an auxiliary acceleration lane for the right-turn movement. This "added lane" is typically later dropped after providing a nominal distance for ramp traffic to merge into one lane before entering the major roadway.

The right-turn maneuver from the cross road is almost always uncontrolled, or free flow at the point of departure. However, at one SPUI the cross road right turn was signalized for the purpose of improving right turn efficiency. Field experience showed that the desired goal was not achieved and the signal was removed.

As mentioned previously, the cross-road right-turn movement flows freely primarily during two of the signal phases. Only during the phase servicing the opposing cross road left turn will the right-turn movement need to yield the right-of-way on the on ramp. During the cross road through phase and the off-ramp left-turn phase, the right-turn movement will be able to flow without interruption. As a result, the efficiency of the cross-road right-turn movement can be quite high. Of course, this efficiency can only be fully realized if the right turn is provided an exclusive lane on the cross road together with an acceleration lane on the outbound on ramp.

The efficiency of a STOP-controlled or YIELD-controlled right-turn movement is also dependent on the driver's ability to verify the safety of the entrance maneuver. In the case of the cross-road right-turn maneuver, the geometry of the junction point is such that the driver's sight angle is relatively flat such that the conflicting traffic stream approaches more to the front than the left. As a result, the complexity of the entrance maneuver is reduced, thereby decreasing the efficiency of this right-turn maneuver.

The operation of the right-turn maneuver is also dependent on the radius of the turning path. In general, turning traffic moves more efficiently along larger radii—particularly in the case of vehicles with longer wheelbases. Based on data collected during the field study, cross-road right-turn radii were found to range from 40 ft to 229 ft with an average of 118 ft.

In spite of some improved efficiency, large radii appear to have several drawbacks. One drawback of large turn radii is the additional right-of-way they require. In urban environments the cost of obtaining this right-of-way can be excessively high.

A second drawback of large radii is the added length of the interchange area and the increased need for large island channelization. This stems from the need for positive guidance in what would otherwise be a large open area of concrete. In addition, the need for island channelization as a place of pedestrian refuge increases as the width of pavement to be crossed increases.

Another drawback of large turning radii relates to the safety of the entry maneuvers of the on ramp. In particular, large radii promote higher left-turn speeds and, thus, higher entry speeds to the on ramp. Problems associated with higher speeds relate to the increased difficulty of safe merging and lane placement during joint entry maneuvers. Superelevation can not be used to treat off-tracking at the on-ramp merge if the two turning movements share the same lane because of the reverse crown created. Auxiliary acceleration lanes are often added on the on ramp where volumes warrant. This type of design can convert YIELD to MERGE control but with the added increase in right-of-way needs along the major road.

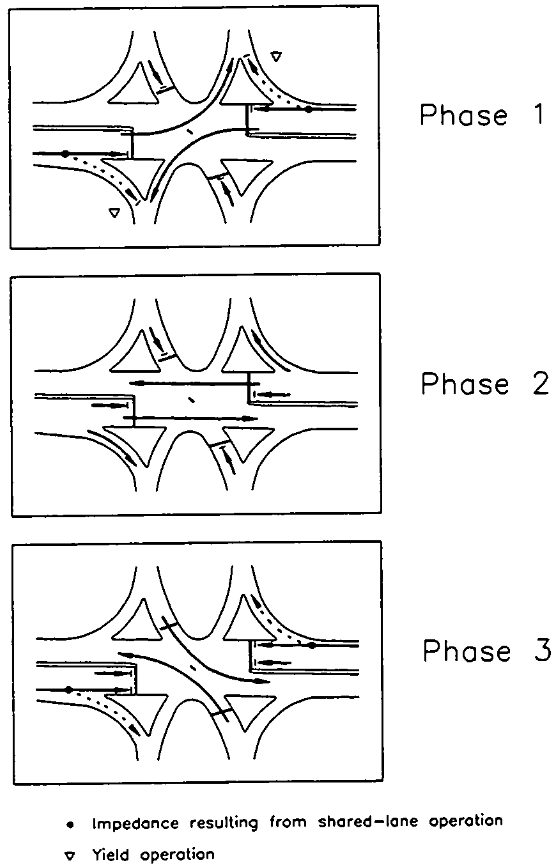


Figure 36. Cross-road right-turn maneuver.

Off-Ramp Right-Turn Characteristics

The intersection of the off ramp and cross road form a junction having a major and minor movement. In all cases, the cross road is the major movement and the off ramp is the minor movement. Like the cross road right turn, the efficiency and safety of the off-ramp right turn is dependent on the type of control used, the complexity of the entrance maneuver, the magnitude of the turn radius, and the availability of separate lanes for right-turn vehicles. Each of these characteristics will be discussed in the following paragraphs.

One of the more significant issues associated with the off-ramp right-turn maneuver is the availability of an exclusive right-turn lane provided on the off ramp. Operationally, a problem can arise when left and right turning traffic share one or more lanes on the off-ramp approach. In general, sharing lanes among two or more different movements will maximize the use of the available pavement; however, to be operationally efficient, the control modes regulating these movements must also be harmonious. Unfortunately, this is not the case for the off-ramp movements at a SPUI without frontage roads. The differences in the control and operation of the off-ramp left and right turn movements at SPUIs can result in inefficient and possibly unsafe operation.

The off-ramp left-turn movement at a SPUI is regulated by the traffic signal, while the right-turn movement is usually YIELD controlled. The relationship between the right-turn maneuver and signal phasing is shown in Figure 37. As this figure indicates, the opportunities for the left turn to move occur only during the off-ramp left-turn phase. In contrast, the right-turn movement can move at any time during the signal cycle—provided that an adequate gap is available in the conflicting traffic stream. However, during two of the SPUI's three signal phases, the conflicting stream is usually quite heavy and does not afford many opportunities for right turn entrance. In fact, there is only one phase, the cross-road left-turn phase, where conflicting traffic streams do not interfere with the off-ramp right-turn movement.

At a SPUI with exclusive turn lanes, off-ramp left-turn vehicles depart during their assigned phase at the maximum possible rate and, as a result, use the phase quite efficiently. However, if

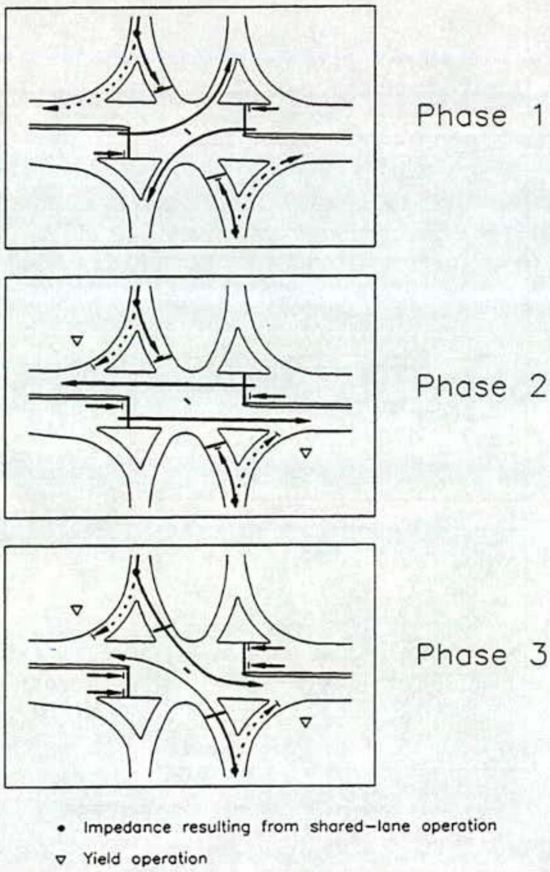


Figure 37. Off-ramp right-turn maneuver.

left-turn and right-turn vehicles share a lane, the right-turn vehicles will likely impede the smooth flow of the left-turn traffic because of their differences in control mode. Specifically, right-turn drivers must yield to opposing off-ramp left-turn traffic, which can be quite concentrated during the off-ramp left-turn phase. As a result, when off-ramp left-turn vehicles move, the adjacent off-ramp right-turn vehicles are likely to be blocked by opposing left-turn vehicles and, if not provided an exclusive lane, will hinder the flow of the left-turn traffic. In a similar manner, off-ramp right-turn vehicles can be hindered by stopped, left-turn vehicles when opportunities exist to safely turn right onto the cross road.

Based on a review of the SPUIs surveyed for this research, the operational benefits of exclusive lanes for the left and right turn movements at SPUIs appear to be well recognized. In particular, 21 of 26 SPUIs were found to have either an exclusive right-turn lane on the off-ramp or over 200 ft of right-turn storage in advance of the cross road. In either case, the separation should be sufficient to eliminate the adverse queueing interaction of the left and right turn movements for the expected traffic volumes and signalization.

The various types of traffic control that have been used to regulate the minor, right-turn movement include STOP, YIELD, MERGE, and traffic signal. The most common type of control was found to be YIELD control. This type of control has the advantages of being relatively efficient in terms of traffic performance and right-of-way need. In particular, YIELD control is operationally efficient because it requires right-turn drivers to stop only when they cannot enter the cross road safely. As a result, right turning drivers can make maximum use of opportunities to proceed with a minimum amount of delay.

STOP control was found at two underpass SPUIs during the field study. It is likely that STOP control was used because of sight distance restrictions along the cross road that were precipitated by a steep upgrade along the off ramp. In both cases, the bridge structure had relatively high parapet walls and curbed islands such that right turning drivers had limited visibility along the cross road until they were at the stop line. At one SPUI the cross road was on an elevated structure and the actual interchange was shifted to one side of the crest vertical curve. The relatively steep upgrade on each cross road approach reduced sight distance along the cross road to the minimum value needed for safe entry. Observations of the off-ramp right-turn movement suggest that drivers had difficulty assessing the adequacy of the entrance maneuver and, as a result, right turn operations were undesirable.

A traffic signal was used initially to control the off-ramp right-turn movement at one SPUI (as shown in Figure 38). The phase sequence of the controller was such that the right-turn phase was concurrent with the cross-road left-turn phase. During this phase, right-turn traffic departed from dual lanes at a relatively high rate of flow and appeared to be quite efficient. When the right-turn phase ended, the right-turn operation reverted to a stop-and-go situation where drivers would yield the right-of-

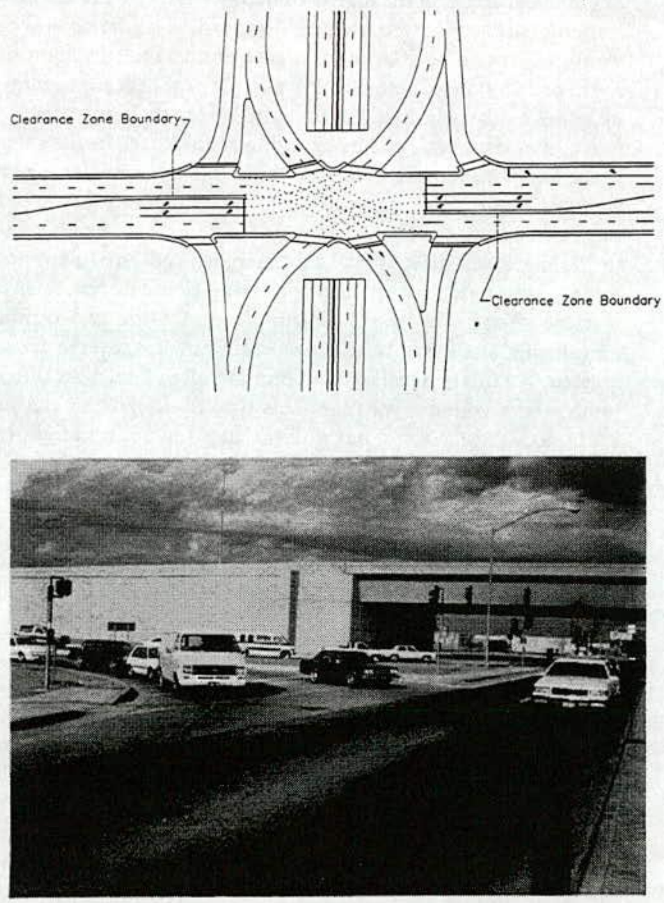


Figure 38. Signalized, dual lane, off ramp right-turn movement.

way to cross-road traffic (particularly for the outside lane). The flow rate for this type of operation was much lower, reflecting the driver's need to verify the availability of a safe gap before entry.

On the basis of observations made during the field study, traffic signal control of the right turn maneuver appeared to operate about as efficiently as YIELD control. The increased efficiency of operation during the green phase was partially offset by the reduced efficiency during the red phase. Any minor improvement in capacity does not appear to justify the added cost of the signal equipment for the off-ramp.

As noted previously, a major disadvantage of traffic signal control for the off-ramp right-turn movement is the increased size of the control area. By including right-turn traffic in the signal sequence, the size of the conflict area and, thus, the length of time needed to clear traffic from within it increase. In general, clearance time is provided at the end of each phase in the form of an all-red interval and represents time that is intended for exiting, or clearing, traffic rather than entering traffic within the intersection area. Although clearance time is essential to safe operation, it adversely affects efficiency because traffic cannot legally enter the interchange. As a result, the efficiency of the interchange will be reduced by the increase in red clearance time associated with signalizing the right-turn movement.

A final type of control that is occasionally used to regulate the off-ramp right-turn maneuver is MERGE control. This type of control requires the use of an exclusive lane or lanes on the cross road for right-turning traffic to enter. This type of control is operationally the most efficient; however, it also requires the most right-of-way and pavement area.

MERGE control essentially moves the entrance maneuver further away from the interchange. In general, this would be a desirable attribute; however, it can create other operational problems if the downstream intersection is relatively near. In particular, a weaving section can develop when the combination of drivers wanting to merge and those wanting to turn right at the downstream intersection reach higher volume levels. This problem can be exacerbated when off-ramp drivers desire to turn left at the downstream intersection, which is frequently the case due to restricted access in the area. When these weaving sections occur, they can have deleterious effects on both off-ramp capacity and the capacity of the cross road.

Common to all of the foregoing types of control is the right turn driver's status as the minor movement. Because of this, the driver cannot enter the cross road through lanes until it is safe to do so. To ascertain the safety of the maneuver, right turning drivers must have adequate visibility along the cross road, upstream of the junction. However, the physical ability of drivers to look to their left, over their shoulder, becomes increasingly difficult as the radius of the turn increases. This difficulty arises because the angle of entry becomes sharper as the radius increases.

Data were collected during the field survey to determine the relationship between the type of control and the magnitude of the off-ramp right-turn radius. From this survey it was found that almost all STOP, YIELD, and traffic signal controlled right turn maneuvers had radii of less than 100 ft. This finding is consistent with the need for relatively small radii as described in the preceding paragraph.

On the other hand, right-turn maneuvers with MERGE control were found to have radii as large as 275 ft. However, in this situation the safety of the entrance maneuver is verified by using the rear-view mirror rather than looking over the shoulder. Because of this difference, MERGE control is not as sensitive to the angle of entry as are the other types of control.

In addition to radius, the task of verifying the safety of the entrance maneuver from the off-ramp is complicated by the different origins and arrival times of conflicting traffic streams. In particular, the signal phasing of the SPUI releases a platoon of cross road through vehicles followed by a platoon of far-side, off-ramp left-turn vehicles past the right turn entrance point. The complication stems from the fact that at typical at-grade intersections (or even conventional diamond interchanges) the source of the conflicting sources of traffic is much nearer to the point of entry. Furthermore, the signal phasing is such that one conflicting movement does not follow the other. However, at a SPUI the greater distance and unique phasing create a complex flow pattern by releasing a second platoon a few seconds after the through phase. This second platoon may surprise right turning drivers who expect to enter freely after the end of the cross-road through phase. In addition, the origin of the second platoon is so distant from the entrance point that it may be difficult for the right turning driver to detect the opposing traffic underneath an overpass bridge structure.

Assessment of Right-Turn Operation

The right-turn movement at a SPUI can directly affect the overall operation of the interchange. This is particularly true when the right and left turn movements share one or more lanes on the off-ramp near the stop line. Differences in control mode typically provide entrance opportunities at different points of time during the signal cycle. As a result, shared lane operation on the off-ramp can seriously degrade performance for both left and right turn movements. Experience has shown that off-ramps with exclusive turning lanes of adequate length do not experience the adverse effects of vehicular interaction.

Observation of more than 20 SPUIs suggests that YIELD control for the off-ramp right-turn movement can be an efficient and cost-effective control mode. This type of control has the advantage of allowing right-turning drivers to enter the cross road whenever possible—without requiring a complete stop. More importantly, the efficiency of the right-turn movement does not come at the expense of excessive right-of-way or signalization. In terms of geometric design, the off-ramp right-turn maneuver appears to operate efficiently when the radius is less than 100 ft and when an exclusive lane is provided along the off ramp.

When operating under STOP or YIELD control, the capacity of the off-ramp right-turn maneuver at a SPUI is highly sensitive to the amount of conflicting traffic. During a typical signal cycle, two phases, i.e., the cross-road through phase and the off-ramp left-turn phase, project dense platoons during the peak hour past the off-ramp right-turn entrance point providing almost no entrance merging opportunities. The only phase that does not interfere with the off-ramp right turn is the cross-road left-turn phase. In contrast, the cross road right turn moves freely during the cross-road through and off-ramp left turn phases but is interrupted by conflicting traffic only during the cross road left turn phase.

Two points can be made here with respect to right turn capacity and phasing. First, the cross-road right-turn maneuver has two signal phases to enter freely, while the off-ramp right-turn maneuver has one. Second, the occurrence of the cross-road right-turn phase is harmonious with the through movement and thus shared lane interaction on the cross road is minimized. On the other hand, the occurrence of the off-ramp right-turn phase conflicts with the off-ramp left-turn phase and necessitates exclusive lanes on the off-ramp for both movements where their queues interact.

The uniqueness of the SPUI's design and operation can make the off-ramp right-turn entry a difficult and complex task. Specifically, the origin and sequencing of conflicting traffic streams are not consistent with driver expectancy and may lead to right-angle and rear-end accidents. One method to mitigate this problem is to locate the off-ramp right-turn entry point further away from the interchange on the cross road using MERGE control. Of course, this would require more right-of-way along the major road. The separation distance to the next downstream intersection is critical if (1) left turns are permitted, or (2) if it is signalized. The separation distance to a downstream signalized intersection having high volume left turns is even more critical for providing safe and efficient off-ramp right-turn operations.

Like the off-ramp right-turn maneuver, the efficiency of the cross road right turn is sensitive to the type of control and geometric design features provided. However, the impact of control type and design features is minimal because of fewer sources of conflicting traffic and better visibility of this traffic. Moreover, the angle of entry is often flatter, which places less physical demand on the right turning driver when trying to view approaching traffic.

The cross road right turn typically has YIELD control on the on-ramp at the junction with the cross-road left-turn movement. A review of accident statistics for several SPUIs suggests that YIELD control is effective in regulating flow at this junction. The use of an exclusive right turn lane on the cross road does not appear to be essential for efficient operation because entrance opportunities for both the cross road through and right turn movements occur at the same time in the signal cycle, i.e., they are both able to move unimpeded during the cross road through phase.

DOWNSTREAM SIGNALIZED INTERSECTIONS

In general, limited access roadways in urban areas concentrate entering and exiting traffic demands at a few, moderately spaced interchanges. This traffic may be either local or regional. Local traffic will access stores and businesses in the vicinity of the interchange. Regional traffic will use the cross road (presumably an arterial) to access intersecting roadways of an equal or lower functional class. Because of this concentration of different trip purposes, traffic flow on the cross road in the vicinity of the interchange will exhibit a relatively large number of turn movements and lane changes. When these maneuvers are combined with high traffic demands, the efficiency of traffic flow along the cross road may be affected.

The combination of high traffic demands and high turn percentages can have an adverse effect on the safety and efficiency of the cross road. Regardless of whether the vehicles are turning at the next downstream signalized intersection or at unsignalized access points, the turbulence created by slower, turning vehicles tends to reduce total cross road capacity, reduce travel speeds, and increase the number of conflicts. Frequently, turning traffic demands in the vicinity of the interchange are so high that they require exclusive turn bays and protected signal phases. In most cases, these added signal phases create additional delay for cross road through traffic.

As alluded to in the preceding paragraph, high volume turning traffic off of (and onto) the cross road, downstream of the interchange, will often require signal control. Although signal control will improve traffic operation at the intersection, it can have an adverse effect on the operation of the SPUI if the distance separating the two junctions is inadequate. In general, the distance between the interchange and downstream intersection should be sufficient to store stopped cross road through and left turn traffic and provide an adequate distance for lane changing upstream of these stopped queues. Larger separation distances are also better suited to signal systems that provide coordination in both travel directions along the cross road.

If the distance between the SPUI's merge point and the downstream intersection is less than some adequate distance, several operational problems may result. For example, if the separation distance is inadequate to store left turning traffic, queues of turning traffic may extend beyond the storage bay and restrict the flow of cross road through traffic. Another problem that can occur relates to the weaving activity (i.e., lane changing) inherent to a cross road in the vicinity of an interchange. Again, if this weaving is constrained to a relatively short length of wide roadway, it can be very disruptive to traffic flow. This problem can be exacerbated when stopped queues further reduce the amount of separation distance available for lane changing. The formation of queues will also degrade the operation of the interchange by blocking traffic flow through the interchange. One movement that is particularly impacted is the off-ramp right-turn movement, as discussed in the preceding section.

Combinations of one or more of the aforementioned operational problems were observed at several SPUIs that had minimum separation distances. The following discussion will examine three specific SPUIs that were found to have operational problems resulting from relatively nearby downstream intersections. As these cases will illustrate, a nearby intersection appears to be one that is located within 1,000 ft of the center of the SPUI. A desirable separation is at least 1,200 ft for high-volume cross-road arterials from the off-ramp merge point to the cross road to the first downstream signal.

Case 1

Case 1 describes an underpass SPUI wherein the major road goes under the cross road. In the vicinity of the SPUI, the cross road operates as major arterial carrying more than 25,000 vpd. It has six through lanes, dual lane left turn bays, and exclusive lanes for the cross road and off ramp right turn movements. In all, the cross road is 10 lanes wide through the interchange, a nominal geometry for SPUIs. The interchange's geometry and other features are shown in Figure 39.

This SPUI is located between two signalized intersections on the cross road. The intersection to the north is located about 850 ft away (centerline-to-centerline). After accounting for the width of the SPUI and intersection, the downstream stop line is only about 650 ft north of the off-ramp right-turn entrance point. The intersection to the south is about 950 ft away; however, the distance from the off-ramp entrance point to the stop line also measures only about 650 ft. It should be noted that the exclusive lanes for the off-ramp right-turn traffic are dropped at the downstream intersections.

During the field study, it was observed that off-ramp right turning drivers did not use the northbound auxiliary lane on the cross road. Rather, these drivers often stopped at the off-ramp entrance point and attempted to merge directly into the downstream. A large number of these off-ramp motorists desire to turn left at the through traffic, presumably to avoid being trapped in the exclusive lane further downstream signal which experiences long queues during rush hours. Traffic flow on the cross road was further aggravated by cross-road traffic wanting to turn right at the downstream intersection. Consequently, the separation between the off-ramp and the downstream intersection did not appear adequate to handle the large weaving volumes that occurred during the peak hours. As a result of these operational problems, lengthy queues were frequently observed along the off-ramp.

Case 2

This case study describes an older overpass SPUI with its cross street having six through lanes and single-lane left-turn bays. The mainline and intersecting highway are major arterials that carry about 70,000 and 50,000 vpd, respectively. The geometry of this SPUI is shown in Figure 40. In general, traffic operations at this older SPUI are good for such high volumes.

To the east of this SPUI along the cross road, however, is a nearby signalized intersection to a shopping mall that was installed after the SPUI was open to traffic. The distance between the SPUI and intersection is about 625 ft centerline-to-centerline but only about 425 ft separate the off-ramp right-turn entrance point and the downstream stop line. The off-ramp right-turn maneuver is YIELD controlled with direct single lane entry.

Observation of traffic operations between the SPUI and the adjacent intersection indicated some operational problems stemming from the minimal separation distance. The major problem at this location results from queue formation on the cross road between the two junctions. Queues develop consistently on the approach to both the SPUI and the downstream intersection during peak traffic periods. Operational problems occur when these queues spill back toward the upstream intersection.

At this particular SPUI, the long queues from the downstream intersection were so frequent and lengthy that they also had an adverse effect on the off-ramp right-turn YIELD movement. This impact has both operational and safety implications. Operationally, spillback of downstream traffic was observed to impact the off-ramp right-turn movement in two different ways. On several occasions, stopped traffic in the cross-road curb lane would block the entrance of off-ramp right turning vehicles. On other occasions, when the queue of downstream left-turn vehicles had grown past the off-ramp entrance point, off-ramp right-turn drivers wanting to turn left at the downstream intersection would

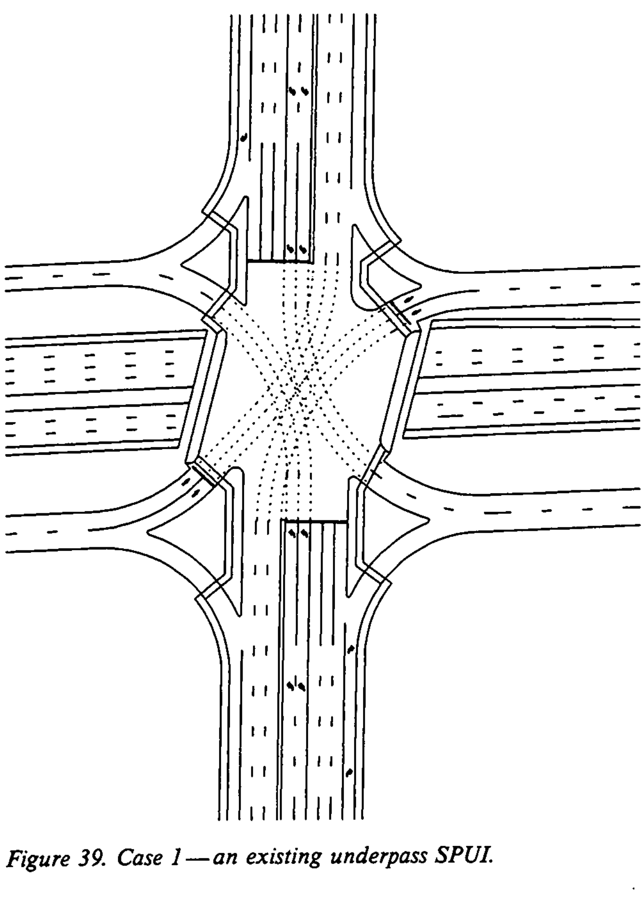


Figure 39. Case 1—an existing underpass SPUI.

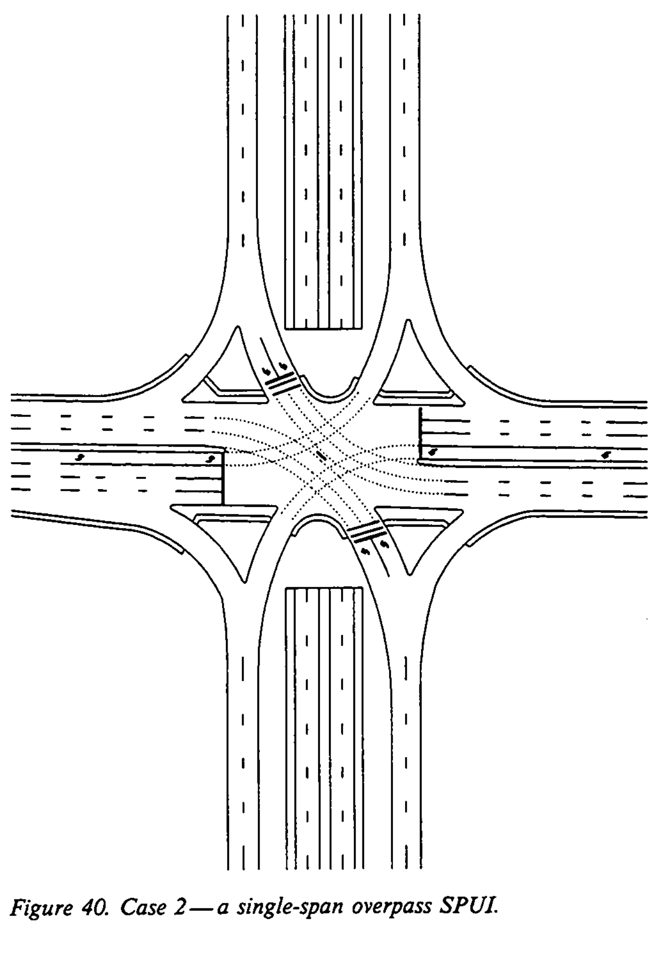


Figure 40. Case 2—a single-span overpass SPUI.

stop and wait in the ramp throat. In each case, spillback on the cross road led to a significant reduction in the efficiency of the off-ramp right-turn movement and the formation of long queues on the off ramp.

Queue spillback from the downstream intersection was also found to have an adverse effect on the safety of the off-ramp right-turn maneuver. This impact stems from the unexpected occurrence of stopped queues of vehicles on the cross road at, or just beyond, the off-ramp right-turn entrance point. When queues are present on the cross road, serious traffic conflicts can occur as evidenced by sudden braking, swerving, and rear-end collisions. Observation of right turn operations at this particular SPUI indicate frequent traffic conflicts of this nature. The accident history of this SPUI indicates that the most frequent type of accident to occur is the rear-end accident between right-turning off-ramp vehicles. Operational problems at this interchange have been recognized and plans are underway to improve its design.

Case 3

This case describes a multispan overpass SPUI that serves major interstate trucking and terminal operations in the vicinity. Triple-trailer trucks are common users of the facility. The cross road through the interchange functions as a minor arterial and carries about 15,000 vpd. Within the interchange, the cross road has four lanes; however, it narrows to two lanes just past the south frontage road. The off-ramp right-turn movement is YIELD controlled. The geometry and other features of this interchange are shown in Figure 41.

Two-way frontage roads exist on both sides of this SPUI along the cross road. Traffic signals at both intersections are interconnected with the SPUI's signal controller to provide coordination of the major movements. The frontage road to the north is offset about 500 ft, while the frontage road to the south is only about 350 ft. Taking the width of the interchange into account, the distance from the off-ramp entrance point to the north frontage road stop line is about 250 ft. The corresponding distance to the south is only about 100 ft.

During the course of the field study, it was observed that a significant restriction to traffic flow resulted from the short separation distance between the SPUI and the downstream intersections. The short distance between intersections combined with high traffic demands (and particularly high truck volumes) has created both weaving-related and queue growth problems. These problems generally result in poor traffic operation and increased traffic conflicts.

The impacts of queue spillback at this SPUI is similar to that found in Case 2. Specifically, queue growth, or spillback, from the downstream frontage road intersection frequently interrupts the flow of off-ramp right-turning traffic at the SPUI. This results in queue formation on the off ramps and increases the potential for rear-end accidents on the off ramps.

To minimize queue formation between the SPUI and downstream intersections the signal controllers are interconnected to provide some coordination among the intersection cross road movements. In spite of this treatment, the close spacing of the intersections combined with the proportionally high volume of turning traffic leads to excessive queue growth during peak periods.

It should be noted that the many triple-trailer trucks operating through this interchange would do so without much impediment if the adjacent service roads were not present. The long left turn radii used for the SPUI promote turning efficiency. Simultaneous operation of triple trailer rigs were often observed to occur in the dual left turn lanes without experiencing any observable delay.

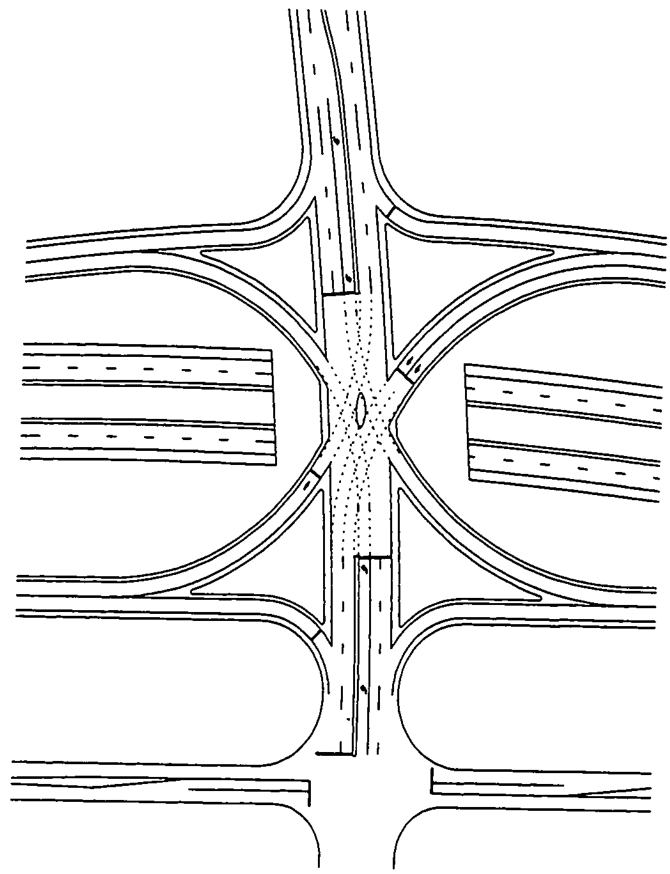


Figure 41. Case 3—a multispan overpass SPUI.

Assessment of Downstream Signalized Intersection Impacts

As the preceding discussion indicates, traffic operations at a SPUI can be sensitive to the relative location of downstream intersections. Operational problems stem from the combination of minimal separation distances, wide arterial, and high traffic demand; particularly, turning traffic demand. This demand can be particularly high in the vicinity of the interchange because of the concentration of local and regional trip destinations (and origins).

In general, operational problems tend to increase as the distance to the downstream intersection decreases. Distance affects the efficiency of traffic flow in an indirect manner by creating suboptimal conditions for traffic progression on the cross road and by limiting the amount of roadway available for lane-changing (or weaving). Arterial width also decreases efficiency by increasing the lateral distance, or number of lanes, that must be crossed during the weave maneuver. When a relatively short separation distance or a wide arterial is combined with high traffic volumes, suboptimal progression and constrained weaving activity will be the likely result. If high traffic demands persist for several cycles, this may lead to reduced cross road capacity and excessive queue growth. Moreover, the existence of stopped traffic on the cross road will further decrease the effective separation distance and further constrain weaving activity.

Field experience indicates that the minimum separation distance to the first downstream signal should be at least 1,000 ft, preferably 1,200 ft on six-lane arterials to provide adequate weaving distance and signal coordination.

PEDESTRIAN CONSIDERATIONS AND SAFETY EXPERIENCE

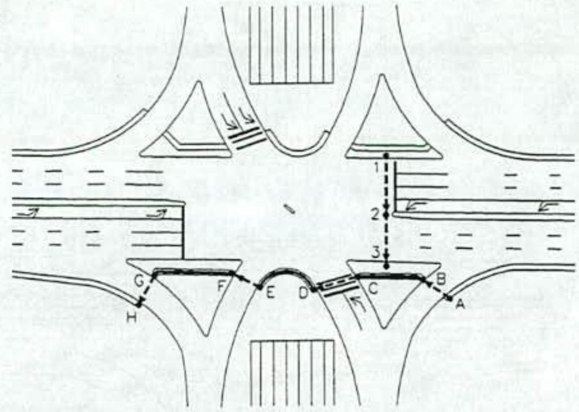
The relatively novel design and operation of the SPUI have evolved from a desire to improve the efficiency of traffic flow through an interchange using a minimum amount of right-of-way. However, the newness of this interchange, in combination with some peculiarities of its signal phasing, has led to some concern about the safety it affords to pedestrians and motorists. In particular, both the physical size of the SPUI and its three-phase signal sequence do not appear to be well suited to providing signal protection to all potential pedestrian movements. In addition, the SPUI's unusual geometry and, again, physical size constrain the application of positive vehicle guidance and may lead to some confusion by unfamiliar drivers.

The problems posed by the SPUI's size, phase sequence, and geometry have been dealt with in a variety of ways. In most cases, pedestrians are discouraged from crossing the cross road at the SPUI by the omission of crosswalks and pedestrian signal heads. Driver guidance through the interchange is provided by a combination of signing, pavement markings, and island channelization. The remainder of this section will be devoted to a more detailed discussion of pedestrian issues at SPUIs.

Pedestrian Considerations

At normal signalized junctions, pedestrians are accommodated within the signalization by provision of a companion through vehicular phase. Unfortunately, at a SPUI this can not be as easily accomplished as it would be at an at-grade intersection. The difficulty stems primarily from the SPUI's three-phase signal sequence and intersection size. The following paragraphs describe these observed difficulties as they relate to the two types of pedestrian maneuvers: crossing the cross road and crossing an on-and-off ramp pair, i.e., parallel to the cross road.

In the case of pedestrians wanting to cross the cross road, the problem is that there is no coincident through traffic phase (except at a SPUI with frontage roads). As a result, either an exclusive, actuated pedestrian phase is needed or the pedestrian movement is not provided; both approaches have been observed at the SPUIs studied for this research. When an exclusive pedestrian phase is not provided, pedestrians crossing the cross road must synchronize their movements with the signal phasing. In particular, the pedestrian would need to cross one-half of the cross road during the one left-turn phase, e.g., the off-ramp left-turn phase, wait on the median for the other left-turn phase, and then complete the crossing. Firsthand experience indicates that this crossing movement can be both difficult and stressful for the pedestrian. Figure 42 illustrates this complex pedestrian crossing maneuver.



A – H Pedestrian crossing across on/off ramps. 1 – 3 Pedestrian crossing across cross road (shown below).

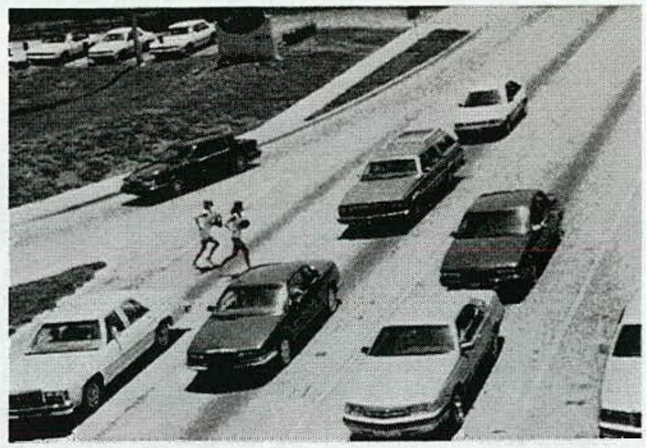


Figure 42. Potential pedestrian crossing maneuvers of ramps and cross street.

When an actuated pedestrian phase is included at a SPUI, the phase is typically assigned to either NEMA movement number four or eight, depending on the crossing direction. This technique has the advantage of providing some concurrent vehicular traffic service when the pedestrian phase is called; particularly, if a pedestrian actuation is received on only one approach. In this situation, the adjacent off-ramp left-turn movement can also be served because it does not conflict with the pedestrian movement.

In spite of the "overlap" capability of an actuated pedestrian phase, the pedestrian phase is still inefficient in terms of the total number of persons, i.e., motorists and pedestrians, served during each signal cycle. This is due primarily to the typically low ratio of pedestrian demand to phase time required to serve this demand and the large size of the SPUI. In general, the excessive width of the cross road dictates relatively long clearance interval durations for the pedestrian phase. Similar to the clearance intervals used for vehicular phases, clearance intervals represent inefficient use of cycle time because flow rates are lower than their maximum values. Because of this poor utilization, any significant amount of pedestrian activity can have a large negative impact on the total person-serving capacity of the SPUI. Pedestrian impacts at a SPUI are more severe than at a tight urban diamond, or TUDI, where through phases are provided in a four-phase sequence which provides some protected pedestrian service across each roadway.

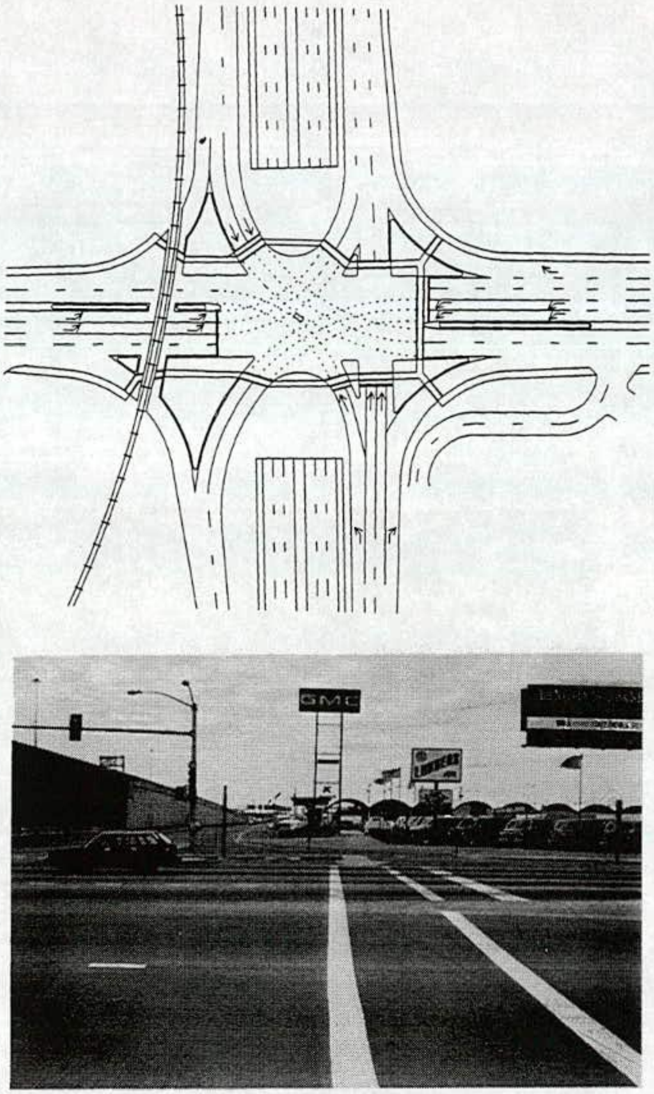


Figure 43. Signalized pedestrian crossings.

Almost without exception, crosswalks were not provided for pedestrians to cross the cross road at the 36 SPUIs studied. This tendency applied equally to SPUIs with and without frontage roads. This finding was not surprising for those SPUIs without frontage roads because of the incompatibility of the signal phasing with a cross road pedestrian movement. It is noted that a recently completed basic SPUI, not in the 1989 field survey, does have pedestrian crosswalks across the cross road. It was expected that pedestrian crossings would be more prevalent at the SPUIs with frontage roads because of the compatibility between the frontage road traffic and pedestrian phases. Contrary to this expectation, only one of the seven operational SPUIs with one-way frontage roads was found to have a pedestrian crosswalk. This particular SPUI is shown in Figure 43.

Other potential solutions to the pedestrian crossing problem may be considered. One costly design is a pedestrian/bicycle overpass located nearby the grade separation structure. One new SPUI located near a school does include a nearby pedestrian overpass. If pedestrians are expected and can not be economically accommodated at the SPUI, pedestrian crossings should be provided at the adjacent intersections along the cross road.

In the case of pedestrians crossing the on-ramp and off-ramp pair on either side of the cross road, one concern is that the on-ramp and off-ramp right-turn movements are not signalized. Providing pedestrian actuated phases to cross these movements would require signalizing them which, in turn, would reduce the capacity of the SPUI. Signalizing the ramp right turn movements would likely reduce the right turn capacity to that available under YIELD control. A second impact pertains to the signalization of the off-ramp right-turn movement. When this movement is signalized, it increases the control area of the interchange and necessitates longer clearance intervals.

Another potential problem for pedestrians crossing the on- and off-ramp pairs stems from the physical expanse of the SPUI. In general, the SPUI is so wide that it cannot be crossed in one signal cycle. During a typical crossing maneuver (see Figure 42), the pedestrian first crosses the yielding off-ramp right-turn movement. Next, he or she crosses the off ramp during either the cross road left turn or through phase. Next, he or she crosses the on-ramp during the cross road through phase. The off-ramp left-turn phase may have U-turn traffic which would affect the safe use of this phase by pedestrians. Finally, the pedestrian crosses the last yielding on-ramp right-turn movement. The entire crossing maneuver usually covers about 400 ft and can easily take 3 min or more.

Most of the SPUIs surveyed had sidewalks and painted crosswalks for pedestrians crossing the on- and off-ramps parallel to the cross road. Observation of pedestrian behavior at these SPUIs indicated that pedestrians were able to cross the ramp junctions safely and with little confusion as to when it was safe to cross during the cycle. Full interchange illumination is a major aid to pedestrians.

Conflicts and Erratic Maneuvers

During the course of the field study, several vehicle conflicts and erratic maneuvers were observed. Many of these were random events, apparently unrelated to the geometric design or operation of the SPUI. On the other hand, some were observed repeatedly and appeared to be related to the design parameters of the SPUI. The following discussion highlights the conflicts and erratic maneuvers that were observed during the field survey.

One particularly frequent conflict observed at several SPUIs was between the clearing and entering vehicles of successive phases. This conflict stemmed from extended use of the yellow interval by clearing vehicles rather than early start-up of entry vehicles. The likely cause of this conflict is an inadequate all-red clearance interval at the end of the phase. In fact, in almost every instance where this conflict was observed, the all-red interval provided was shorter than the actual time taken to clear the interchange. Fortunately, the clearing vehicle almost always entered the intersection before the end of the yellow change interval and thus had the remainder of this interval in addition to the red interval to clear the intersection. However, when the clearing vehicle entered the interchange after the end of the yellow interval, the potential for a conflict with entering traffic was high.

Although conflicts were observed between all movement pairs, conflict was most often observed between the clearing off-ramp and entering cross-road left-turn movements. The reason for this frequency can be explained by the location of the conflict point for the two left-turn movements and the duration of the clearance interval. This pair of left-turn conflicts is unique in that it combines a long clearance distance with a short entering distance which creates the longest time separation requirement in the SPUI signal phase sequence. At many SPUIs the clearance interval provided for the clearing movement is less than the time separation needed. In fact, the clearance intervals provided at many SPUIs are often less than that needed for traffic to clear the interchange.

Another type of conflict observed at several SPUIs was between off-ramp right-turn and cross-road through traffic. In particular, the off-ramp right-turn maneuver, albeit a difficult maneuver, is more difficult at a SPUI because of its geometry and travel paths. The safety of the entrance maneuver requires the right turning driver to monitor (over the left shoulder) two different sources of conflicting traffic, to negotiate a curve to the right, and to monitor traffic conditions downstream on the cross road (directly ahead). It is likely that these complications contributed to the frequent conflicts observed between the off-ramp right turn and cross-road through movements.

The off-ramp right-turn maneuver is made more complicated when the next downstream intersection is relatively near to the off-ramp right-turn entrance point on the cross road. In particular, the safety of the right-turn maneuver is reduced when traffic queues from the downstream intersection back up to near the entrance point, when excessive weaving activity on the cross road restricts the capacity of the entrance point, and when a high percentage of right turning drivers desire to turn left at the downstream intersection (thereby necessitating a large amount of lane-changing activity).

One frequently observed erratic maneuver involved the cross-road left-turn maneuver. At many basic SPUIs observed during the field study, drivers were observed turning left from the cross road through lane rather than from the left turn bay. In particular, these left turning drivers were observed to stay in the inside through lane while traveling through the interchange until reaching a point near the far side, left-turn on-ramp entrance and past the small center median island. At this point, these drivers abruptly turned left from the median through lane across oncoming through traffic. It is possible that these drivers were confused by the SPUI's design and were behaving as if the interchange had a more typical diamond configuration wherein they would make their left turn beyond the overpass structure. Truly, left turns can be physically made two ways from the cross road at basic SPUIs. When frontage roads are present, even the off-ramp left turns can and sometimes are made two ways by motorists. While this maneuver seems to be a systemic problem, it is exacerbated by restricted sight distance, the use of lower class signage on the cross road, and traffic control plans used during construction.

This erratic maneuver was observed at some sites more frequently than once per hour. Moreover, this frequency was observed at SPUIs that had been in operation for 15 years as well as those open only a few months. Based on these observations, it might be concluded that complete driver acclimation to this design will only come about with an increase in its application (on a regional and national basis). In the interim, increased use of overhead advance guide and lane use signing together with a systems approach to design are the best means to improve traffic operations.

Another unusual maneuver observed in the survey was when drivers stopped well beyond the stop line. On several occasions during the field study, drivers were observed to stop as much as a full car-length beyond the stop line. This maneuver could be categorized as an erratic maneuver, particularly when the stopped vehicle interferes with the passage of crossing traffic.

The reason drivers stop past the stop line is presumed to be because of the SPUI's unusual design and large conflict area. Drivers approaching the interchange may be uncertain of the SPUI's operation and, thus, may be unable to identify conflicting traffic streams and the most appropriate place to stop. Another factor that may contribute to this response is the nontypical mounting locations for the signal heads. Signal heads are normally located on the far side of an intersection; however, the signal heads at most overpass SPUIs are mounted on the near side bridge face. As a result, drivers approaching an overpass SPUI are confronted with an unusual signal arrangement that may lead to incorrect judgments about the location of the conflict area and the appropriate stopping point. Use of wide stop bars would be expected to improve this behavior.

Accident Experience

The safety of any roadway element, including a SPUI, is difficult to assess. Conflict studies are often used in place of accident records to measure safety. Observation of conflicts at several SPUIs seems to indicate that the design presently may experience some systemic erratic traffic behavior and vehicle conflicts while reducing or eliminating others. This section provides accident histories at the few SPUIs for which accident data were available during the survey.

Accident data for five SPUIs were examined to evaluate their operational safety. The annual accident rates found from this examination, given in Table 4, indicate some variability among sites that is characteristic of the random nature of accidents. For comparative purposes, a typical, signalized at-grade intersection has an annual rate of about 1.2 accidents per million entering vehicles (*20*). With the exception of SPUI 1, it would appear that the SPUI design does not lead to a higher number of accidents. A closer examination of the left-turn-related accident data did not indicate that a safety problem existed.

A review of the accident data at SPUI 1 provided limited insight into the high accident rate at this location. Based on this review, it was found that the most common type of accident was a rear-end accident on the off ramps. Further field examination

Table 4. Annual accident rates collected in survey.

SPUI	Type	Annual Accident Rate
1	Overpass	2.70
2	Overpass	0.88
3	Overpass	1.35
4	Underpass	0.64
5	Overpass	1.80

Notes:

1 - Rates are annual accidents per million entering vehicles.

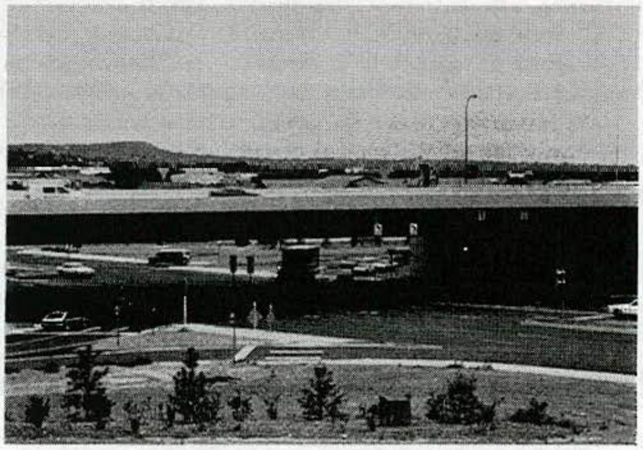


Figure 44. Traffic control device applications at overpass SPUI.

Figure 45. Traffic control device applications at underpass SPUI.

of this site indicates that there may be two factors contributing to this particular accident pattern. The first is that the left and right turn lanes are not exclusive at the two-lane off ramp. As a result, there is a high degree of interaction between the left and right turns. This friction between off-ramp movements stems from the differences in operation and control of the respective movements.

Another contributing factor to the higher accident rate at the SPUI 1 may have been the existence of a relatively close (about 425 ft) downstream intersection. During the field study, traffic was frequently observed to spill back from the downstream intersection and disrupt the flow of off-ramp right-turn movement. Because this spillback occurred randomly from cycle to cycle, it was unexpected by off-ramp drivers and frequently led to conflict when vehicles behind were following too close. This interchange is currently undergoing redesign to address this issue.

A SPUI study in Utah (*21*) examined their brief accident history with SPUIs and TUDIs. The study found that accident frequency and severity probably would be less in Utah at a SPUI than for a comparable TUDI for equal volumes of traffic.

Assessment of Pedestrian and Safety Issues

In general, the geometric design of the SPUI is "new" to many drivers and pedestrians. This is evidenced primarily by the conflicts and erratic maneuvers that may stem from driver misunderstanding of the SPUI design and operation. Because pedestrians are not fully accommodated at many SPUIs, an accurate assessment of pedestrian behavior was not possible. Pedestrians crossing the on-and-off ramps appeared to complete their passage safely. Crossing the surface arterial at the SPUI is difficult and should be encouraged to be made at the next signalized intersection by providing crosswalks with pedestrian signals. Good interchange visibility and illumination are recommended enhancements for pedestrian traffic.

The accident histories of several SPUIs indicates that a high proportion of accidents occurs during right turn entry from the off ramps onto the cross road. There are several possible explanations for this finding; namely, the three-phase signalization, limited merge entry design, and the existence of a relatively close downstream signalized intersection on the cross road. Traffic interaction between the SPUI and downstream intersection, such as queue spillback and constrained weaving, tends to induce unexpected conflicts at the off-ramp right-turn entrance point to the cross road. In spite of the observed conflicts and erratic maneuvers, available accident data suggest that a modern SPUI design is as safe as a signalized at-grade intersection or TUDI interchange operating at the same volume levels.

TRAFFIC CONTROL DEVICE APPLICATIONS

Traffic control devices are all signs, signals, markings, and other devices used at SPUIs to regulate, warn, and guide motorists, particularly unfamiliar motorists, safely and efficiently through the interchange. In addition, roadway lighting usually has been provided to enhance the general visibility of the area. Engineers have recognized the relatively high information requirements at SPUIs in their overall traffic control plans.

The field survey conducted in the summer of 1989 provided a wealth of data on the types and applications of traffic control devices at SPUIs. The following is an overview of those systems observed and a general appraisal of their applications. In all applications, an important determinant is whether the main highway is provided a grade-separated overpass or underpass. This attribute affects the design of the signing, signal system, and the need for special roadway lighting of the cross road.

The application of traffic control devices at an overpass SPUI is illustrated in Figure 44 for a new SPUI. Pavement markings are used to delineate lane lines, turning paths, stop lines and pedestrian cross walks. Various regulatory, warning, and guide signs are used to properly inform the motorist of the desired path and actions. Finally, general area and spot roadway lighting are provided, together with supplemental lighting provided underneath the bridge and on its support walls, to further enhance visibility within the interchange and connecting highways.

Similar traffic control device applications to an underpass SPUI recently constructed are shown in Figure 45. Here, it is apparent that the general visibility of the intersection conflict area is superior and much different from that of the overpass design. Establishing a clear definition of the intersection location is a desired design objective as the cross street left turns have to be initiated well in advance of the traffic signals.

Interchanges having one-way frontage roads are further complicated by their larger size and increased alternative routes. All are overpass designs. Figure 46(a) illustrates traffic control device applications on a frontage road approach to a new SPUI and Figure 46(b) shows its continuation outbound of the intersection. In this case, the frontage road traffic signals were span-wire mounted and connected to the bridge face. Pole-mounted span-wire configurations were observed to be more typical for this case in other states.

Pavement Markings

The field survey revealed that pavement markings tend to be conventional for interchanges in the locale except, in some cases, for the central intersection area of the SPUI. In the central area where wear and general visibility are a special concern, a wide variety of pavement markings were observed, including traffic paint, cold-rolled plastic tape, thermoplastic, and sometimes even special light fixtures. Lane tracking lines in the central area are usually provided to better define elongated left turn paths, as shown in Figures 47(a) and 47(b). In a few cases in the South where snow removal is not a problem, raised pavement markers

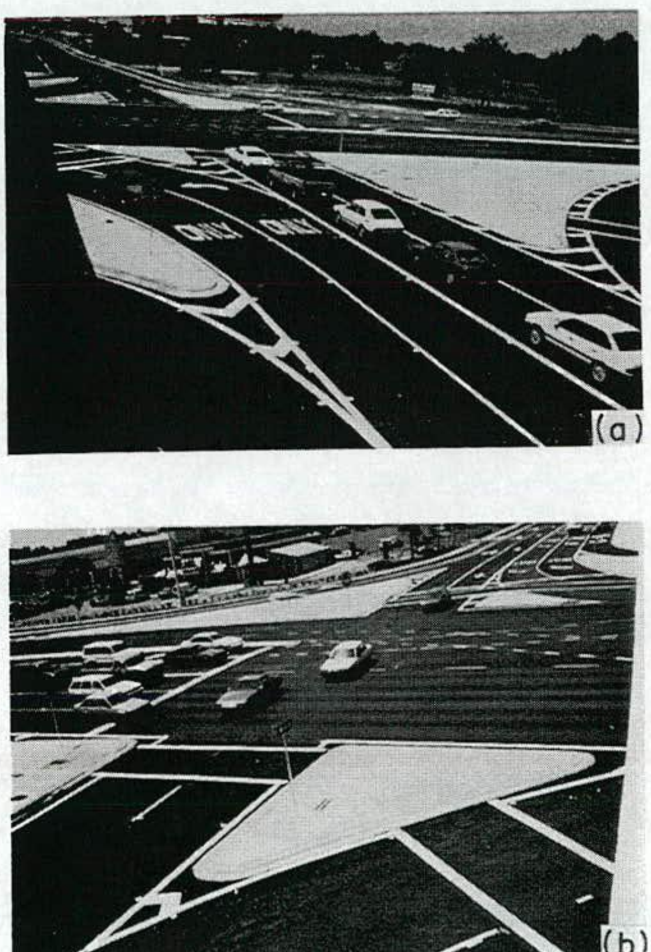


Figure 46. Traffic control device applications at overpass SPUI with one-way frontage roads: (a) frontage road approach; (b) frontage road departure.

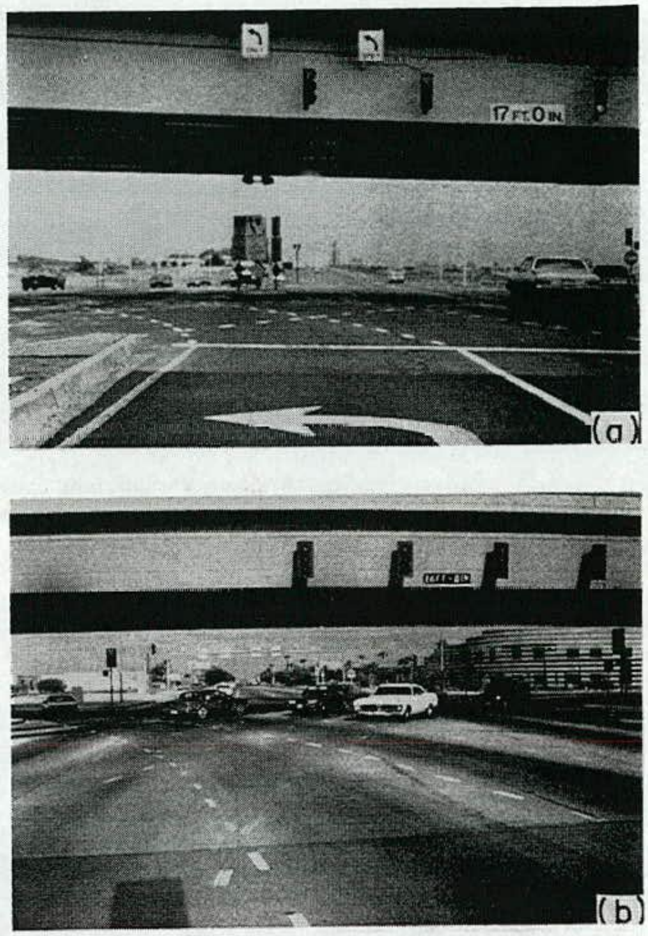


Figure 47. Pavement marking applications: (a) with center island; (b) without center island.

were added to enhance visibility in rainy weather, as depicted in Figure 48(a).

In two of the earliest SPUI designs, airport runway lights were installed, perhaps raised retroreflective pavement markers were not available then, to delineate the central area lane lines. Figure 48(b) shows the runway markers lights installed in an early site. Further design details of the runway markers lights are provided later. These novel lights are a maintenance problem and do not appear to be cost effective.

In a few cases, portland cement concrete pavements were used in the intersection area underneath the overpass bridge in lieu of the asphalt pavement found along the cross streets (see Figure 47(b)). Concrete pavement provides some enhanced visibility and brightness in the bridge shadow cast on the cross street. Perhaps reduced future street maintenance was an additional consideration for using concrete pavements, especially where runway lights are used.

Traffic Signs

This section addresses some of the signing issues found somewhat unique to or needing special attention for successful SPUI operations. A large variety of traffic signs are required at a SPUI to provide sufficient regulatory, warning, and guidance

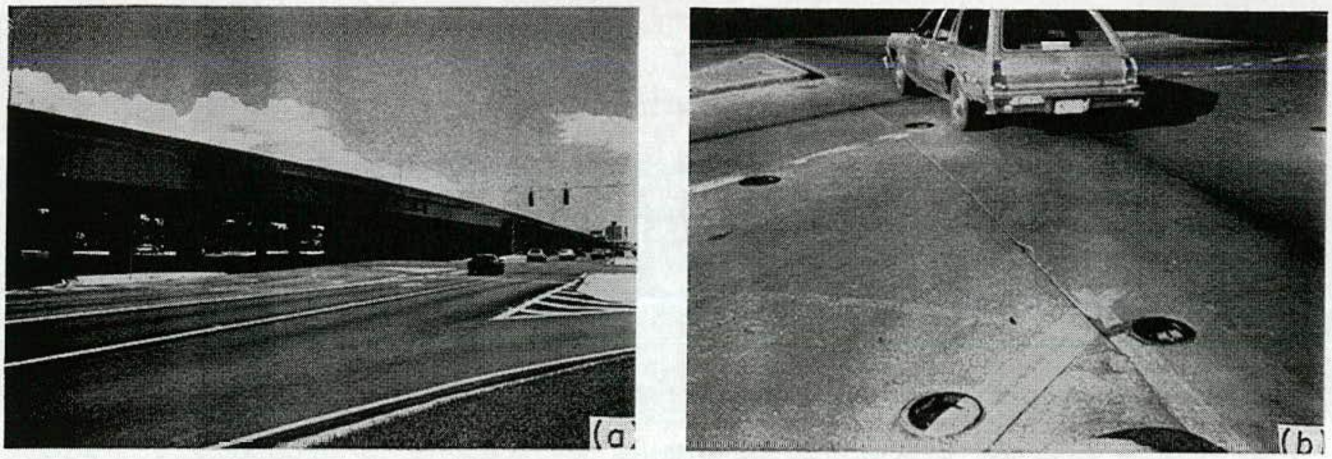


Figure 48. Advanced pavement marking applications: (a) raised pavement markers; (b) runway marker lights.

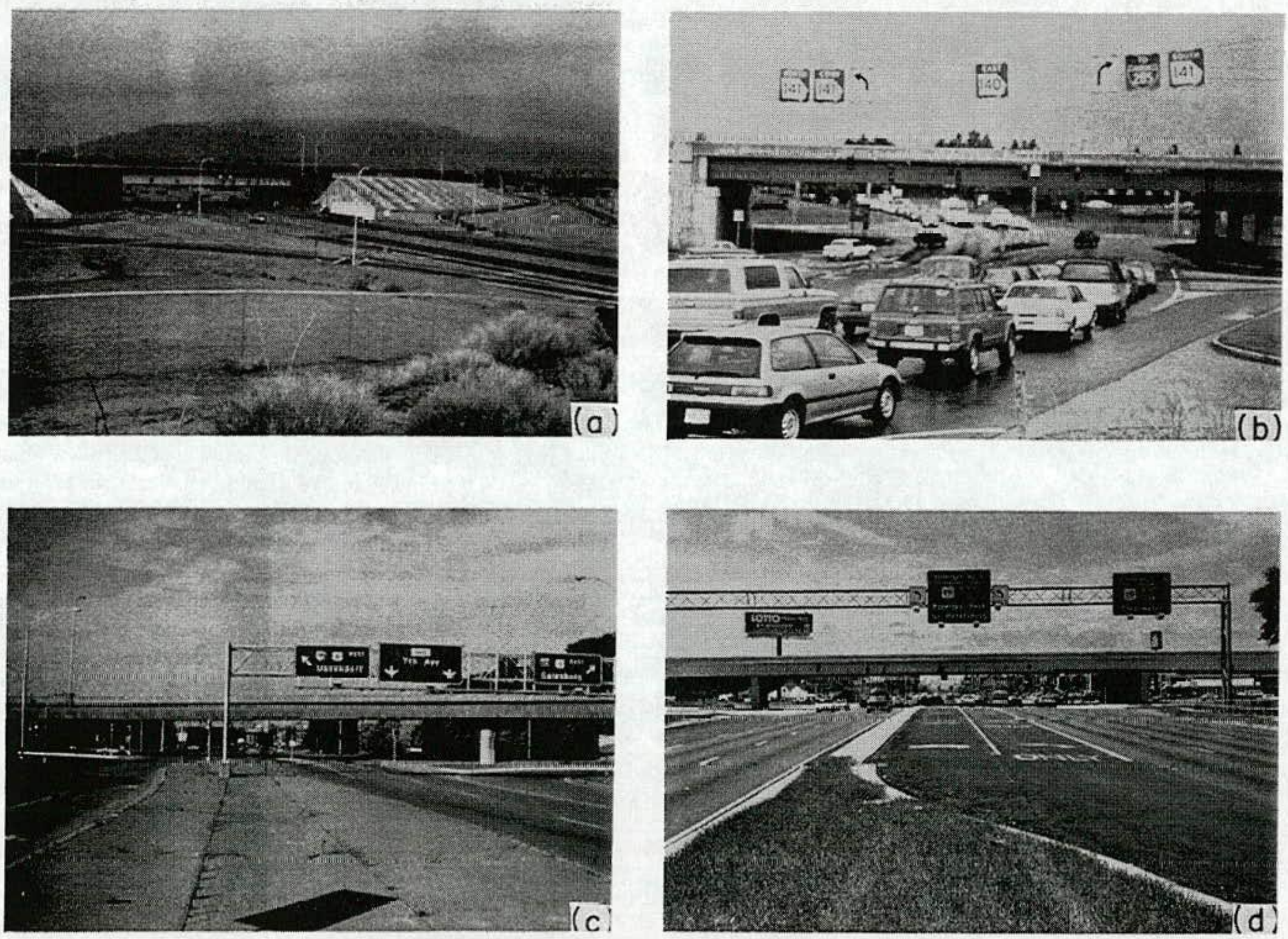


Figure 49. Examples of guide signing applications on cross street: (a) in developing rural area; (b) on suburban arterial; (c) on major urban arterial; (d) high-speed major urban arterial.

information needed for motorists to safely and efficiently travel through this interchange. In general, more and larger signs are needed if both interchanging highways are numbered routes and traffic speeds are high.

Overpass designs produce distinct signing requirements but provide convenient sign placement along the cross street. As many of the previous figures show, regulatory signs defining allowable turning paths and sometimes even route direction guide signs are placed overhead between or above the cross street traffic signals on the outer beam of the overpass bridge. This is a common SPUI practice where the bridge height is acceptable, properly located, and the signs and related traffic signals have been incorporated early in the bridge design process.

Guide signing is a critical element in the traffic control plan. Overhead versus ground-mounted guide signing on the approaches to the SPUI is a major design issue, given due consideration of information needs, approach visibility, costs, and aesthetics. The traffic engineer is faced with a continuum of possible situations from low-volume nearly rural sites, as depicted in Figure 49(a), to intermediate-volume suburban arterials, as noted in Figure 49(b), to high-volume major urban arterials combined with state highways, as shown in Figure 49(c) and Figure 49(d). As can be seen in the figures, ground-mounted advanced guide signing is used in the rural site, transitioning into overhead span-wire guide signing on the narrow urban arterial, and finally transitioning into interchange-level overhead guide signing on the high-volume urban arterials combined with state and interstate highways.

Operational problems were noted at several overpass SPUIs widespread around the country where arriving cross street motorists failed to initiate their left turn at the left turn bay prior to reaching the overpass, but made their left turn from the through lane either prior to reaching the center island, as shown in Figure 50(a), or more typically after proceeding underneath the bridge and past the center island shown in Figure 50(b). Ground-mounted advance guide signs located on the shoulder appear to be inadequate in providing the information needed for overpass SPUIs located in urban areas. Figures 49(c) and (d) presented more effective overpass signing designs. Provision of interchange level, overhead guide signing is needed in overpass urban SPUI designs having route designations to provide positive guidance.

Design feature rarity usually means that highly visible signage will be needed to achieve effective communication with unfamiliar motorists. SPUIs combined with frontage roads, both of which are uncommon to most motorists, will most likely also need overhead guide signing like that depicted in Figure 51.

Signage of the center island is practiced by most highway agencies, regardless of the SPUI's structural configuration. Of the 36 SPUIs, 20 SPUIs had post-mounted KEEP LEFT signs sometimes combined with object markers. Figure 52(a) illustrates a typical underpass design, and Figure 52(b) presents overpass signing. If a center signal cluster is suspended vertically from the overpass bridge deck as in the latter figure, post-mounted signs are suggested to provide positive guidance around the low signal heads. Some apparent inconsistency in the definition of the desired left turning path may result when using standard regulatory guidance signs, as can be seen in Figure 47(a).

Some states have elected not to use any center islands with no apparent operational problems, but with the advantage of reduced maintenance if the guide signs are struck by errant vehicles. Clustered center signals are not used in this case. As examples of the no center island design, see Figure 45 and Figure 47(b) for underpass designs and overpass designs, respectively. Designers have taken special care to ensure that adequate night-time visibility is provided in their designs.

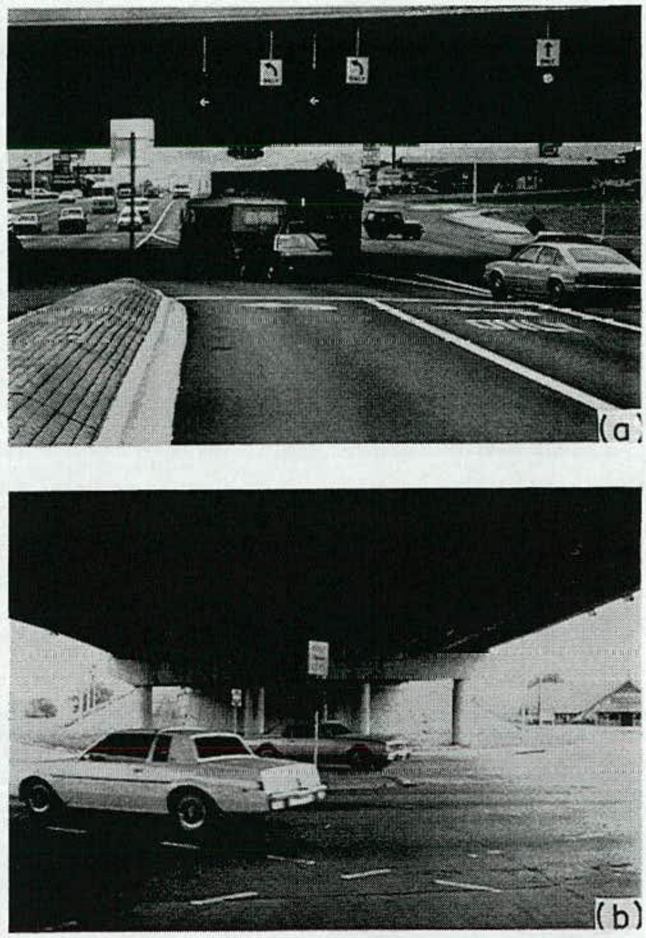


Figure 50. Traffic operational problems: (a) turns from median through lane; (b) delayed turns from median lane.

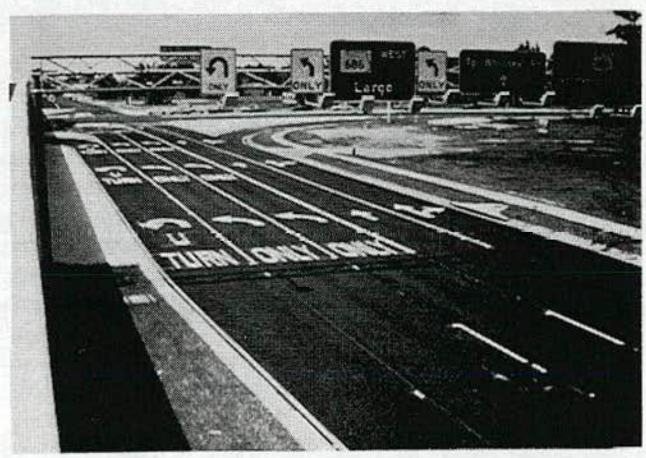


Figure 51. Example of frontage road guide signing application.

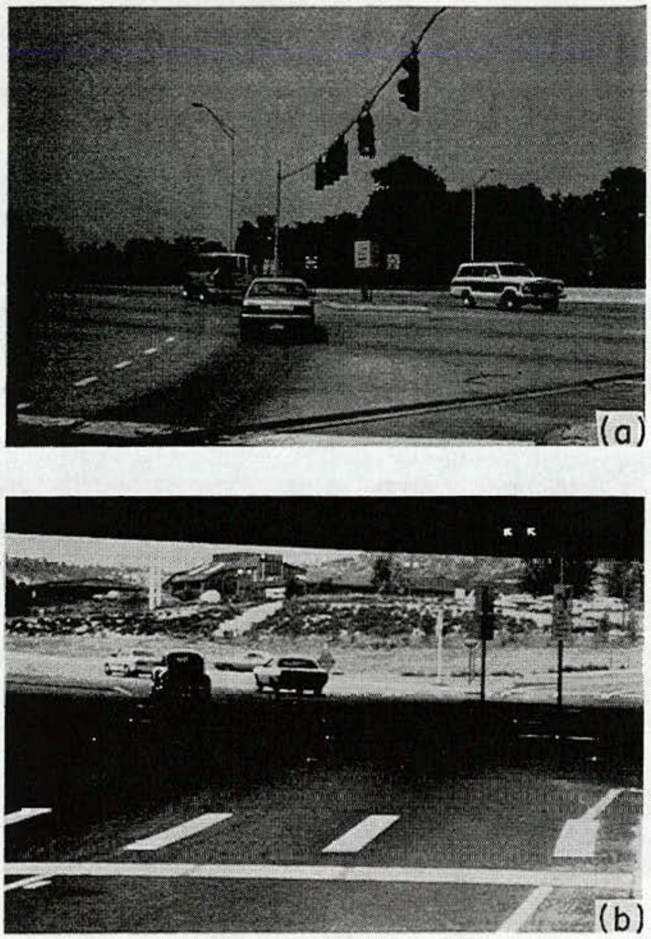


Figure 52. Examples of signing of center island used at some SPUIs: (a) underpass design; (b) overpass design.

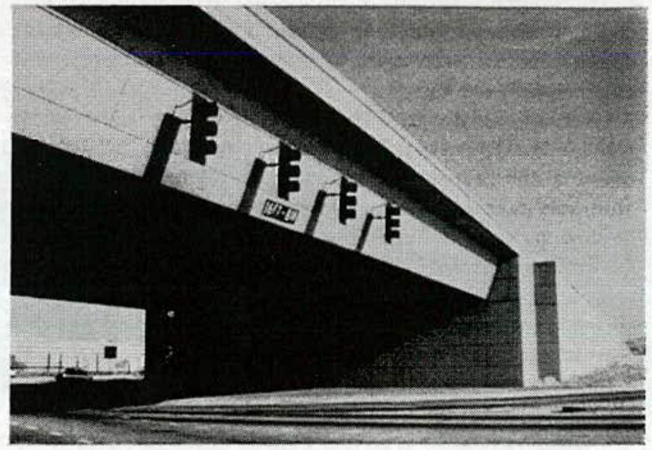


Figure 53. Example of traffic signal installation on bridge.

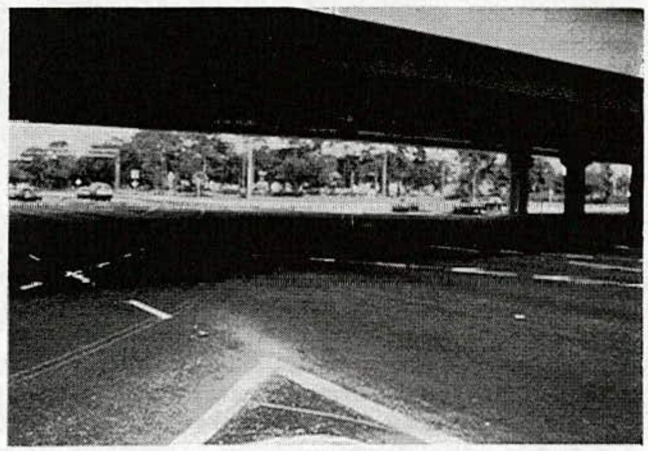


Figure 54. Traffic signal cluster vertically hung over a center island.

Traffic Signals

The types of traffic signals used are driven by the type of SPUI structure and the type of traffic signal phasing used. Signal requirements also vary by approach road. Because most basic SPUIs use a three-phase system having protected left turns and advance left turn bays on the cross street, separate left turn traffic signals are required in advance of the interchange on the cross street. These signals must be 12-in. signals and be clearly visible to oncoming minor road traffic. For overpass SPUIs without frontage roads, the cross street traffic signals are usually placed on the side of overpass bridges, as shown in Figure 53. The signals are either hung from the bridge deck, or affixed to the side of the bridge. Field observations revealed several cases where the traffic signals were mounted too high above the cross street where large SPUIs had been built. Early coordination with bridge design is desired if the signals are to be placed on the bridge itself. The survey revealed that span-wire mounting of cross street traffic signals is commonly used with frontage roads. Steel poles with mast arms have also been used, depending on local practice.

Overpass structures having simple spans are usually designed too low to vertically mount signals underneath the bridge and still maintain a 15-ft minimum signal clearance height. This minimum clearance would require at least a 19-ft minimum bridge height. Most single-span bridges are about 17 ft, as depicted in Figure 47(a). Selection of a vertically mounted signal cluster design over a trapezoidal center island, shown in Figure 54, permits the lower bridge height of about 17 ft. One state used horizontally mounted signals under the bridge in one economically restricted design. Exterior pedestal or post-mounted signals are normally employed where the cluster signal is not used, although variations were noted in the field survey. Decisions on the central area signal should be made early in the design process.

Provision of traffic signals adequately visible to the off-ramp left-turn traffic is a very critical design task. The task is complicated in overpass designs by the basic SPUI layout and bridge location. Figures 55(a) and 55(b) illustrate two designs with signal mounting on the bridge siding. General visibility is further complicated when the freeway overpass is also elevated, further restricting visibility and promoting higher ramp approach speeds, as illustrated in Figure 55(b). Some overpass sites also have a post-mounted signal in the off-ramp divisional island (see Figure 55(b)) to provide advance visibility, which is sometimes supplemented by a complementary far side "pull-thru" signal backed onto the opposing left turn approach signal. The far side signal would appear warranted if no center-mounted cluster signal is used as motorists' may otherwise lose sight of critical signal indications while traveling into the central intersection area.

Multispan bridges can reduce the geometric restrictions and greatly increase general visibility in the area. In addition, they can be physically separated to further improve geometrics should the required right-of-way be available. Figure 56(a) shows a more conventional example in mountainous terrain and Figure 56(b) shows a true "X" SPUI design in an urban area. Traffic signal placement for the off ramps can be more varied with multispan bridges, but still remains an issue.

Signalization of underpass SPUIs is different from that for overpasses. Ramp speeds are usually much lower and general spatial visibility much higher. However, motorists' determination of the desired turning paths and exit ramp locations can be a problem. Therefore, placement of the signals over their related turning paths is desired. Proper placement can be a challenge because most underpass designs use simple span wires for hanging the signals over the central intersection area, as shown in Figure 57(a). Figure 57(b) shows an alternative design using a small truss. Steel poles with mast arms also have been used as well as have monotube designs.

ROADWAY LIGHTING

Roadway lighting is frequently provided at SPUIs to aid in revealing its geometric features and operational requirements. Roadway lighting usually can be classified as being either a continuous lighting or an area lighting system. Sometimes spot or safety lighting systems are also used to specifically light a point such as a ramp merge. The field survey revealed that continuous roadway lighting usually was already existing on the

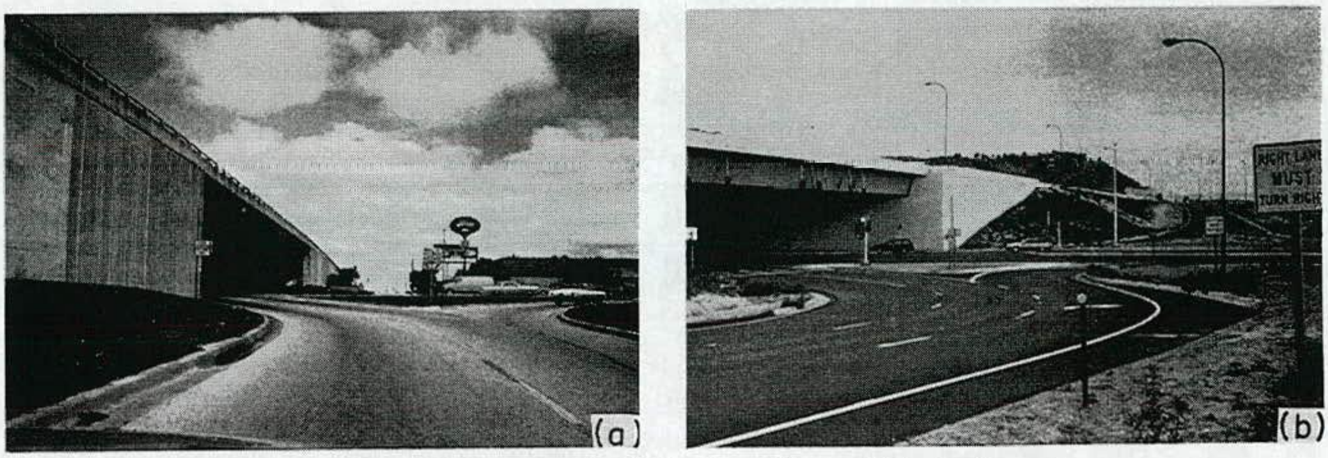


Figure 55. Examples of traffic signals on off ramp: (a) level exit ramp; (b) downgrade exit ramp.

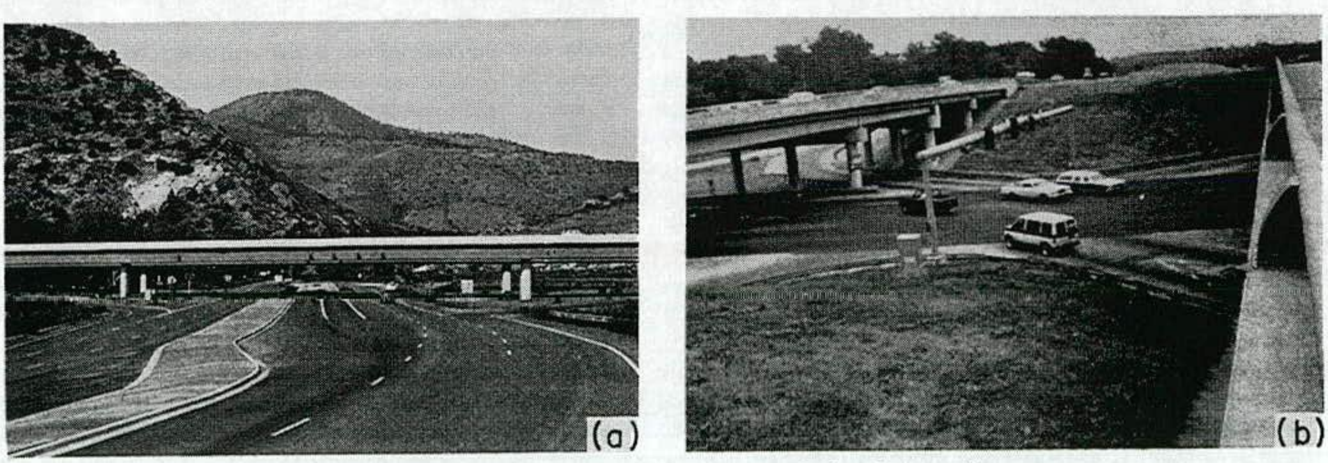


Figure 56. Examples of multispan overpass bridge designs: (a) multispan design in mountainous terrain; (b) multispan "X" design in urban area.

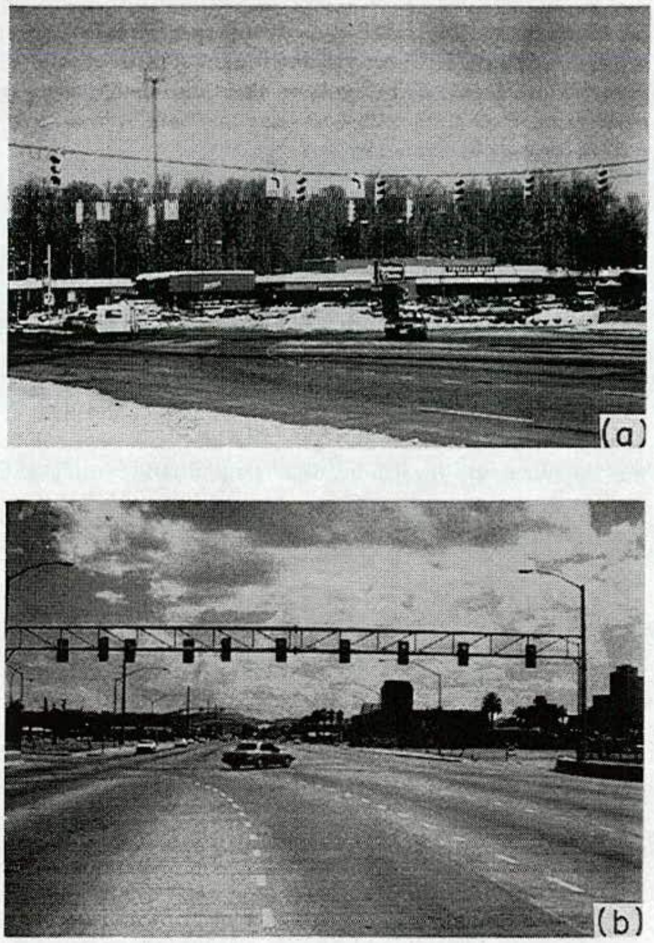


Figure 57. Examples of traffic signal placement for underpass designs: (a) simple span wire design; (b) small steel truss design.

major and minor highways intersecting at new SPUIs, although not always, primarily depending on the existing lighting policies of the local state and city highway agencies. Lighting also must include consideration of the lighting requirements underneath the overpass bridge structures as this otherwise darkened area is the most critical conflict zone of the SPUI and good visibility here is desired.

The field survey revealed that roadway lighting was often used to enhance the nighttime visibility of the SPUI. Of the 36 SPUIs observed in the survey, 32 had some form of fixed roadway lighting. One state did not use freeway or interchange roadway lighting in urban areas. In some cases, the interchange lighting was simply a continuation of the existing main road and cross road continuous lighting system through the interchange area. In other cases, a detailed interchange area lighting plan was prepared that considered the specific lighting requirements of the proposed SPUI together with needed compatibility with the existing street lighting systems, if any. Figure 49(a) presents such a lighting system. High-mast tower lighting was also used in a few cases where a new expressway overpassed the cross street.

Overpass SPUIs were observed to have the highest lighting requirements. Dark shadows were sometimes noted underneath the bridge at night, hiding critical details of the central area, notably the center island and related signing. This problem appeared to be due to original lighting design oversights together with inadequate lighting maintenance. In addition, another lighting uniformity problem observed at many SPUIs was the occurrence of hot spots and direct glare arising from the use of overly strong "wall pack" lighting units mounted vertically nearby on interior bridge headwalls or bridge bents (piers). Wall packs are normally used to light under conventional overpasses, as well as SPUIs, for drivers and pedestrians. In normal use, there are no left turn ramp maneuvers entering in the middle of the interchange; therefore, direct glare to oncoming motorists is not a problem. However, in SPUI designs potential direct glare problems may arise where wall pack lights are used to provide illumination for pedestrians. A softer lighting source should be used.

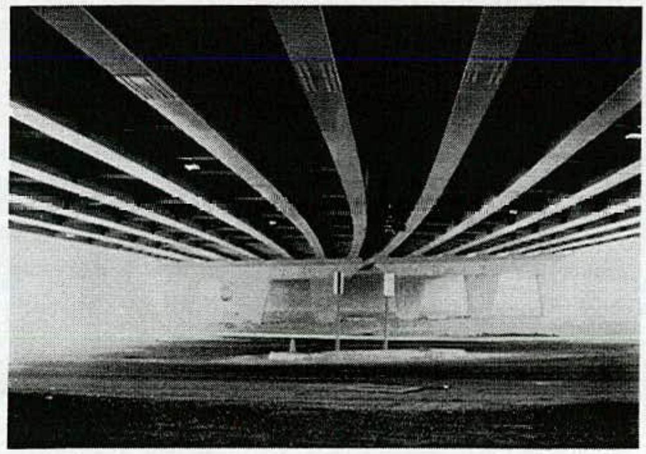


Figure 58. The dark spot area underneath overpass with typical overhead lighting fixtures.

The field survey revealed that area lighting underneath the bridge structure was provided at most interchanges. Typical lighting fixtures are shown in Figure 58. Levels of horizontal illumination were measured at ten sites at night using a modern digital photometer. Average maintained levels of horizontal illumination of 1 to 2 footcandles were recorded in the central area with hot spot values caused by wall packs exceeding 5 foot-candles (ft-c) not uncommon at the pavement edge. Uniformity of illumination, both horizontally and vertically, was judged as being an important design criterion.

Perusal of those overpasses not provided with general area lighting revealed that these SPUIs generally had open, multispan bridges wherein the adjacent ambient lighting was expected to provide the desired illumination. Wall packs were used to provide supplemental lighting in all cases. In most cases, overhead lighting was also provided under the bridge. Again, reliance on the wall pack units to provide central area illumination often produced hot spots, glare, and a poorly illuminated conflict area.

Underpass designs were also generally illuminated, but again not always. Good practice would suggest that the SPUI should be illuminated to the same or higher levels as the connecting highways. In addition, the exiting roadways should be readily visible to arriving and departing vehicles. Any serious limitation of geometric features promotes the need for area illumination. Use of curbed islands or elevated cross road overpasses likewise merits use of area illumination.

PAVEMENT MARKING LIGHTS

A novel feature at some SPUIs is the use of Federal Aviation Administration (FAA) approved airport runway lights embedded in the pavement to delineate lane lines through the central intersection area. Delineation of left turn paths has been the primary emphasis. The lane lights turn on when the related movement's green is on and turn off, some systems sequentially, when the phase ends. No studies have been reported that document the relative effectiveness of the marking lights, although new and well-maintained systems do appear to provide positive guidance.

The first two SPUIs believed to have been built in the United States had runway lights installed in them with no proof that they were needed or would be effective. Anxiety about the uncertain future operation of motorists passing through untried SPUIs was given as the main reason for their initial installation in one state, shown in Figure 48(b), given that they were being installed in another. The field survey revealed that no SPUI has ever had pavement marking lights installed after the SPUI was opened to traffic to treat an observed operational problem. In fact, the reverse trend is more evident. Of the 7 lane light systems installed on the 36 SPUIs opened by the summer of 1989, only 3 were still fully operational. However, the original system in Clearwater, Florida, has been replaced with a new system. Operational systems are installed in Colorado (see Figure 52(b), Florida, and Georgia. In the latter state, external interchange illumination is not used as a matter of policy.

Individual locales that have begun to use the runway lights may find themselves in a precedent setting situation where they believe local motorists expect the runway lights to be used. While this low-level expectancy may exist in the immediate vicinity of the SPUI, four cases are known where the original marking system decayed to extinction and no apparent traffic problems resulted.

All pavement marker lane delineation systems used versions of FAA's standard runway marker lights. Figure 59 displays a unit installed in the pavement. The lane light assembly consists of three basic parts. Fitting flush with the pavement surface is a lampholder assembly 10 in. in diameter that holds a 65-W tungsten-halogen prefocused lamp having a rated life of 1000 hours with green lens screen, also available in red or yellow, but not white (see FAA advisory circular AC 140/5345-46A and specification L-852). Note that the green color does not meet MUTCD standards for white lane markings. The lampholder fits on a mounting base 18 in. deep buried beneath the pavement (see FAA specification L-868). Enclosed in the base is a 120/10 V transformer and related conductors to operate the lamp (see FAA advisory circular AC 150/5345-47). Openings in the base permit all lamp wiring underground to a common service box. Georgia leaves the transformers in the base, whereas recent Colorado designs have moved the transformers to a separate service cabinet next to the traffic signal cabinet. In addition to improved maintenance, moving the transformers to roadside cabinets may reduce the need for extensive pavement coring to install the lights. A typical SPUI installation has about 50 lights embedded in the pavement with a large supply of spares. Recent total job costs were $87,000 and $104,500 installed.

Operational reliability and maintenance of the lane lights are of primary concern. Most early users had difficulty keeping their pavement lights operating for a variety of unexpected reasons. Moisture invasion into the recessed lens was and continues to be a constant problem. However, recent solid-state designs for wattage control and timing have improved overall operational reliability. These newer designs provide a small drip current (about 12 W) in the lamps to keep them warm and dry when the signal phase is red. This small current will not make the lights visible, however.

Figure 59. FAA standard runway marker lights used for lane lights.

Another major maintenance problem still persists. Pavement debris routinely covers may lights due in part to the recessed design (to protect from airport snowplows) of the FAA-specification lampholder. In addition to traffic-carried debris that gets into the pockets, street sweeper passage is reported to routinely fill the lens cavities. Motorists are annoyed by this continued problem. Snow cover and snowplowing operations are also other considerations in northern climates. Any paving overlay would create large potholes over the lights that would further pond water. Thus, providing diligent maintenance of the lights has been difficult and is somewhat hazardous because the markers are firmly embedded in the pavement in the middle of a very busy intersection. Use of concrete pavement in the central area is recommended to provide added pavement stability, pavement life, and nighttime visibility where the lane lights are to be used.

OPERATIONAL SUMMARY

The SPUI is frequently compared with the tight urban diamond interchange (TUDI) and the high-type at-grade intersection (AGI). Much of the SPUI's similarity to the TUDI can be seen in their physical characteristics. In particular, both interchange types use one-way diamond-type ramps in each quadrant to the grade separation structure. Moreover, they both have a similar ability to economize on right-of-way needs along the major road.

In contrast to the physical similarities between the TUDI and the SPUI, the signalization found at most SPUIs is much like that found at high-type AGIs. One example of this similarity is that the SPUI and the high-type AGI both normally use a single traffic-actuated controller to regulate conflicting traffic movements. Another similarity the SPUI and AGI have is in their signal phasing. In most instances, both the AGI and SPUI have dual ring, actuated control logic that provides concurrently timed cross road left turn and through phases in that order.

The following summarizes other relevant findings of SPUI operations based on the field survey conducted in the summer of 1989 and on current literature on the subject:

- SPUIs without frontage roads operate on a basic three-phase signal system. SPUIs with frontage roads operate on a four-phase system.
- Basic SPUI signal phasing is not conducive to pedestrians crossing the cross road as its parent through phase is missing. Only one interchange is known to have fully addressed this issue.
- Protected/permitted left-turn signal operation is not used at SPUIs.
- The large size of the central conflict area requires longer red clearance intervals at SPUIs.
- The average through saturation flow rates from the cross street of a SPUI are the same as those of a TUDI. Left turn operation at a SPUI, particularly for the off ramps, provides higher saturation flow per lane and is more amenable to large truck operations than at a TUDI.
- The capacity of off-ramp right turns at SPUIs can be an unexpected problem. The off-ramp capacity may be only half that of the cross road right turn per lane in high-volume conditions. This right turn's operational efficiency is highly dependent on any nearby downstream intersection.
- It may be relatively difficult to provide a high level of positive guidance along the relatively long, curving travel paths within the SPUI's conflict area because of the heavy, multidirectional use this central area receives.
- Large overhead advance guide and lane-use signing on the cross road and off-ramps may be needed to efficiently guide drivers through some SPUIs. Interchange level signing and illumination are suggested.
- The overall operational safety of a SPUI is at least as good as that of a competitive TUDI.
- The use of novel airport runway lights does not appear to be cost effective. There are long-term maintenance problems associated with their use.

CHAPTER FOUR

SYSTEMS PLANNING AND DESIGN GUIDELINES

Many states in the United States are actively considering the use of SPUIs to address local traffic problems. By the end of 1989 more than 30 were operational and dozens more in design and construction, as Figure 60 illustrates. In Phoenix alone, five new SPUIs have become operational since 1989, with many others still being planned. In other parts of the country, perhaps only one SPUI has been built and highway planners are considering many more in their future plans. As an example, one overpass SPUI was recently completed in Reno, Nevada, and nearly a dozen potential sites have been identified for future implementation in Las Vegas. Some may be overpasses and others underpasses. Other states have no SPUIs whatsoever. Other than their basic policy on geometric design of highways, most states have little planning, design, or operations guidelines on SPUI design. The following chapters provide the desired supplemental material on this new type of efficient urban interchange.

This chapter is directed at providing a strategic planning and design perspective of SPUIs as developed from the field survey results, analytic studies, and salient literature on the subject. Basic application of SPUIs in previous highway projects is described first followed by consideration of the general impacts of traffic growth on traffic demand and the need for design alternatives that provide a sizeable increase in capacity. A critical lane analysis of traffic signal capacity then follows. Finally, a series of summary guidelines are provided to assist planners in testing their designs. The probable ultimate design for a two-level SPUI is identified.

SPUI APPLICATIONS

As noted, major efforts are currently underway to rebuild the national urban highway system and restore its operational quality to desired levels of service. In this regard, several types of operational problems seem to have been addressed when SPUIs were selected as the design interchange form. The following material describes these observed target applications in urban highway project development.

New Freeways

In one notable case, a complete new freeway was built from scratch on new alignment in a highly developed urban area in Arizona. This large project was the completion of the inner loop of the Interstate Highway System and, consequentially, it was not built on existing freeway right-of-way. In addition, there was no through traffic to maintain during construction. Alternate routes were readily available so that construction could proceed at an optimal schedule. In none of the numerous SPUIs built in this project were frontage roads used. In most cases, cast-in-place construction of the underpass and overpass bridges was used. Because mainline alignments were not constrained, other than to satisfy the existing design speed, a wide variety of designs were selected to best fit local conditions. Construction of urban

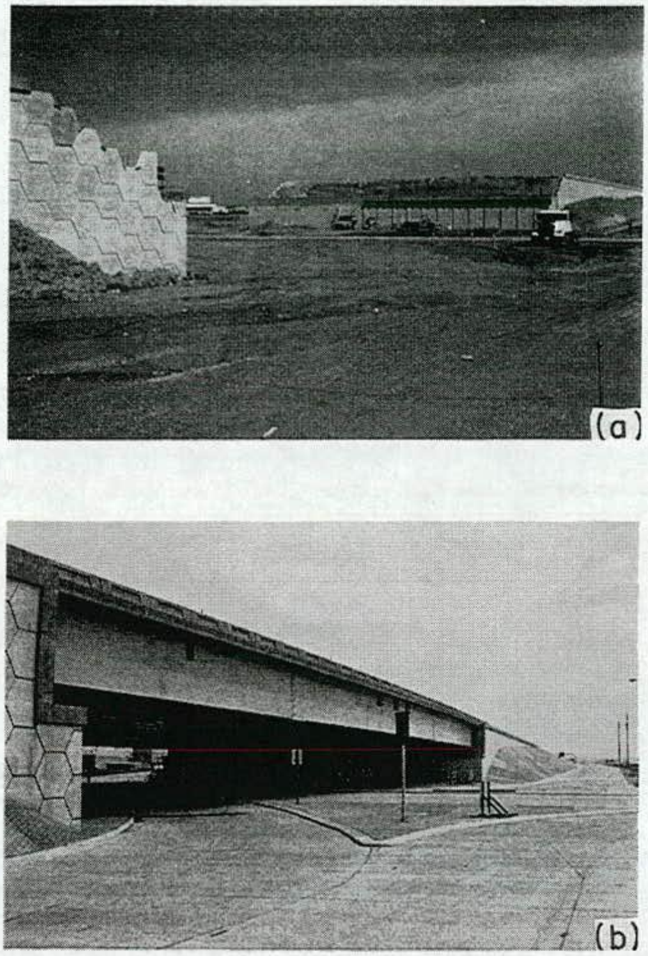


Figure 60. A large overpass SPUI under construction: (a) early construction at site; (b) final construction at site.

freeways on new right-of-way is likely to be a special case in the future.

Freeway Upgrades

In this more common case, SPUIs have been installed as the agency was rehabilitating and rebuilding the Interstate Highway System. The overall plan may have called for added freeway main lanes and new structures with individual capacity upgrades at congested cross arterial interchanges. Design constraints of maintaining existing alignments and working within existing rights-of-way were evident. Right-of-way costs were usually very expensive.

SPUIs were used in freeway upgrade projects under the following conditions where no frontage roads exist. First, SPUIs were used at an existing underpass interchange of an undersized tight urban diamond (TUDI) where a two-span platform bridge can be constructed while maintaining all existing traffic flow. Off-ramp construction can be performed within the existing right-of-way by using retaining walls and soffit-designed left turning ramps at the bridge. A second case sometimes occurred where the freeway was being widened and an undersized cloverleaf interchange was being replaced because of operational problems. A full-sized SPUI having dual, left turn lanes on all approaches was built within the existing right-of-way. Typical examples of freeway upgrade construction using freeway underpass SPUIs are shown in Figure 61.

Converting an existing TUDI freeway overpass structure to a SPUI will be a challenge for several reasons. If a single-span overpass bridge is used, an additional 6 ft or so of vertical grade separation will be required to transition from, perhaps, a 70-ft clear span overpass having 3-ft deep beams, to a 200-ft center span, or more, having 9-ft beams. In flat topography, this additional 6 ft or so of vertical separation is difficult to attain without major disruptions to traffic for extended periods of time. A three-span bridge will reduce the grade separation perhaps 2 ft, but the added bridge construction time and cost may offset any direct benefit of the reduced bridge height and fill cost. High strength materials may also reduce span depth. Should the minor road also be widened, however, then some mainline grade change would likely be required for an upgraded TUDI because of the longer span.

Operational experience strongly suggests that undesirable compromises considered for alignment or bridge clearance should be avoided in freeway upgrades. Situations having undulating topography, producing a swale or saddle-back, or an elevated freeway section are more conducive to satisfying desirable

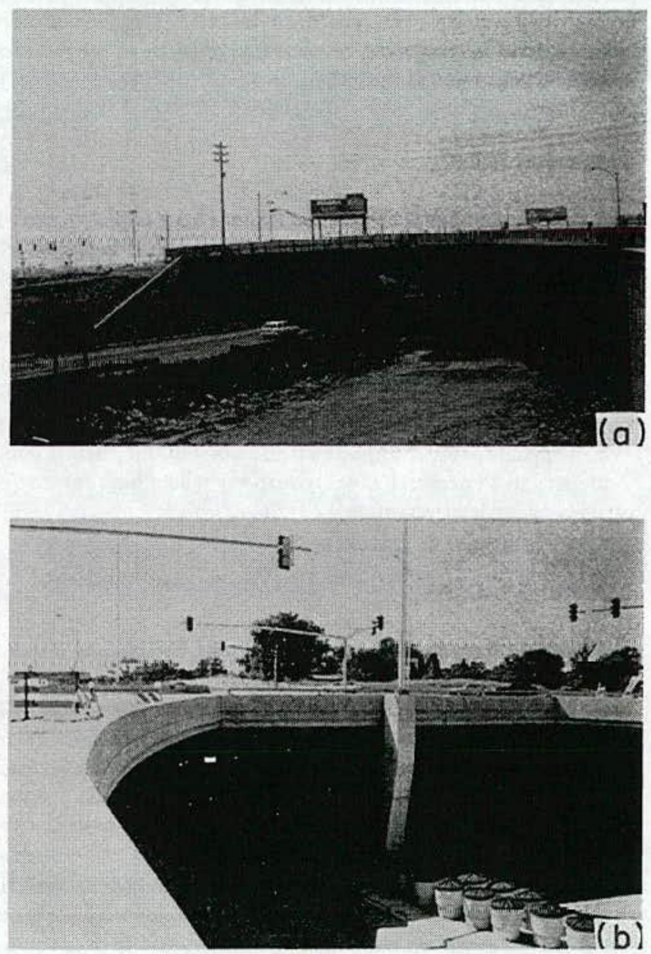


Figure 61. Freeway upgrades using underpass SPUIs: (a) site under construction; (b) site open to some traffic.

alignment design requirements at lower construction costs.

Obviously, traffic control of high volume freeway traffic during any reconstruction project of this size is a critical and difficult proposition. Nowadays, the upgrade design that can be built the fastest on an urban freeway may be considered more desirable, given that it has adequate capacity.

Arterial Upgrades

At least four states have been using SPUIs as a principal interchange in the process of upgrading existing signalized urban arterials to expressway-freeway standards. This work has focused on increasing the mainline corridor capacity over an extended distance, rather than just for local congestion relief. The acquisition of additional right-of-way along existing urban arterials is expensive and often becomes the principal design constraint.

Arterial upgrades in Florida, Georgia, and Alabama are examples. In all three of these cases, one-way frontage roads were built to handle the arterial traffic during construction. In these cases, all mainline bridges were overpasses. Multispan bridges with turnaround lanes were used in almost all cases. Once the bridge construction was completed, the one-way frontage roads used to handle detour traffic were left in place. The frontage roads with turnaround lanes provide enhanced access to local businesses that depend on established local service for their economic vitality. Every state mentioned the provision of convenient frontage road access using turnaround lanes as an important project selling point to the public.

Congestion Relief

A sizeable number of SPUIs to date have been built to provide relief from existing or projected traffic congestion at the intersection of a freeway or major arterial street with another crossing arterial. These designs to relieve spot congestion have widely variable attributes depending on the specific local conditions. They may have been the first SPUIs built in the state and, therefore, were somewhat of a test case. In some areas, their selection appears to have been based somewhat on their high-tech appeal to relieve expected traffic congestion. Their proposed use as gateway interchanges to airports where high turning volumes occur is an example.

DEMAND ANALYSIS

The field survey indicates that most future SPUIs will be built at locations presently experiencing peak hour congestion. In these cases, the current total intersection volume-to-capacity (v/c) ratio during the peak hour is about 1.0, or perhaps higher. Assuming the existing intersection traffic demand is at capacity permits a direct comparison between forecasted traffic growth and resulting v/c ratio together with inferred congestion and delay, assuming no capacity increase.

To provide some perspective on the future capacity requirements at congested intersections, Figure 62 presents the percentage increase in traffic demand that would occur over the next 20 years at typical rates of growth per year. The primary focus of this congestion analysis is on the critical signalized traffic

Effect of Annual Growth Rates on Volume-to-Capacity Ratios of Signalized Movements

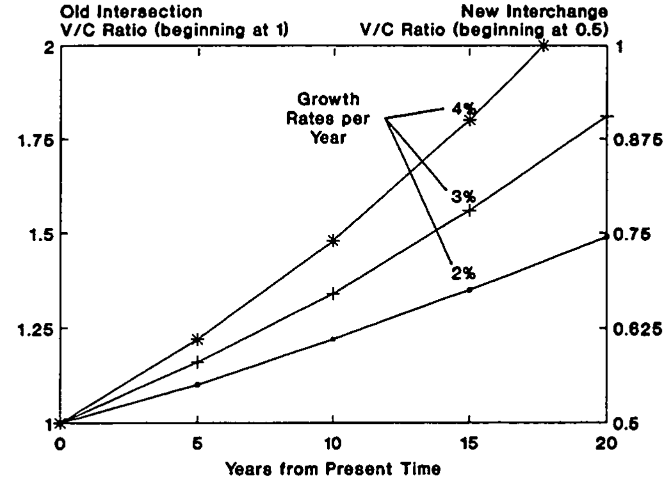


Figure 62. Volume-to-capacity ratio on critical phases related to traffic growth.

movements and not on the freeway flow provided. A base of 1.00 is assumed, permitting direct projections of resulting volume-to-capacity ratio growth on the total intersection. Also shown along the secondary Y-axis in the figure is the resulting v/c ratio if the present intersection were converted to an interchange having twice its present capacity for the remaining critical movements. Normal traffic growth rates are thought to be about 3 percent per year in urban areas where SPUIs may be considered, with low end growth rates of 2 percent and high growth rates of 4 percent. Figure 62 illustrates that a congested intersection today must have nearly a doubling in capacity for the new interchange to readily function for 20 years into the future.

CAPACITY ANALYSIS

The total capacity provided by an interchange primarily determines the amount of congestion relief provided by the new facility. Total capacity, however, is a complex subject, particularly for signalized diamond interchanges, and should not be taken lightly. Total capacity depends on a multitude of factors, including the total number of lanes, geometric design features, and traffic control.

The following analysis is limited to those candidate interchanges that are signalized and have two-level grade separation, primarily the single point urban interchange (SPUI) and the tight urban diamond interchange (TUDI). Because of the probable, extremely high cost of acquiring large parcels of developed urban land, a new signalized parclo interchange is not considered as being a viable interchange upgrade option for congestion relief in most developed urban areas.

Base Case

If the basic case experiencing congestion is an at-grade intersection (AGI), the solution scope is much different from that if the base case is an existing two-level interchange. In the former case, simply grade separating the major flow reduces the demand on the AGI by perhaps 40 percent or more. In addition, the number of conflicting phases is reduced from four to three, collectively increasing the total capacity of the remaining phases by nearly 50 percent. Any two-level grade separation that provides only modest improvements for the remaining traffic movements probably would provide acceptable congestion relief for a reasonable design life, as indicated by Figure 62.

The more complex problem occurs when an older two-level interchange is congested, and needs upgrading. In this case, mainline traffic flows have already been removed from the intersection and no other large reduction in demand can be achieved unless multilevel, or directional interchanges are used, which are outside the scope of this analysis. Thus,the remaining turning movements must be provided significantly more real capacity at grade in the intersection area. In this case, a realistic design approach requires that at least 75 percent more critical lanes be added to satisfy probable 20-year traffic growth. Most interchange design options will have to provide dual turning lanes to satisfy this capacity need.

Number of Lanes

The total number of lanes feeding (and exiting) the interchange primarily determines the total interchange capacity. Lanes are provided in three general locations: on the cross street at the throat to the interchange, on the cross street within the interchange, and along the ramps. Any plan to upgrade a two-level interchange to another two-level interchange should include widening the crossing arterial by at least two main lanes regardless of interchange type. Thus, upgrading is expedited when the existing cross arterial is a four-lane divided facility. Conversion to a SPUI will also require the use of dual advance left turn lanes and probably auxiliary arrival and departure right turn lanes. Consequently, planners should be prepared to provide an 8 to 10 lane cross section at the throat to the SPUI.

A similar number of lanes should be expected with a TUDI, with one exception possible. Some favorable traffic patterns may not require both advanced left turn lanes when using a TUDI because the inbound arterial phase carries both left and through traffic into the interchange. However, the TUDI then pays a penalty for this feature because the opposing left turns are both normally offset from the arterial centerline even though neither operationally conflicts with the other. Thus, the TUDI's interior cross section may have one or two more lanes than the SPUI, which would require more space. Two factors may offset some of the expected increase in cost for this case. Firstly, the state already owns most if not all the additional right-of-way needed. Secondly, the wider space required for TUDI overpass designs can be spanned with a cheaper, two-span median supported bridge than can the single span bridge used by a SPUI. If the TUDI bridge design has to span the total area because of design restrictions, little cost differential may result.

Capacity per Lane

Review of the 1985 "Highway Capacity Manual" shows that it does not provide an explicit analysis methodology on signalized diamond interchanges (*22*). Thus, most of the following capacity results were derived from previous work by Messer on TUDIs (*23*). Recent saturation flow and phase lost time studies by Bonneson (*11*), Poppe (*12*), and Hook (*13*) provide useful field data on SPUI operations. Bonnesón's work was conducted in Clearwater, Florida, as a part of this research; whereas, the studies of Poppe and Hook were recently completed in Phoenix, Arizona.

The total throughput capacity of a signalized diamond interchange is the product of the total number of input approach lanes, the number of phases, and the average phase capacity per lane on the critical input approaches. The well-known sum of critical movement volumes (SCV) results from the latter two components. The SCV method provides a planning tool for assessing the total input efficiency of the interchange. However, the equivalent average phase capacity (APC) method provides more insight on a design's ability to serve the hourly traffic volumes.

The sum of critical movement volumes (SCV) and their resulting average phase capacity (APC) per lane which serve the critical traffic movements on the external approaches to the interchange are developed from previous work by Messer on diamond interchange capacity (*23*). The sum of critical lane volumes for any signalized diamond interchange can be shown to be given by the equation:

$$SCV = S[1 + \Phi/C - n(p + c)/C] \qquad (3)$$

Dividing by the number of critical input phases, n, on the external approaches yields the average phase capacity per input approach lane as given by:

$$APC = S[1/n + \Phi/nC - (p + c)/C] \qquad (4)$$

where at the interchange SCV = sum of critical input movement volumes, vphpl; APC = average phase capacity per lane, vphpl; S = average saturation flow per lane, vphgpl; C = average cycle length, sec; n = number of critical phases in cycle; p = average lost time to start up platoons per phase, sec; c = average clearance lost time per phase, sec; and Φ = total phase overlap in the cycle, sec.

This methodology assumes that the internal turning movements at a TUDI have more capacity than the external movements and that they will not become critical movements. This will almost always be the case for nominal cycles if the spacing between the intersections does not exceed 400 ft (*23*).

Saturation Flow

One feature of the SPUI that appears to give it more capacity is the large radii of its left-turn paths. In general, drivers appear to have adapted well to the large radii and make efficient use of the protected left-turn phases. As noted below, the large radii induce higher saturation flow rates and operating speeds through the interchange as compared to an at-grade intersection (AGI) or a tight urban diamond (TUDI). However, these higher flow rates are attained at the expense of higher phase lost times needed to provide safe clearances between phase transitions.

Saturation flow per lane essentially represents the traffic carrying capacity of a lane when the signal is effectively green.

The 1985 "Highway Capacity Manual" (*22*) assumes that the saturation flow rate for through movements at AGIs is the highest at a typical value of 1,800 pcphgpl under ideal conditions. Left-turn phases are stated to be 0.95 of the adjacent through traffic, all other factors being the same. The left turn factor for dual left turn lanes for an AGI is 0.92 in the HCM.

Recent field studies of SPUI operations conducted in Florida (*11*) and Arizona (*12,13*) indicate that saturation flow rates at interchanges are higher than given in the 1985 HCM. In addition, these studies found that the left turn saturation flow at SPUIs is much higher than at AGIs. Poppe (*12*) recommended ideal saturation flow rates of 2,000 pcphgpl for both SPUI through and left turn movements. Bonneson (*11*) found that the operating speed, saturation flow rate and phase lost time all increase as the left-turn radius increases. Hook and Upchurch (*13*) studied seven TUDIs in Arizona and compared their data to Poppe's SPUI results collected nearby the year before. Like Bonneson's results, they found that left turn saturation flow increased with increasing radius of turn across both interchange types. This important and timely research study (*13*) also found that (1) there was no significant statistical difference in saturation flows for through movements between SPUIs and TUDIs of equal lane width; and (2) there was a significant difference in saturation flow rates for ramp left turns with the SPUI being larger, presumably because of its larger left turning radius.

Based on these recent field studies, the average saturation flow rate for an interchange through movement under ideal design conditions is assumed to be 2,000 pcphgpl. Furthermore, Bonneson's left turn equation developed from Florida data was calibrated to the Arizona data sets at a left turn radius of 200 ft where saturation flow rates of 2,000 pcphpl are predicted. The resulting calibrated equation for left turn saturation flow in passenger cars per hour green per lane (pcphgpl) is

$$S_{lt} = 3{,}600/(1.50 + 1.11/R^{0.245}) \qquad (5)$$

where S_{lt} = saturation flow of left turn maneuver, pcphgpl; and R = average left turn radius of turning maneuver, ft.

The application of this equation to signalized interchanges could easily follow the 1985 HCM procedures except that the saturation flow for "ideal conditions" would be 2,000 pcphgpl and the protected left turn factor, f_{lt}, as derived from Eq. 5 would be

$$f_{lt} = 1.0/(0.833 + 0.617/R^{0.245}) \qquad (6)$$

Quantifying the effects of trucks and other heavy vehicles on saturation flow is another matter. The passenger car equivalency (PCE) used in the HCM is 1.5 for through traffic only, but Molina (*24*) has shown that the PCE for typical through-moving urban truck traffic averages about 2.7, with a range from 1.7 for small trucks to 3.7 for five-axle trucks. Of concern also is that casual field observation suggests that large truck PCEs apparently increase with decreasing turning radius for a given lane width in some as yet unquantified way. Large truck volumes turning left from off-ramps under tight geometric conditions could have a major impact on capacity.

Interchanges Studied

The previously cited field studies provided saturation flow and lost time data used in the foregoing equations to make comparative analyses of three interchange types: the basic three-phase SPUI, the four-phase SPUI with frontage roads, and the four-phase TUDI with and without frontage roads as described in Ref. 23. The signal phasing of the SPUI design types is shown in Figures 32 to 34. The average saturation flow must be determined based on the type of phases used and the geometry of the interchange. A list of some of the values selected for analysis is presented below, followed by some derivations of geometric correlations among variables used in the comparisons. In all cases, the external phase durations were set to 30 sec based on the field survey of rush-hour conditions. Cycle lengths were adjusted accordingly.

Type of Interchange	Average Through Saturation Flow, pcphgpl	Average Cycle sec	Number of Phases	Lost Time per Phase p + c, sec
SPUI3	2,000	90	3	2 + c
SPUI4	2,000	120	4	2 + c
TUDI	2,000	100 to 114	4	2 + 2.5

Saturation flow rates used in the analysis depended on the turning movements that occur and assumed geometry of the interchange. The saturation flow rate for through movements was assumed to be 2,000 pcphgpl based on the above three field studies (*11,12,13*). Bonneson's generalized equation for left turns calibrated to the Arizona data (*12,13*) was used for all left turns, as given previously in Eq. 5. TUDI left turn radii were all assumed to be 60 ft with a 3 percent reduction assumed for the effect of dual left turn lanes as suggested by the 1985 HCM (*22*). This produced a left turn saturation flow rate for all TUDIs of 1,828 pcphgpl, but this does not account for any effects of truck operations.

A large high-type signalized intersection is assumed to have a total lost time per phase of 4.5 sec, composed of a 2.0-sec platoon startup time and a 2.5-sec clearance lost time per phase expected at the end of a phase for a base intersection about 50 to 60 ft wide. Poppe (*12*) estimated the clearance lost time for a SPUI as related to the signal change interval, *CI*, of the phase as

$$c = 0.947\ CI - 2.292 \qquad (7)$$

with an R-square of 0.97. It is desired to estimate the *CI* in Poppe's equation above, using the ITE signal change interval equation described earlier in Chapter Three as

$$CI = T + V/2d + (W + L)/V \qquad (8)$$

Assuming that the standard values are T = 1.0 sec, L = 20 ft, and d = 10 fpsps, and that a nominal approach speed of 35 mph results in a direct estimate of the clearance interval, *CI*, in terms of the conflict travel distance, W, in feet,

$$CI = 3.956 + 0.0195\ W \qquad (9)$$

Substituting Eq. 9 into Eq. 7 provides a direct estimate of the clearance lost time for a travel speed of 35 mph as related to the intersection conflict travel distance, W, for the movement within the SPUI interchange control area as:

$$c = 1.454 + 0.01847\ W \qquad (10)$$

A value of $W = 56$ ft produces a clearance lost time of 2.5 sec as assumed for large intersections. These results compare well with Bonneson's Florida data (*11*).

The phase overlap times used for tight urban diamond interchanges are estimated from data collected in Texas over two decades ago (*23*). These data are widely accepted as average values in the PASSER III diamond interchange signal timing program and are used nationwide. The total overlap is estimated from the following travel time equation for vehicles beginning from a stopped position as

$$\Phi = 2[0.5 + sqrt(0.45\ S) - 2.0] \qquad (11)$$

where S is the stop-line to stop-line travel distances, in feet, between the two intersections of the TUDI, and also roughly equals the center-to-center of the ramp spacings between the two intersections, D.

Comparative studies were made between SPUIs and TUDIs having the same center-to-center ramp-to-ramp spacings, D. The following relationships were assumed: for TUDIs, $S = D$, and $W = 36$ (and 56); for SPUI3s, $W = S = D$; for SPUI4s, $W - 36 = S = D$, and where for SPUIs W is for the through movements only. Data from the field survey were used to obtain geometric correlations between SPUI left turn distances within the central control area and D. These studies revealed that $L = 0.87\ D$, which can be estimated from basic surveying. In the previous equations, D is the center-to-center ramp separation distance, in feet; W is the width of the central control area, and L (= W for left turns) is the average turning path length in the SPUI conflict area for a given radius R, in feet, for a given size D. The SPUI bridge is assumed to have a nominal design length. The left turn correlations are an average of single-span and three-span bridges for overpass SPUIs. Some minor differences were noted between single-span and three-span bridges, as expected.

Capacity Results

Figure 63 presents the calculated capacity results for nominal designs and for passenger car operations only. Effects of large truck operations are not included. Figure 63(a) shows the sum of critical input lane volumes, *SCV*, (from Eq. 3) as a function of center-to-center off-ramp separation distance, D. Again note that off-ramps are assumed to be 36-ft wide and this distance is also assumed to be the same as the nominal stop-line to stop-line separation distances of through traffic for SPUI3s and TUDIs. As Figure 63(a) shows, the TUDI clearly provides a higher *SCV*, but must serve 4 input volumes. The SPUI3 and SPUI4 have about the same *SCV*, but the SPUI4 must serve four critical input traffic movements; whereas, the SPUI3 does not have to serve the frontage road through traffic.

Figure 63(b) provides additional insight on interchange capacity as it depicts the resulting average phase capacity per lane, APC (from Eq. 4) for the three interchange types. This figure suggests that a SPUI3 is slightly more efficient than a TUDI on a per lane basis (perhaps 50 pcphpl per phase per lane at nominal ramp spacings), but the advantage diminishes as the size of the interchange becomes larger. At a nominal stop-line separation distance of 265 ft, they become equal. The SPUI4 clearly does not have as efficient lane capacities per phase for passenger car operations, primarily because of the large phase lost times incurred, unless perhaps if significant left turn truck volumes are present on restricted off ramps. Likewise, Figure 63 indicates the capacity benefits to all SPUIs by keeping the intersection conflict area, W, of a well-designed SPUI as small as possible, thereby minimizing clearance lost times.

Right Turns at SPUI3

Right-turn capacities of the SPUI3 and TUDIs can be very dissimilar. While the capacity per lane of the SPUI3 right turn from the cross street is about the same as the TUDI, its off-ramp right turns may have less capacity per lane than does the TUDI and is operationally much less dependable. Off-ramp right turns should be carefully evaluated and addressed in both geometric and signal design. Failure to apply the design procedures described in Chapters Five and Six may result in surprising congestion along the off ramps under some traffic conditions, as explained in the following.

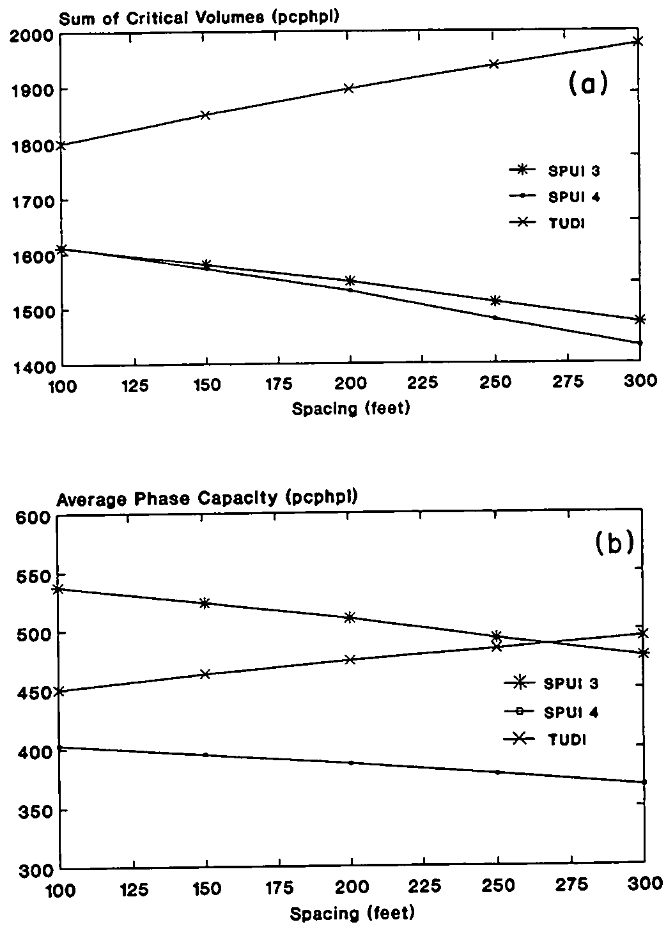


Figure 63. Sum of critical input volumes and resulting average phase capacity for three interchange types as related to center of off-ramp spacing: (a) sum of critical input volumes; (b) average input phase capacity.

All TUDI right turns have one parent through signal phase to protect its flow onto the intersecting link as does the SPUI3 right turn from the cross street. Normally, the through and right turn volumes are correlated and the through volumes are large enough per lane to protect the rights. However, the off-ramp right turns from the SPUI have no parent through phase to carry them, and this fact can cause problems. The right turn can have protected entry only during the near side left turn phase from the cross street. These right and left turn volumes are not usually temporally correlated. Consequentially, during rush hours when the subject right turn volume is high and if the opposing left turn volume is also high, an additional phase may have to be called in just to serve the remaining right turn demand. In contrast to its off-ramp capacity, the SPUI3's arterial right turns have one additional phase to protect it, the off-ramp left turns from the mainline (the ones that would protect TUDI or a SPUI4 right turns but do not protect the SPUI3). Thus, cross-street right turns are seldom an operational problem at the signal, but may experience downstream merging problems if restricted merging occurs.

To summarize, planners should be advised that off-ramp right turns at a SPUI3 may have less right-turn capacity than expected from traditional intersection experience. Downstream weaving to local access points can also be heavy and difficult. Consequently, special attention should be given to evaluating the off-ramp right turns as compared to those from the cross street, particularly in high-volume situations having balanced traffic patterns. Dual off-ramp right-turn lanes should be considered. Several computer programs, including the Highway Capacity Software, TRAF-Netsim, and TRANSYT 7-F, can be used for assessing these operational issues.

Freeway Merging

Since dual-lane left-turn capacity from the cross road of a SPUI is efficient and can carry 900 vph or more during the rush hour, the potential exists to overload the downstream freeway on ramp, especially when considering the efficiency of the cross-street right turns that likewise could output up to 900 vph. A capacity analysis of the freeway on-ramp merging volume should be conducted to assure that potentially high on-ramp volumes from the SPUI can be accommodated. The analyst should also consider whether ramp metering or HOV bypass lanes may be used at the on ramp in the future and provide any additional queue storage needed.

SPUI DESIGN GUIDELINES

The single-point urban interchange, or SPUI, is a new and innovative highway facility for improving urban mobility. It has attributes of both a high-type signalized intersection and a large grade-separated interchange. Operational experience at more than 25 interchanges reveals that well-designed interchanges that emphasize the principles of positive guidance can be designed to operate safely and efficiently under a wide range of conditions. However, field observations also indicate that every effort should be made to provide adequate funding for a design meeting desirable standards for geometrics and traffic control devices. All aspects of the physical features of the SPUI should be thoroughly planned down to the smallest visible details before the preliminary design is completed to ensure operational efficiency is attained. As an example, in no other interchange are the details of the bridge design so interrelated to geometric, traffic control, and illumination design. The following design process is recommended for providing a quality SPUI design.

Design Objectives

A SPUI design that satisfies the following objectives would be expected to operate safely and efficiently for the design life of the project. A systematic design process should be followed to assure that all design elements are in harmony and provide a high level of positive guidance. Adequate operational transitions from mainline operations to operating in a SPUI environment are essential. Early planning and coordination of geometric design, traffic control device design, and bridge design are encouraged to more efficiently achieve the desired system design goals. The recommended principal SPUI design objectives are as follows:

1. Size the interchange to provide adequate capacity to satisfy vehicular traffic demand expected for the design year in a safe and efficient manner.
2. Select the most desirable grade separation type, overpass or underpass, for existing conditions. Simple, multispan bridges are preferred.
3. Provide a bridge design that can efficiently add two future main lanes without major structural modification or significant impact on mainline traffic.
4. Provide a design that can be readily expanded to a full 6-2-2-2 configuration during the design life of the interchange (see Figure 1).
5. Provide a design that can be efficiently constructed, given site-specific conditions, in a minimum amount of time and traffic interruption.
6. Provide a design sensitive to the local aesthetics and environment.
7. Provide adequate visibility and sight distance of the critical geometric and operational features, both day and night.
8. Provide facilities appropriate to serve the pedestrian traffic demand expected for the site. Minimize pedestrian impacts on traffic capacity.
9. Provide the traffic control devices best suited to fulfill the needs of unfamiliar motorists operating on this class of interchange.
10. Obtain adequate right-of-way to satisfy design year traffic demand for the traffic movements at desirable operating speeds.

The foregoing list is certainly not exhaustive of all the important design goals and decisions that need to be addressed and satisfied; however, the above list of objectives is provided to help focus the attention of the design team on ten items believed to be the most critical in providing a good SPUI design based on the results of the field survey conducted within this research. All elements of the design should be thoroughly coordinated, down to making sure that the orientation of signal lens arrows and lane use arrows are in concert with the guide signs and geometrics.

Principal Design Parameters

This section describes some of the major design parameters and constraints usually found in a SPUI design problem. This information can be used to guide the assessment of the feasibility of a SPUI as a viable design alternative when examining an existing site, or to guide designers as they prepare preliminary design plans. The following checklist of typical constraints needs to be considered early in the preliminary design process:

1. Need to maintain traffic flow of intersecting highways during construction.
2. Need to maintain existing mainline vertical and horizontal alignments.
3. Presence of mainline frontage or service roads.
4. Interface with local cross street alignment and cross section.
5. Need for immediate local access to abutting property.
6. Need for nearby major intersection with local street system.
7. Physical constraints, such as a nearby parallel railroad.
8. Location and nature of existing utilities and easements.
9. Geotechnical issues related to support strength for substructures, the level of the water table, and the need for and convenience of storm water drainage.
10. Local right-of-way costs and parcel availability.
11. Agency policy decisions regarding use of elevated structures.
12. Level and nature of construction funds available.

Principal SPUI Design Decisions

The formulation of a SPUI design must early on consider the basic physical and cost elements in the design. A typical SPUI may cost $10,000,000, or more. Of this cost, one-third can be for right-of-way, one-third for constructing the bridge, and one-third for all other items combined. A wide variability in cost of right-of-way is to be expected, depending on site-specific economics. The large bridges used in SPUI designs merit critical analysis of design options, particularly for overpass designs where unusually long single-span lengths exceeding 200 ft may be proposed.

Some initial planning must address the requirements for maintaining traffic flow on the mainline and cross street during construction, the durations that any or all lanes can be closed, and how this work fits into the overall construction program in the area. The following major design decisions must be addressed, given that a SPUI is to be constructed:

1. To build the most cost-effective SPUI that satisfies the previous design objectives in a timely manner.
2. To use an overpass or underpass grade separation of the mainline highway.
3. The selection of desirable rather than minimum level AASHTO geometric design controls, particularly for design speed and sight distance.
4. The selection of cross section for the cross street, including the use of median divider for advanced left-turn lanes, center island, and any provision of paved shoulders, bicycle lanes, and pedestrian sidewalks.
5. Bridge design, including type, length and number of spans, location of bents and abutments, vertical clearance, and provision in the bridge design for future added mainline capacity.
6. Traffic control device system to be used, and location of traffic control devices on approaches, within islands, and on the superstructure of the bridge.
7. Type of roadway lighting system used, if any.
8. Use of FAA runway lights to dynamically mark left turning lanes.

It is most desirable that the formulation of the bridge design alternatives and preparation of the preliminary bridge design plans be coordinated with all other critical design features to best satisfy the design goals and objectives of the project.

BRIDGE DESIGN GUIDELINES

The large grade separating bridge structure is the principal design feature of a single-point urban interchange (SPUI). By removing the largest through movement flowing through the intersection, the effective capacity available to the remaining traffic movements is increased by over one-third. The added cost of providing the bridge(s) is likewise large.

Bridge Type

The data base reveals that a wide variety of bridges are used to provide the grade separation. Of a total of 36 bridges in the survey, 27 (75 percent) are mainline overpasses of the cross road and 9 (25 percent) are underpasses. Of the 27 overpasses, 7 are single-span bridges, 20 are multispan bridges. Of the 9 underpasses, 2 are single span and 7 are multispan. Most bridges are unit designs but 2 presently have separate structures that provide for additional roadway capacity expansion in the future. Several of the other unit-designed bridges likewise provide abutment designs with room for adding an additional freeway lane in each direction in the future.

Superstructure design also varies widely, primarily dependent on site-specific design requirements and local cost-effectiveness results. Local construction capabilities together with local design preferences have affected the bridge options considered. Three bridge designs are common for single-span, overpass SPUIs having clear spans in the 180 to 280 ft range: (1) steel plate girder (3), (2) steel trapezoidal box girder (3), and (3) concrete cast-in-place box girder (1). The number of bridges contained in the sample of the type described is shown in parentheses.

A range of multispan (usually 3 spans) bridge types were also noted for overpass SPUIs. As in the prior case, three types of superstructure designs were noted as: (1) steel plate girder (12), (2) steel trapezoidal box girder (2), and (3) concrete cast-in-place box girder (1).

Underpass designs also varied in material type and number of spans. It is important to note that most underpass designs use a mid-span bridge support bent, unlike no overpass design in the survey. All multispan (usually 2 spans) underpasses used the mid-span bridge support located in the median of the underpassing highway. Three types of bridges contained in the data base for underpass designs are: (1) steel single span (1), (2) steel multispan (2), and (3) concrete cast-in-place multispan (3).

Bridge Span

A long clear span bridge is required for overpasses to provide the open space needed in a SPUI for the four inverted left turns to operate inside one another. Several major interchange design elements have increasing cost impacts with increasing span length. Among these are bridge superstructure and substructure, roadway embankment and realignment, and ultimate right-of-way takings.

Simple span bridges are frequently used at SPUIs where the freeway overpasses the cross road. Observed span lengths are given in Figure 30 for these designs. Span length varies with size of the interchange but there is a common range of single spans around 200 to 220 ft. The longest operational single span bridge of 270 ft was found at a new SPUI located on I-215 in southern Salt Lake City, Utah. Figure 60 shows this interchange under construction.

Multispan bridges are more frequently used for overpass designs. Field observations indicate that they provide an openness and generous visibility that are desirable features. However, setback locations of the supporting bridge bents (piers) can be a design issue for providing desirable sight distance to crossing movements. A typical design without frontage roads may have three spans on the order of 100-180-100 = 380 ft. The smaller bridge bents are structurally attached only to the mainline girders. This provides added visibility compared to simple-span bridge abutment designs whose cross section is generally wider than the total roadway cross section. In multispan bridges, one design control requires a minimum proportional spacing between the two supporting bents to eliminate any upward deflection of the tips of the end spans when large trucks are critically located in the center span.

Multispan bridges have been used in overpass designs for several reasons. Multispan bridges are probably required when an overpass SPUI4 is to be built along a freeway (or divided highway) that has one-way frontage roads. This is because frontage road turnaround (U-turn) lanes under the structure will probably be desired for local traffic circulation. The added spanning distance of these turnaround lanes requires the bridged distance to be lengthened another 200 ft or so, which can be economically provided only with a multispan design having two additional 100-ft end spans. An example of this type of frontage road system was observed in Huntsville, Alabama, where four SPUI4s have been built using multispan bridges to provide the necessary clearance for the U-turn lanes. Figure 64 (top) shows one of these multispan bridges.

Multispan bridges have also been used where there is significant curvature in the horizontal alignment. Figure 64 (bottom) shows one of these bridges nearing completion. All multispan bridges are continuous beam designs that reduce the required structural beam depth about 1 ft, but require some design flexibility in locating the supporting bents and bridge abutments to attain the span ratios that minimize uplifting the beam ends from the bridge seat. The general openness and good aesthetics of multispan bridges compared to simple span bridges having steep-sided embankments or retaining walls may also be important decision attributes in some terrain conditions and urban locations.

Underpass designs, while sometimes having special drainage problems, usually can be designed with about one-third the clear span due to the use of a median bent. Thus, an underpass SPUI design may need no additional vertical separation, assuming bridge clearances of the respective highways are the same. This can be an important cost consideration in upgrading an existing intersection.

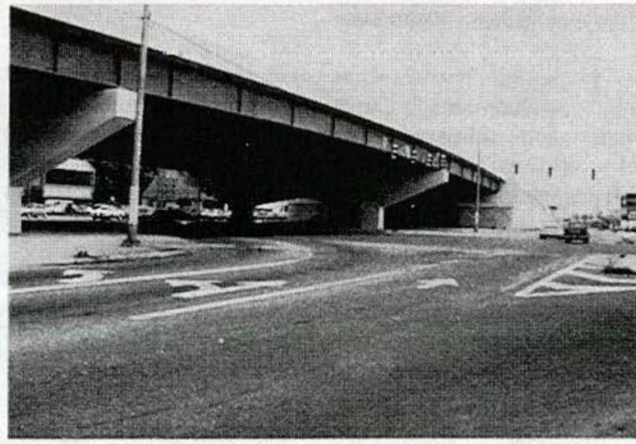


Figure 64. Examples of steel girders used to construct multispan bridges.

As a matter of information, planners may desire to know that AASHTO bridge design guidelines (*25*) provide formal numerical design guidelines relating bridge clear span and depth of the girder. For composite girders, the ratio of the overall depth of girder (concrete slab plus steel girder) to the length of span preferably should not be less than $\frac{1}{25}$, and the ratio of steel girder alone to length of span preferably should not be less than $\frac{1}{30}$. These design requirements may be exceeded, however, at the discretion of the designer when certain design criteria are satisfied. Figure 65 illustrates the impact of bridge span on depth of girder based on the $\frac{1}{30}$ guideline. Also shown in Figure 65 is the potential total height of the bridge, assuming a vertical clearance to the bottom of the bridge of 17.0 ft and a 1.0 ft pavement depth. A composite design requires thicker spans but less steel, probably being a less attractive design approach for most SPUIs.

The above AASHTO structural requirement usually causes major revisions to be needed in either the mainline or cross-road vertical alignment profiles when existing interchanges of simple diamond designs having clear spans of 60 to 80 ft are converted into single span SPUIs having 200 to 240 ft clear spans, respectively. Assuming no added vertical bridge clearances are needed, the vertical profile for either the mainline or cross road will need to be separated an additional 6.0 ft or so. This is a common problem with overpass SPUI designs. A review of several case histories suggests that usually either the mainline is raised or the cross road is depressed to achieve this nearly 6-ft additional grade separation. Two cases are known where both original roadway alignments were adjusted some to achieve the required vertical separation in grade lines. To be sure, widening the cross road at the interchange will likely require some added span depth regardless of the type of configuration, SPUI or TUDI, and any significant elevation change in vertical alignment would have a related effect on the reconstruction cost of the interchange.

A COMMON DESIGN PROBLEM

Traffic congestion is normally due to inadequate physical capacity of the intersection, perhaps combined with inefficient signal operations. Where inadequate capacity is the dominate reason for the intersection congestion, grade separating the mainline traffic greatly reduces the traffic demand on the remaining at-grade facility and congestion will probably disappear for 10 years or more regardless of the resulting type of interchange selected. However, modifying an existing interchange to reduce congestion will not receive the dramatic benefits of grade separation and, therefore, proposed interchange upgrades should be carefully evaluated and trade-offs understood. The following scenario is provided to highlight critical design issues.

Operational Scenario

A typical operational problem that must often be addressed by traffic planners is an at-grade-intersection (AGI) that is experiencing congestion during the morning and afternoon peak hours. While each case may vary, it would appear unlikely that a SPUI would be considered to replace a congested intersection until the ultimate build-out capacity of the AGI has been reached within the available right-of-way and the existing intersection volume-to-capacity ratio is near or exceeding 1.0 during the peak hours. This design scenario implies that some steady increase in traffic demand be occurring at the intersection, say perhaps 3.0 percent per year.

As noted in Figure 62, the above traffic growth over the design life of the project provides a design goal for the level of capacity improvement needed. If one assumes a design life of 20 years from the day of decision, the following increases in physical capacity are estimated from Figure 62 to be needed for the AGI, and for comparison purposes an existing older TUDI, to provide undersaturated traffic flow until reaching the projected design hour conditions 20 years in the future.

Action	Existing Facility AGI (%)	Existing Facility TUDI (%)
20-year demand increase at 3 percent	81	81
demand reduction per lane due to grade separation	31	0
net increase in physical capacity required to achieve v/c = 1.0	50	81
net increase in physical capacity required to achieve v/c = 0.8	62	100

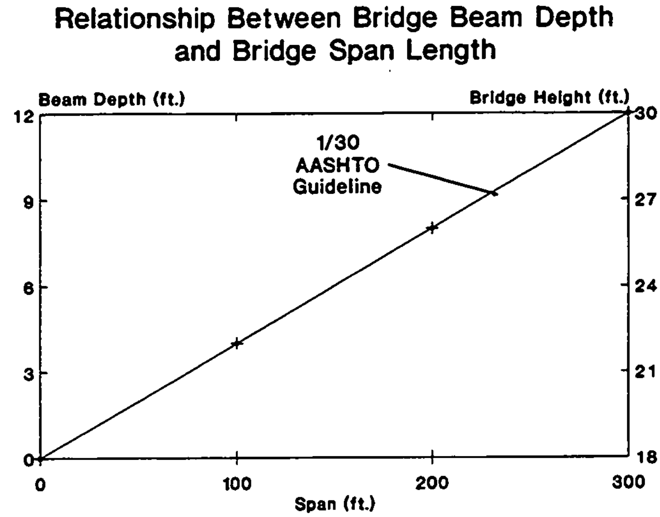


Figure 65. Generalized relationship between bridge depth and bridge span based on AASHTO design criteria.

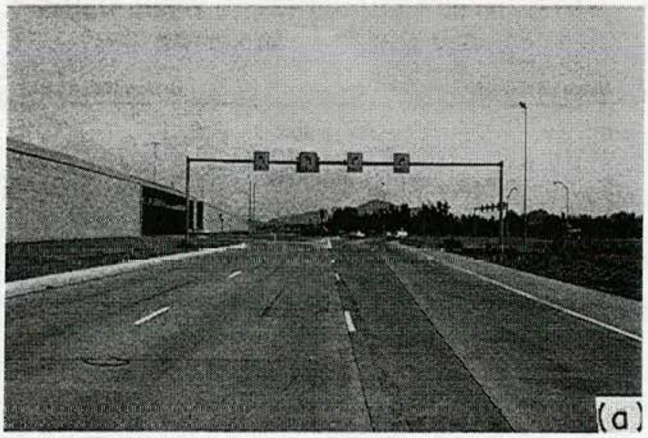
This simple analysis clearly identifies the scope of solutions required within existing right-of-way. For an existing AGI, the new design needs to have about 50 percent more capacity at the resulting intersection just to satisfy capacity requirements. If the existing design is a TUDI, 81 percent more capacity is required just to provide existing traffic conditions 20 years into the future. If one designs for a volume-to-capacity ratio of 0.8 in the design year, the net capacity increase that should be provided by the new facility rises to 62 percent and 100 percent, for the present AGI and TUDI, respectively. Clearly, any new two-level interchange design must provide at least 50 percent more lanes at the intersection, and a new design for an old TUDI would need to double the existing interchange signal capacity. Resulting mainline operation was not assumed in this analysis to be experiencing congestion.

Solution Scope

There are only two places to provide more lanes at the intersections of an existing two-level diamond interchange. One place is on the cross street by widening the cross road, say from four to six lanes, and then simultaneously providing dual left-turn lanes. Another is to increase the number of off-ramp lanes from perhaps two to four by splitting the off-ramp turns prior to reaching the interchange. Usually, auxiliary right-turn lanes on the cross street have already been used in the existing design. This proposed upgrade solution was commonly observed in the field survey for congestion relief SPUI interchanges, as depicted earlier in Figure 1. The added critical lane upgrade ratio in the SPUI design of Figure 1 was 8/5 = 1.6, or a 60 percent increase in the number of critical approach lanes. Improvements in saturation flow per lane added perhaps another 10 percent increase in interchange capacity.

Ultimate SPUI Design

The ultimate build-out SPUI interchange configuration without frontage roads appears to be a 6-2-2-2 configuration for both



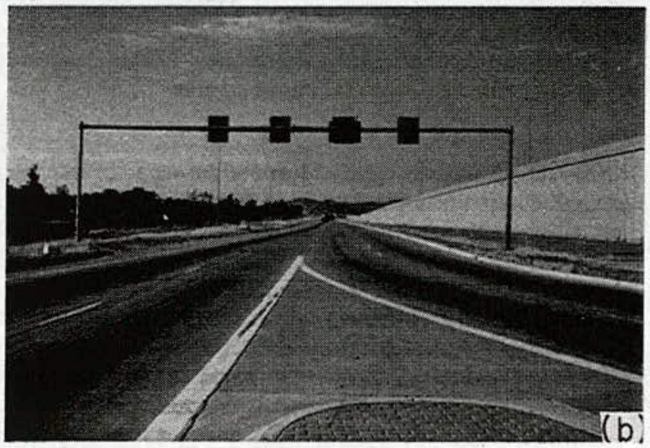


Figure 66. Ultimate off-ramp design for 6-2-2-2 SPUI: (a) normal view looking downstream; (b) back view looking upstream.

the overpass and underpass designs. This design would be a six-lane cross arterial having two-lane left-turn lanes at the junction, as shown in Figure 1, combined with two lanes for off-ramp left turns and two lanes for off-ramp right turns, as depicted in Figure 66. A 240-ft single-span bridge may be needed. The overpassing bridge abutment can and should be designed for supporting one additional freeway lane in each direction at some future date without any major modification or difficulty. For AGIs, a four-lane freeway could be built initially, with ultimate horizon year expansion provided for six lanes. With existing TUDIs, a minimum of six freeway lanes should be provided with a horizon year expansion capability of eight lanes. Once the bridge is constructed, the cross-street capacity cannot be increased without grade-separation to three levels.

Future expansion to three-level interchanges can be provided easier from two-level underpass SPUI designs than from overpass designs by using cross-street flyovers, essentially converting the design into a three-level stacked diamond (*26*).

CHAPTER FIVE

INTERCHANGE DESIGN GUIDELINES

The quality of an interchange design depends on the harmonious integration of all the physical features that comprise the interchange. The design of each component and of the entire interchange should be adequate to serve current and future traffic demands in a safe and efficient manner. To ensure that the SPUI design has these qualities, appropriate geometric design controls must be established and applied uniformly to every element of the design. Such controls should be based on provision of an acceptable level of traffic service, adequate sightlines and sight distances for all drivers, smooth roadway surfaces, comfortable transitions at changes in cross section and alignment, and features that are generally consistent with driver expectation. Nationally approved geometric design controls and guidelines that should be used are provided in the AASHTO Green Book (*1*). The engineering staff should assure that optimal trade-offs between the structural controls of the bridge design and operational impacts of the geometric design have been made.

The geometric design guidelines published in the AASHTO Green Book form a sound basis for the design of typical streets, highways, intersections, and interchanges. Most design issues are discussed in the Green Book and those that are not can often be resolved by extension of the Green Book's principles. However, because of the unique operation of the SPUI, some of its design features may have requirements different from the more common interchange forms. Several features of the SPUI, e.g., the left-turn travel paths, raised islands, grade separation structure, and so forth, are not typical for a conventional interchange and need special attention.

Several geometric design elements of the SPUI appear to have operational requirements not expressly addressed in the Green Book. Supplemental design guidelines for these situations are covered in this chapter. These design elements include horizontal and vertical alignments, channelization and pedestrian service. Because sight distance is the primary design control for the horizontal and vertical alignment, both will be discussed in a single section on sight distance. The other design elements will be discussed in individual sections.

The last section of this chapter will focus on the relationship among the various design components of the SPUI. As with all designs, the efficiency of an interchange depends on the harmonious integration of the geometric design and bridge design with the traffic control plan. The design of the SPUI is particularly dependent on this harmonious relationship because of its special geometry and operation. As a result, the SPUI designer should use "desirable" (versus minimum) geometric design controls whenever possible to mitigate driver unfamiliarity and the complexity of SPUI operation. The generous application of sight distance is recommended.

SIGHT DISTANCE

The safety of a roadway is highly dependent on the amount of sight distance available to the motorists traveling along it. Drivers must be able to see far enough ahead along the roadway to detect hazardous or unexpected situations and to make appropriate adjustments in their travel paths. When adequate sight distance along the roadway is not available, the driver may not have adequate time to perceive and react to an unusual situation and, consequently, an accident may result.

It is the responsibility of the designer to ensure that adequate sight distance is provided along the roadway. In most situations, adherence to standard design guidelines will provide adequate sight distance along typical design elements. However, the designer should verify the adequacy of the sight distance provided by the total design. Situations that may require special consideration include locations with unusual, or complex, operating conditions. These locations may require longer sight distances than more typical designs. Similarly, locations with atypical design elements may present an otherwise unexpected obstruction to the driver's line of sight. In this regard, sight distance within the SPUI should be checked along both the horizontal and vertical alignments. Special attention should be given to the effects of bridge abutments and piers on collision avoidance sight distances within the intersection area, as illustrated in Figure 60. Assumed operational speeds onto freeway off ramps and subsequent driver responses should recognize the probable driver unfamiliarity with SPUI operational requirements and make as liberal design assumptions as feasible regarding the selected operational design criteria.

Adequate sight distance at a SPUI is important because of its special design and its relatively unusual operating conditions. The following sections describe the basic types of sight distance typically considered in design and their recommended application to the SPUI.

Driver Sight Distance

The amount of sight distance the driver needs can vary, depending on the type of hazard or unexpected situation that must be avoided. The principal types of sight distance that should be provided in the design of an interchange include: stopping sight distance, decision sight distance, and intersection sight distance. Recommended application of each of these geometric design controls in SPUI design are presented in the following paragraphs.

Stopping Sight Distance

Stopping sight distance is the minimum sight distance available along a roadway that is just sufficient for a motorist traveling at the design speed to perceive an unexpected hazard and come to a complete stop before reaching it. This distance has two components. The first component is the distance traveled by the driver while he or she sights the hazard, decides on an action (i.e., stopping), and then initiates that action. The second component of stopping sight distance is the distance needed by the driver to safely stop his or her vehicle.

The stopping sight distances used in roadway design are provided in Table III-1 of the AASHTO Green Book (Ref. 1, p. 120). These stopping sight distance values represent the stopping distances that would be obtainable under wet pavement conditions for the majority of the driver population. It is highly recommended that desirable design conditions be selected where feasible, particularly for downgrade off ramps. Stopping sight distance is provided along the entire roadway and is unrelated to the presence of an intersection or interchange along it.

Decision Sight Distance

Under normal driving conditions, the provision of stopping sight distance along the roadway is adequate for drivers to perceive and react to conditions in the driving environment. However, as the environment becomes more complex and the number of information sources increases, as occurs within an intersection, drivers generally require more time to perceive, interpret, and respond to this additional information. The travel distance associated with this increased time is commonly referred to as decision sight distance.

Decision sight distance is different from stopping sight distance both in the time required for the perception-response time and the type of maneuver that occurs. As its name implies, stopping sight distance allows the driver time to stop prior to reaching a hazardous obstacle. On the other hand, decision sight distance allows time for drivers to react to a hazardous situation by making changes at the vehicle guidance level. In this regard, the assumed maneuver in calculating decision sight distance is a lane change. This response is based on the premise that the driver will be required to change lanes to avoid an unexpected obstacle in his/her lane rather than stop. Because extra time is needed to process the information under complex conditions, decision sight distance is about twice as long as stopping sight distance.

AASHTO Green Book values for decision sight distance for various avoidance maneuvers are provided in Table III-3 therein (Ref. 1, p. 127). Provision of decision sight distance for avoidance maneuvers D or E is encouraged for vertical alignment control of off ramps and recommended for vertical alignment control of the cross street throughout the interchange area.

Intersection Sight Distance

Drivers approaching the intersection of two roadways need an unobstructed view of the intersection to ascertain the safety of traversing it. These drivers must be able to see all traffic movements that conflict with their path as they approach and travel through the intersection. When traffic at the intersection

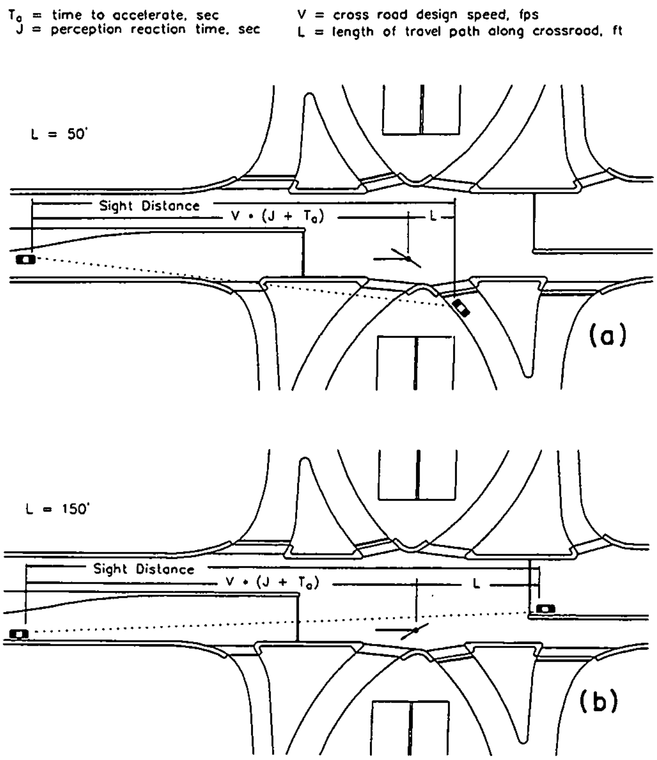


Figure 67. Sight distance components for signalized left-turn movements: (a) off-ramp left-turn movement; (b) cross-road left-turn movement.

is controlled by signs or signals, the unobstructed views may be limited to the control area. Because of the different types of impending vehicle motion and relative location of conflicting vehicles, sight distance on the approach to an intersection is different from the sight distance needed to travel through the intersection. These two types of sight distance, approach and departure, should be addressed in the design.

Approach sight distance is the minimum distance approaching drivers can be from the intersection that still provides sufficient time to make necessary changes in speed or path to avoid a conflicting vehicle. To provide this sight distance, the included corner between two conflicting movements should not have any physical features that would obstruct the approaching driver's vision of the conflicting traffic stream. At signalized intersections, YIELD-controlled approaches must provide this sight distance.

Another type of sight distance needed at intersections is departure sight distance. This sight distance is the minimum distance that a stopped vehicle must have of the conflicting traffic stream to provide sufficient time to enter the traffic stream safely without major disruption to the conflicting traffic. This sight distance is measured along the cross road from the point of entry of the minor or controlled movement. Departure sight distance is typically restricted by vertical structures and by significant changes in vertical or horizontal alignment.

The main signalized conflicts in basic SPUI design are related to the two left turns depicted in Figure 67. Sight distance for the signalized left-turn movements at SPUIs is needed because of the increased driver work load and potential hazard associated with crossing the large conflict area. In this regard, the left-turn driver must be able to see along the cross road far enough upstream to ascertain the safety of crossing the oncoming traffic. The Green Book calculates departure sight distance for the left-turn movement based on the speed of the approaching traffic and the crossing distance. Special consideration needed at a SPUI is the distance the turning vehicle travels toward the oncoming vehicle before clearing its path. Although this distance is minimal at typical intersections, at SPUIs it can measure 50 to 150 ft.

Intersections controlled by traffic signals are considered by many not to require sight distance between intersecting traffic flows because the flows move at separate times. However, due to a variety of operational concerns associated with intersection operations, AASHTO policy states (Ref. 1, p. 760) that departure sight distance based on Case III procedures should be available to all drivers using the signalized intersection, including those right turns on red from signalized off ramps.

The off-ramp right-turn movement onto the cross road is operationally sensitive to departure sight distance. If adequate sight distance is not available to motorist to yield before entering the cross road, increased conflicts between right-turning and cross-road drivers may occur. If departure sight distance is not available, an approaching cross-road driver may have to slow to well below the average running speed or make an unexpected lane change to avoid the entering right-turn driver. Moreover, if departure sight distance is not provided, this may result in a reduction in the capacity of the right-turn movement. Specifically, driver apprehension resulting from obviously inadequate sight distance may indirectly increase the duration of the yield maneuver for each right-turn driver.

The AASHTO Green Book calculates departure sight distance based on the speed of vehicles on the street being crossed or entered, the magnitude of speed reduction required of this vehicle to avoid a conflict with the entering vehicle, and the acceleration time of the entering vehicle. See the AASHTO Green Book for specific sight distance guidelines and design criteria (Ref. 1, pp. 739–769).

Application Of Sight Distance To SPUI Design

The most common technique for incorporating sight distance needs into the design process is as a design control in the development of the horizontal and vertical alignments. Because design guidelines for more typical interchange elements can be found in the AASHTO Green Book, the following discussion will focus on sight distance requirements in the vicinity of the SPUI's signalized junction. Sight distance considerations along the major roadway alignment and within the ramp and major road junctions at a SPUI are similar to those at other interchange types and, as a result, their design would follow the guidelines set forth in the Green Book.

On-Ramp Alignment

Sight distance along the on-ramp turning path can be restricted by the edge of a bridge abutment or barrier wall. The first restriction would be found at an overpass SPUI where the major road passes over the cross road. With the overpass design, measures are often taken to minimize the overall length of single-span bridges by bringing the bridge support structure as close as possible to the cross road (and on ramp). However, if the abutment is too close to the on ramp, the driver's view along the turn path may be blocked. Care should be taken to ensure that any auxiliary facilities, such as drainage structures and signal cabinets, do not restrict the view.

One technique that can improve sight distance along the on ramp and minimize the length of the center bridge span is the use of a three-span bridge with open-end spans as opposed to a single-span bridge with closed abutments. As shown in Figures 64 and 68(a), the open-end span poses minimal restriction to sight distance along the on ramp, improves visibility through the interchange, and permits the use of a shorter center span (at a shallower depth) than does a single-span bridge with closed abutments. Increasing the openness of the intersection area reduces the "dark hole" effect and, thereby, driver anxiety when traveling under the structure. However, if other design considerations require the use of closed abutments, the bridge abutment should be sufficiently offset from the roadway so that it does not restrict sight distance to off-ramp traffic. Minor improvements in sight distance also can be obtained by notching out the corners of the end abutments, as depicted in Figures 1, 38, 53 and 66(a).

A barrier wall can also limit sight distance along the on ramp if the wall is too high or if the on ramp is on a crest vertical curve. In either case, the wall may block the driver's sight distance along the curved turn path. For overpass designs, safety barriers protecting bridge piers, especially those mounted on top of curbed islands, may exceed driver sight lines. Care should be taken to see that this restriction does not occur. For underpass designs, bridge parapet walls may limit sight lines.

To provide safe operations on the on ramp, sight distance equal to or exceeding the stopping sight distance should be provided everywhere along its length. However, the sight distance along a horizontal curve is dependent on the lateral offset to a potential visual obstruction and the degree of curvature of the roadway. The relationships among safe stopping sight distance, minimum lateral offset, and degree of curvature are given in the Green Book (Ref. 1, p. 219) as:

$$M = (5730/D)\,(1 - \cos SD/200) \qquad (12)$$

where M = lateral offset (or middle ordinate), ft; D = degree of curvature, where $D = 5{,}730/R$; R = radius of horizontal curve, ft; S = stopping sight distance, ft; and cos α = cosine of the angle α.

Using Eq. 12, the lateral offset for the range of stopping sight distances and curvature commonly found in practice typically varies between 15 ft and 20 ft. It should be noted that the lateral offset and degree of curvature used in this equation are measured from the centerline of the inside turning lane to the obstruction.

Just as lateral offset is dependent on degree of curvature and stopping sight distance, curvature and sight distance are dependent on the design speed of the roadway. The relationship between stopping sight distance and design speed is based on studies of driver reaction time and acceptable braking levels. In a similar manner, the relationship between curvature and speed is based on studies of driver speed preference and tire-pavement friction on horizontal curves. From these studies, a maximum degree of curvature was defined for each design speed. Speeds faster than the design speed for a given radius would place demands beyond limits considered safe or comfortable for most

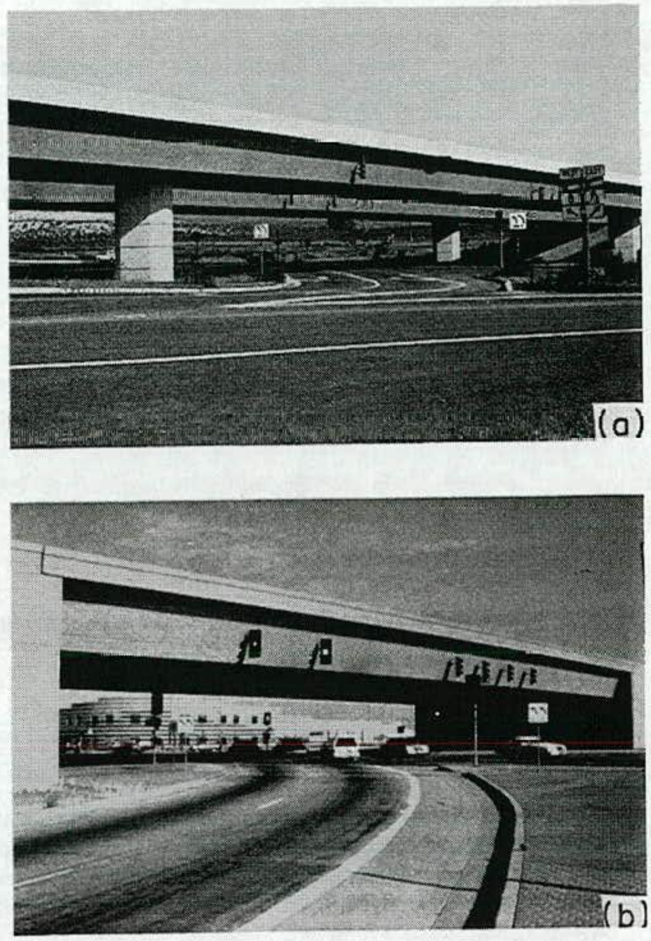


Figure 68. Sight distance through the grade separation structure: (a) structure with open-end spans; (b) structure with closed abutments.

drivers. The relationship between maximum degree of curvature and design speed is given by the following equation:

$$D_{max} = 85{,}660\,(e + f)/V^2 \qquad (13)$$

where: D_{max} = maximum degree of curvature; e = rate of superelevation (typically small on SPUI ramps); f = side friction factor for given design speed; and V = design speed, mph.

Using the previous relationships among maximum degree of curvature, stopping sight distance, and design speed, a minimum value for the middle ordinate can be calculated. This value represents the smallest lateral clearance from the centerline of the inside turning lane to the obstruction that would be permissible for each design speed. If a lateral offset greater than this value can be provided, any degree of curvature (less than the maximum) will provide more than adequate sight distance along the horizontal curve. The minimum lateral offset for each design speed is given in Table 5 for operations along the SPUI ramps.

For the analysis of minimum lateral clearance, a nominal superelevation rate of 2.0 percent was assumed. This assumption is believed to be consistent with the design of most SPUIs. A superelevation of 2.0 percent is usually adequate for drainage purposes and can be easily transitioned into the cross slope of

Table 5. Minimum lateral clearance to sight obstruction.

Design Speed (mph)	Side[1] Friction f	Maximum[2] Curvature (degrees/ft)	Sight Distance[3] Assumed (feet)	Sight Distance[3] Design (feet)	Lateral Clearance[4] Assumed (feet)	Lateral Clearance[4] Design (feet)
20	0.27	62.00 (92)	125	125	20.3	20.3
25	0.23	34.25 (167)	150	150	16.5	16.5
30	0.20	21.00 (273)	200	200	18.1	18.1
35	0.18	14.00 (409)	225	250	15.4	18.9
40	0.16	9.75 (588)	275	325	16.0	22.3

Notes:

1 - Friction factors taken from the AASHTO Green Book *(1, Figure III-21, p. 196)*.

2 - Maximum curvature reported as both degree of curvature and its equivalent minimum radius (radius, in feet is listed in parenthesis). The superelevation is assumed to be 0.02 ft/ft.

3 - Stopping sight distance for Assumed and Design speeds. Assumed speed is equal to the average running speed for low volume conditions.

4 - Minimum lateral clearance calculated using rounded values of stopping sight distance from the AASHTO Green Book *(1, Table III-1, p. 120)*. Lateral clearance measured from centerline of inside turning lane laterally to obstruction.

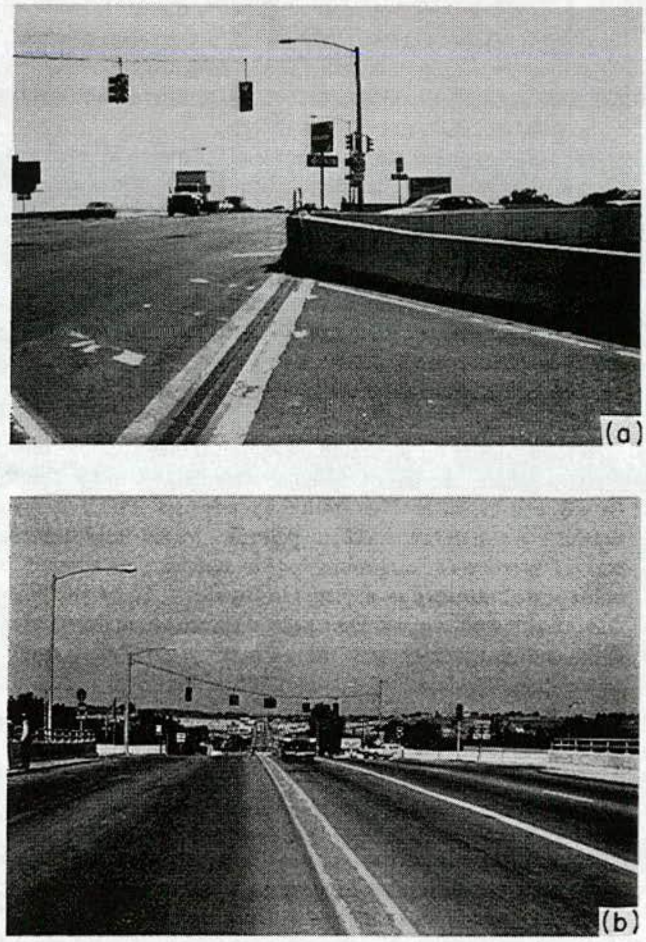


Figure 69. Potential barrier wall restrictions to sight distance: (a) restriction to the off ramp driver's view of cross-road traffic; (b) restriction to cross road driver's view of off-ramp traffic.

the cross road. However, in recognition of the potential use of other superelevation rates, a sensitivity analysis of the results in Table 5 was conducted to determine the effect of superelevation. On the basis of this analysis, it was found that the lateral offset would increase about 1 ft for every 1 percent increase in superelevation because of the assumed higher operating speeds associated with larger superelevation.

During the design process, if the minimum lateral offset reported in Table 5 could be maintained, then adequate stopping sight distance could be ensured for any speed and degree of curvature less than or equal to the design maximums. For example, if the design speed on the ramp is 30 mph and the minimum lateral clearance of 18.1 ft (about 11 ft from the back of curb) is maintained, any curve radii of 273 ft or more would easily provide the required sight distance for ramp drivers traveling along the ramp at or below the ramp design speed.

Off-Ramp Alignment

There are three conditions where sight distance should be evaluated for the off ramp at a SPUI. The first check is along the off ramp to the SPUI. The more critical sight distance control is the decision sight distance needed for the design speed of the ramp. Application of this guideline implies that the off ramp should be at least as long as the required decision sight distance as measured from the point of exit from the major road or the distance from the first point along the ramp where the surface of the intersection area ahead is continuously visible. The selection of off-ramp design speed should be appropriate for the expected operating conditions, including the effects of ramp downgrades from elevated freeway sections.

The second sight distance check is the stopping sight distance along the left turning path. Restrictions to sight distance along this path can occur when a high retaining wall or barrier wall is too close to the inside edge of the turn path. A potential sight distance restriction also may result from a high barrier wall, as shown in Figure 69(a). A better location for barrier walls is shown in Figure 3. This latter type of restriction is identical to that described for the horizontal alignment of the on ramp in the preceding section. The design guidelines recommended in that section should also be applied to the horizontal alignment of the off ramp.

A third sight distance check for the off ramp is the sight distance along the cross road, as shown in Figure 67(a). In this regard, there are two potential restrictions to the off-ramp driver's sight distance. The first pertains to obstructions in the horizontal plane such as the bridge abutment, barrier wall, or landscaping. The second restriction is in the vertical plane and could occur if a high rate of curvature exists in the vertical alignment of the cross road. This type of restriction is illustrated in Figure 69(b) for an underpass SPUI with a fairly high rate of vertical curvature on the cross street.

Both of these latter two sight distance controls should be evaluated for their impact on the off-ramp driver's view along the cross road. In each case, the off-ramp driver should be able to see upstream along the cross road for a distance at least equal to the cross road driver's stopping sight distance. Desirably, the off-ramp driver would have an unobstructed view equal to or exceeding the departure sight distance needed for a left-turn maneuver.

Because of the relative proximity of the bridge abutment to the off-ramp left-turn lane, the restriction to sight distance posed by the bridge abutment could have a significant impact on the operation of the cross-road left-turn movement. Provision of sight distance to this movement along the cross road requires a nominal setback of the bridge structure from the cross road. To verify adequacy of the bridge abutment setback, the sight triangle between the off-ramp driver, approaching driver, and abutment should be checked using the AASHTO Green Book procedures (*1*).

Sight distance along the cross road also can be obstructed by curvature in the cross-road alignment. If the cross road is on a crest curve, the roadway itself may block the off-ramp driver's view of approaching traffic (as shown in Figure 69(a)). On the other hand, if the cross road is on a sag curve, the bridge structure may block the right-turning driver's view. Each of these situations is adequately covered in the AASHTO Green Book for general SPUI applications.

Cross-Road Alignment

There are two sight distance controls that should be evaluated for the cross road in the vicinity of the SPUI. The first sight distance control is the stopping sight distance along the cross road. Restrictions to stopping sight distance along the cross road can occur whenever there is horizontal or vertical curvature in the alignment. The second sight distance control is decision sight distance along the cross road. This sight distance element is different from stopping sight distance because it focuses on accident avoidance by increasing the time needed to make a change in lanes (or direction) rather than the minimum distance needed to stop when an unexpected situation occurs.

The design of the cross road horizontal and vertical alignments in the vicinity of a SPUI is largely the same as at any signalized junction. In this regard, the alignment design should be based primarily on the procedures recommended in the AASHTO Green Book. However, as noted in the preceding section on off-ramp alignment, the design of the cross road vertical alignment also should consider the sight distance needed by the off-ramp right-turn drivers.

Decision sight distance should be included in the design of the cross road in the vicinity of the SPUI because of the SPUI's relatively unusual design and operation. Applications should address those elements of the SPUI that are operationally more complex. Specifically, decision sight distance should be considered when designing the signing plan and at all roadway division points.

U-Turn Lanes

Sight distance for the U-turning driver is also an important consideration at a SPUI having one-way frontage roads, as shown in Figure 70. U-turning drivers must be able to see approaching frontage road traffic and cross-road left-turn traffic to ascertain the safety of their entrance maneuver. If the U-turn lane is designed for YIELD or STOP control where U-turn drivers have direct entry to the frontage road, departure sight distance must be provided. However, if an acceleration lane is provided for the U-turn entrance maneuver, only stopping sight distance would be needed for the MERGE control maneuver onto the frontage road.

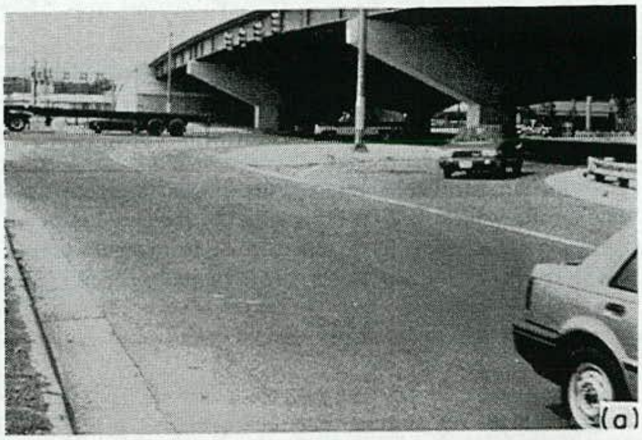


Figure 70. U-turn operations along one-way frontage road: (a) upstream view of merging operations; (b) downstream view of merging operations.

In general, sight lines for a YIELD (or STOP) controlled left turn entry are more complicated than that of the right-turn because the driver must first look through his or her vehicle, i.e., across passengers, headrests, etc., to see approaching traffic. Fortunately, sight lines to approaching frontage road traffic are relatively close to right angles and, thus, pose no difficulty to the U-turning driver. In contrast, sight lines to the approaching cross road left turn are behind and to the right of the driver—perhaps the worst possible location with respect to driver visibility through the vehicle. In addition, sight lines to cross-road left-turning vehicles are often partially obstructed by the columns comprising the bridge bent, and may be limited by traffic safety barriers placed to protect the bridge bents. Because of the difficulties involved with providing adequate sight distance for the U-turn driver at SPUIs, it is recommended that an acceleration lane be provided in the cross section of the outbound, on-ramp frontage road whenever feasible.

LEFT-TURN PATH GEOMETRY

The main goal in the design of a turning roadway is to achieve a proper balance between speed, comfort, safety, and cost. To be sure, related impacts on significant right-of-way and bridge costs are principal trade-off factors. Operationally, a turning vehicle generates a centrifugal force in proportion to its speed which acts on the vehicle and its occupants. As the speed increases, this outwardly acting force can increase to uncomfortable or unsafe levels. In roadway design, tire friction and superelevation are used in combination to minimize the adverse effects of centrifugal force and maintain the desired balance between design goals.

In many ways, the left-turn path design for a SPUI is different from the high-type at-grade intersection (AGI). In particular, the overall length and radii are generally much larger than at AGIs. For example, the typical SPUI has a radius of 200 ft with a length of 180 ft, while the typical AGI has a radius of about 70 ft and a length of 100 ft. Another difference between the SPUI and AGI is the location of the turn path. For the SPUI, part of the turn path is located on the ramp and part is within the intersection conflict area. At an AGI, the turn path is located entirely within the conflict area. At the overpass SPUI the total conflict area is usually hidden from view until the motorist has begun the left turn, unlike at the AGI. Because of these differences, the design of the SPUI turn paths requires additional considerations beyond those applied to turn paths at AGIs. This section discusses the design of the cross-road and off-ramp left-turn paths at the SPUI.

Considerations in Design

The design of an intersection turning roadway has many elements that must be considered. Depending on the particular roadway, these may include: design speed; superelevation rate; side friction factor; turn radius; type of curve (simple radius, compound radii, or broken back); width of turning path; and separation of opposing turn movements.

The first four design elements are related by the laws of motion for turning vehicles. The equation describing this relation in highway design is as follows:

$$e + f = \frac{V^2}{15\,R} \tag{14}$$

where e = rate of roadway superelevation, ft/ft; f = side friction factor for the design speed; V = design speed, mph; and R = radius of curve, ft.

The fifth design element represents a fundamental design decision as to the geometry of the travel path. And, the last two design elements relate to the greater roadway width needed by turning vehicles for purposes of comfort and safety. Each of these elements will be discussed more thoroughly in the following paragraphs.

Design Speed

The selection of a design speed for a given roadway is based primarily on its functional classification with consideration given to the speed preference of the driver population. An added consideration in this selection is the cost of constructing a roadway to a given design speed. In most instances, the more generous design controls that stem from higher design speeds can lead to an exponential increase in construction cost.

Left-turn path design at a SPUI is a good example of the conflicting relationship between design speed and construction cost. This relationship stems from the interaction among the structural, geometric, and operational elements of the SPUI. In terms of conflicting design goals, one goal is to provide a turn path that drivers can negotiate safely and efficiently. A second goal is to provide this operation at minimal cost. To accomplish the first goal, the turn paths must have adequate sight distance; they should be relatively smooth, i.e., of relatively constant radius, and they should have adequate lateral separation from adjacent lanes, opposing lanes, and other physical obstructions. Because bridge structural costs are the greatest SPUI cost component, attempts to minimize cost are naturally directed toward minimizing bridge size. Provision of a more economic bridge should not be attained at the expense of resulting minimal sight distances along the turn paths, broken-back curves, or limited lateral separation of traffic movements.

The choice of design speed for the turn paths is critical to the safety and efficiency of the SPUI's operation; likewise, it can have a significant effect on the cost of the structure. Desirably, a proper balance should be maintained between the design speed and the size of the structure. Observations at more than 30 SPUIs indicate that efficient operations can be achieved when design speeds of 25 to 35 mph are used for left turns. In most cases, this results in a central bridge span of about 200 to 220 ft.

As a part of this research, operational field studies were conducted at three SPUIs and two intersections (*11*). From this data base, 95th percentile left-turn free flowing speeds were related to the radius of the left turn path using statistical regression analysis techniques. A strong statistical relationship ($r^2 = 0.95$) was found. The following equation was developed:

$$V95 = 4.53\,R^{0.357} \tag{15}$$

where $V95$ is the 95 percentile speed (mph) and R is the radius of the left-turn path centerline (ft). A $V95$ of 30 mph would be predicted for a radius of 200 ft, a nominal value of left turn radius for SPUIs. The $V95$ values could be used to guide the selection of turning speeds for SPUI applications.

Superelevation Rate

The left-turn path through the SPUI is complicated by the fact that about one-half of the path lies within the interchange conflict area and the remainder lies on the ramp. Thus, within the interchange, the effective superelevation rate can vary between positive and negative values (depending on the cross slope of the cross road), while the part on the ramp will usually have some positive superelevation.

The design of the turn path should consider the following aspects. Because superelevation is not available for much of the turn path, only side friction should be used when determining the design radius for a given design speed. However, some minimum superelevation should be used on the ramp portion of the turn path for drainage purposes. In this regard, the AASHTO Green

Table 6. Minimum turn radii for intersection curves.

Design Speed (mph)	Side Friction Factor f	Assumed Minimum[1] Superelevation (ft/ft)	Minimum[2] Radius (feet)	Minimum[3] Radius (feet)
10	0.38	0.00	25	25
15	0.32	0.00	50	50
20	0.27	0.02	90	99
25	0.23	0.04	150	181
30	0.20	0.06	230	300
35	0.18	0.08	310	453
40	0.16	0.09	430	667

Source: AASHTO Green Book (*1, Table III-17, p.197*).
Notes:

1 - Superelevation rates assumed by AASHTO.

2 - Minimum radii recommended by AASHTO.

3 - Minimum radius values calculated based on the assumption that no superelevation is available.

Book recommends that a minimum rate of 0.02 ft/ft be used on all curves at intersections (Ref. 1, p. 777). The maximum rate of superelevation recommended by AASHTO is dependent on curve radius and design speed and can be found in the Green Book (Ref. 1, Table IX-12, p. 777).

One other consideration on the use of superelevation on the ramp portion of the turn path is the distance required for superelevation runoff. AASHTO recommends a maximum edge slope ratio of 0.50 and 0.67 percent for design speeds of 50 and 30 mph, respectively (Ref. 1, Table IX-13, p. 778). Thus, a single-lane left-turn path with a design speed of 30 mph and 2 percent superelevation would require at least 35 ft to transition to the flat grade within the central conflict area.

Side Friction Factor and Turn Radius

The side friction factors used for turning roadways are not the same as those used for highway design. This is because drivers accept higher centrifugal forces on curves through intersections. The values of side friction recommended by AASHTO and the corresponding minimum radius of curvature are given in Table 6. This table also illustrates the relationship between design speed, friction, and curve radius for various superelevation rates.

Curve Geometry

Drivers turning at intersections and interchanges generally follow an arcing path of constantly varying radius. In highway design this path can be approximated by inserting a spiral curve between the tangent section and a curve of constant radius. In intersection design, where the turns are tighter and the speeds lower, the path is often approximated by a constant radius or a three-centered curve.

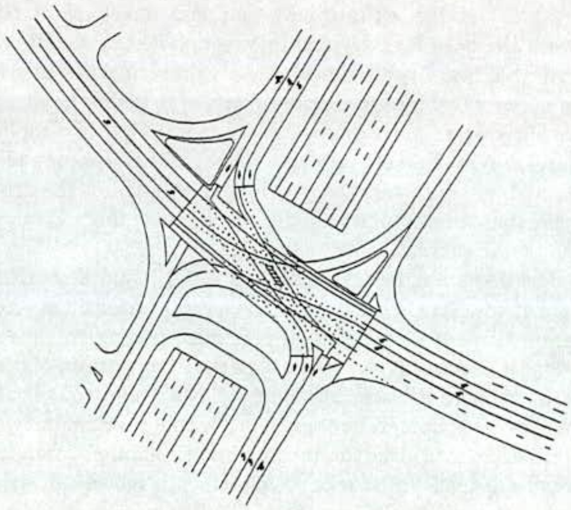
At SPUIs, the left-turn path radii and travel speeds fall between the open highway and intersection design conditions described above. In this regard, drivers on turn paths at a SPUI do not exactly follow a simple curve. On the other hand, speeds are not generally high enough to warrant the use of complicated spiral curves. In most instances, turn paths of simple radii can be used, provided that the lane is wide enough for the driver to make some transition within the traffic lane at the beginning and end of the curve.

The design of the turn path at a SPUI is more complicated than at an at-grade intersection for several reasons. One complication stems from the need to minimize bridge length. As the bridge is shortened, it conflicts with the ramp portion of the turn paths for both left-turn movements. As a result, the turn paths must correspondingly be compressed within the remaining width of the cross road. This compression creates another complication because it tends to push opposing left-turn movement pairs together, thereby minimizing their lateral separation distance. Ultimately, the compression of the left-turn paths can be so great that simple curves can no longer be used and compound or broken-back curves must be considered.

In general, simple radii are most commonly used in the design of left-turn paths at SPUIs and should be preferred over compound curves or broken-back curves. The reason for this preference is based primarily on driver comfort and steering ability. While negotiating a curve of simple radius, the driver needs only to turn the steering wheel and hold it steady through the curve. However, in compound or broken-back curves, the driver must turn, straighten, and then turn the steering wheel again to negotiate the turn path. This added complexity to the driving task in combination with relatively long travel paths, minimal driver guidance, and special design of the SPUI may reduce operational safety. In particular, vehicle encroachment into adjacent or opposing turn lanes is minimized by using simple radius curves for turn paths of recommended width.

Figure 71 illustrates a SPUI that incorporates broken-back curves in the design of the turn paths. Operational experience with this SPUI has been very poor. Observation indicates that drivers are frequently confused as to where to exit the interchange and, as a result, they travel through the interchange slowly and cautiously. In addition, the long tangent sections on the turn paths give drivers the uncomfortable feeling of driving on the wrong side of the road. Drivers are frequently observed driving outside of the lane and on the wrong side of the road. It should be noted that pavement marking lights are being used at this SPUI to delineate the edges of the turn paths but, apparently, have had only marginal impact. Other SPUIs that have used broken-back curves in the design of the left-turn path have also experienced operational problems and have resorted to a less efficient, direction-separated phasing for the left-turn movements to minimize traffic conflicts. Based on these observations, broken-back curves are not recommended for use in the design of the left-turn paths at SPUIs.

Compound curves have also been used in the design of the left-turn paths at SPUIs to minimize bridge length. Similar to broken-back curve design, the left-turn path has two curves of minimal radius; however, instead of connecting these curves with a tangent, another curve of relatively large radius is used. And, like the broken-back curve, similar operational problems can occur on compound curves if the radius of the central curve is significantly larger than that of the curves on either end. In



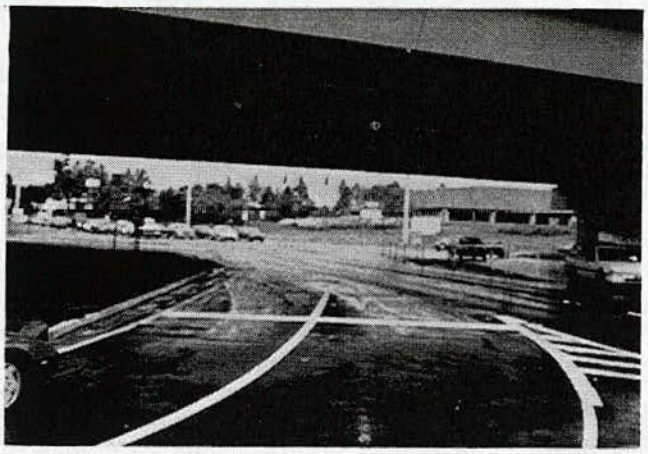


Figure 71. Example of broken back, left turn curvature.

recognition of this problem, AASHTO recommends that the ratio of the radius of the flatter, central curve to the radius of the sharper curve on the end should not exceed 2:1 (Ref. 1, p. 201). For SPUI design, however, one long simple curve is recommended.

Width of Turning Roadway

On turning roadways, the width of the pavement is generally increased for several reasons. One reason is to better accommodate the off-tracking characteristic of vehicles on curves. Another reason for added width on turning roadways exists if the turning roadway has raised curbs. In this situation, the turning roadway must be wide enough for vehicles to pass a stalled vehicle. A final reason pertains to the increased driver comfort associated with curves having a generous lateral offset to adjacent lanes and roadside obstructions.

For these reasons, it is recommended that the left-turn paths at SPUIs be widened beyond the nominal lane width for the tangent section. The design width for the turn path should be based on the design vehicle, the turn radius, and the presence of raised curbs. Methodology for determining pavement widths based on these criteria is described in the Green Book (Ref. 1, pp. 202–211). The design widths determined by this methodology, as applied to the SPUI, are given in Table 7.

Under the Case I classification, the pavement widths are based primarily on the off-tracking width of the design vehicle and have no provision for passing within the lane. This classification is consistent with the operation of the left-turn travel path through the SPUI conflict area. Therefore, it is recommended that Case I be used to determine the width of the travel path through the conflict area.

The Case II and Case III classifications are based on the need to provide adequate width to accommodate two vehicles abreast on the ramp. In particular, Case II allows for a vehicle to pass a stalled vehicle on the turn path, while Case III allows for two full-width traffic lanes. These classifications are more consistent with the design needs of the off-ramp portion of the left-turn path at a SPUI. Therefore, it is recommended that Case II be used to determine the width for single-lane ramps and Case III for dual-lane ramps.

To check the reasonableness of these recommended design widths, the lane widths used at the 36 SPUIs studied for this research were reviewed. Based on this review, it appears that most SPUIs used 12-ft lane widths (outside lanes had an additional 1 to 2 ft for curb and gutter) for the tangent sections of the left-turn paths. However, upon entry to the curve, the travel path was widened to between 14 and 16 ft through the intersection, which is consistent with the Case I classification. On the

Table 7. Design width of pavements for turning roadways.

	Pavement Width (ft)								
	Case I[2]			Case II			Case III		
Radius[1] (ft)	A[3] (ft)	B (ft)	C (ft)	A (ft)	B (ft)	C (ft)	A (ft)	B (ft)	C (ft)
50	18	18	23	23	25	29	31	35	42
75	16	17	19	21	23	27	29	33	37
100	15	16	18	20	22	25	28	31	35
150	14	16	17	19	21	24	27	30	33
200	13	16	16	19	21	23	27	29	31
300	13	15	16	18	20	22	26	28	30
400	13	15	16	18	20	22	26	28	29
500	12	15	15	18	20	22	26	28	29

Source: AASHTO Green Book *(1, Table III-21, p. 209).*
Notes:

1 - Radius is measured to the inner edge of the pavement.

2 - Operational Cases:
- I = No provision for passing a stalled vehicle; one lane, one way.
- II = Provision for passing stalled vehicle; one lane, one way.
- III = Two-lane operation.

3 - Traffic Conditions:
- A = Predominantly passenger cars with some consideration for single-unit trucks.
- B = Sufficient single-unit trucks to govern design with some consideration for semitrailer trucks.
- C = Sufficient bus and combination-type trucks to govern design.

ramp sections, the width of the single-lane ramps varied between 20 and 24 ft and the width of the dual-lane ramps varied between 35 and 48 ft. These widths meet or exceed those given in Table 7 for radii over 150 ft and Type A traffic conditions.

Separation of Opposing Turn Movements

The safety and efficiency of any roadway serving two traffic streams moving in opposite directions is highly dependent on the distance separating the opposing streams. This separation is commonly provided by wide traffic lanes and shoulders. In fact, the effects of lane width and lateral offset on efficiency have been thoroughly studied and documented (Ref. 1, p. 333). Based on previous research, adequate separation distance is generally achieved by the use of lane widths of 12 ft with an additional 6 ft of lateral clearance to obstructions. Although previous research is based on studies of through movements, its results would appear applicable to SPUI turning movements. This section discusses the application of these concepts to the lateral offset between opposing left turn movements at SPUIs.

The combination of phase sequence and geometry of the left-turn movements at a SPUI results in the occurrence of two movements traveling in opposite directions through the interchange. In particular, both cross-road left-turn movements are served during a common phase and, later in the cycle, both off-ramp left-turn movements are served. In each case, the geometry of the left-turn paths is such that turning vehicles pass by one another at some minimal distance near the center of the conflict area. Because provision of this distance often comes at the expense of increased bridge length, it is usually kept to a minimum during the design process.

From a review of past research on lane width and lateral offset, it appears that there has been little, if any, research on the separation needed between opposing left-turn movements. However, research conducted in the earlier days of highway construction indicates that drivers of passenger cars prefer a body clearance of about 5 ft when meeting other passenger cars on tangent highway sections (*27*). Recognizing the added driver apprehension that results from the centrifugal force associated with cornering and the fact that turning drivers are passing on the right side, it is reasonable to assume that lateral clearances in excess of 5 ft are preferred when opposing drivers meet on curves such as those found at the SPUI.

The 36 SPUIs studied for this research were reviewed to determine the amount of separation actually being provided between opposing turn paths. Examination of these SPUIs indicates a wide range of vehicle body clearance distances. Measurements between the near side edges of opposing left-turn vehicles (at their closest point) indicates that separations used in practice range from 5 to 55 ft. The average vehicle clearance is about 18 ft for the off-ramp movements and 22 ft for the cross-road left-turn movements.

If it is assumed that the width of the turn path is consistent with the width identified under Case I in Table 7, there should be about 2 ft between the vehicle body and the outside edge of the turn lane. Thus, a minimum vehicle body clearance of 4 ft would result if the lane widths recommended in Table 7 are used in the design of the opposing turn lanes when they are placed as close together as possible. However, on the basis of driver preference for a 5-ft body clearance on tangent sections and in recognition of increased driver apprehension on curved paths, a minimum body clearance of 10 ft would appear more reasonable for design. If the lane widths in Table 7 are used, a 10-ft body clearance will result if a 6-ft separation distance is provided between the adjacent edge lines of the opposing movements. Therefore, it is recommended that a minimum of 6 ft be provided between the near side edge lines and a minimum 10-ft body clearance for the opposing left turning movements within the SPUI conflict area.

Some agencies may elect to install a divisional island in the central area between the four opposing left turns. If installed, it should be visible and of adequate size, e.g., 100 ft^2, to safely handle the traffic control devices installed thereon.

BRIDGE LENGTH

One of the main operational advantages of the SPUI is its intrinsic ability to serve opposing left-turn movements at moderate speeds during the same signal phase. This is accomplished by inverting the left-turn paths of opposing left-turn movement pairs such that they do not conflict with one another. Consequently, this type of operation leaves only a small unused pavement area within the central intersection conflict area. As a result, when the SPUI design has an overpassing major road, the bridge structure is designed to span the entire conflict area. SPUIs of this type commonly have bridges with central clear spans ranging in length from 200 to 220 ft.

The actual clear span length needed to span over the conflict area is determined by the setback of the bridge abutments (or bents) from the cross road and on-and-off ramps. Thus, the length of the bridge is affected by the cross road width (e.g., number of lanes, lane width, median width), the width and separation of the on-and-off ramps, and the design speeds of the intersection roadways. A minimum lateral clearance distance between the bridge abutments and the ramps must be provided to preclude any restriction to the ramp driver's stopping sight distance, as indicated in Table 5. In addition, a minimum lateral offset distance must be provided as a clear recovery zone for errant vehicles.

Factors Affecting Bridge Length

Many factors affect bridge length during the process of making trade-offs to ascertain the most cost-effective design. Initially, the right-of-way available for a given expenditure level sets the local site parameters for the bridge. Narrower mainline right-of-way usually means longer bridges for a given number of lanes. In locations where adequate right-of-way will permit the ramps to be moved away from the mainline and permit construction without retaining walls, significant reduction in bridge length can be achieved. Another general bridge length parameter is the overall size of the interchange, as defined by the number of lanes required to serve the traffic demand. In this context, the number of mainline lanes is also included in the size estimate. In general, the more total lanes needed for capacity, the longer the bridge will be for a given set of design speeds. Design speed also affects bridge length with higher design speeds generally resulting in longer bridges. Many other site specific factors affect bridge length as the following sections will describe.

The length of the bridge span over the cross road is dependent on the type of support provided. In particular, a single-span bridge commonly uses a vertical retaining wall abutment that is as wide as the overpassing major road. Conversely, with a multispan bridge design, the columns forming the bridge bents are often inset from the edge of the major road. As a result, the bents can be located closer to the cross road which generally leads to a decrease in the length of the center span. An examination of several SPUIs with both types of bridge design indicated that the central span of a multispan bridge is generally 15 to 25 ft shorter in length than a comparable single span bridge. However, the total length of the multispan bridge is generally 150 ft or more longer than the single span bridge.

Because of the sensitivity of bridge support location (and, thus, bridge length) to ramp location and cross road width, any factor that affects ramp location or cross road width will also affect bridge length. One factor found to have a significant effect on ramp location and the effective cross road width is the skew between the intersecting roadway alignments. Skew is defined as the angle between the cross road centerline and a line perpendicular to the major road; a clockwise rotation of the cross road represents a positive skew. A second factor that affects cross road width is the width of the median (as measured at the stop line).

The sensitivity of bridge length to changes in skew angle, median width, and ramp separation was examined for this research. For this examination, geometric relationships between the ramp, cross road, and bridge abutment locations were developed that are sensitive to skew angle and cross-road width. These relationships were used to develop a comprehensive model of SPUI geometry. The predictive capability of the overall model was validated using six SPUIs having a wide range of skew angles and cross-road widths.

Two SPUI scenarios were considered in this examination. The first represents a "small" SPUI which has a four-lane major road, a four-lane cross road, and single-lane left-turn bays. The second scenario represents a "large" SPUI which has a six-lane major road, a six-lane cross road, and dual-lane left-turn bays. These two scenarios essentially span the range of SPUI sizes found during the field study.

Several assumptions were made to facilitate the examination. In particular, it was assumed that the major road had a 16-ft median and two 10-ft shoulders. It was also assumed that the distance separating each ramp and the outside edge of the major road shoulder was 45 ft, that the median nose had a 2.0-ft radius, and that all traffic lanes were 12 ft wide. A 0.0 degree skew was assumed for the examination of median width and an 8-ft median width was assumed for the examination of skew angle. The bridge was assumed to have a single span with a vertical retaining wall abutment. These particular assumptions are representative of typical single-span overpass SPUIs located in urban settings.

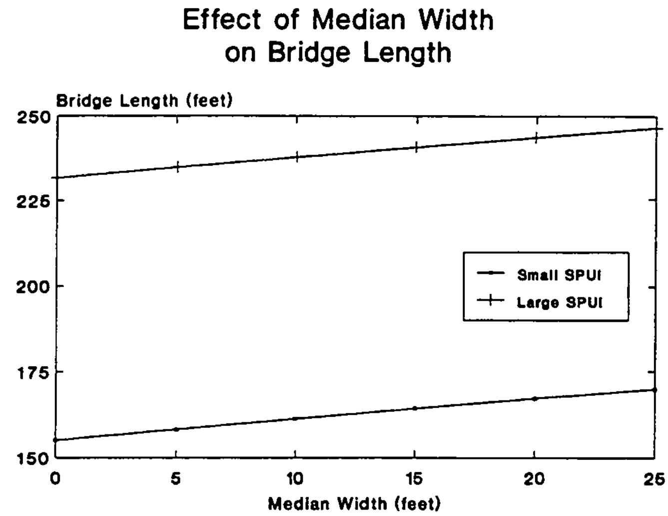


Figure 72. Effect of median width on bridge length.

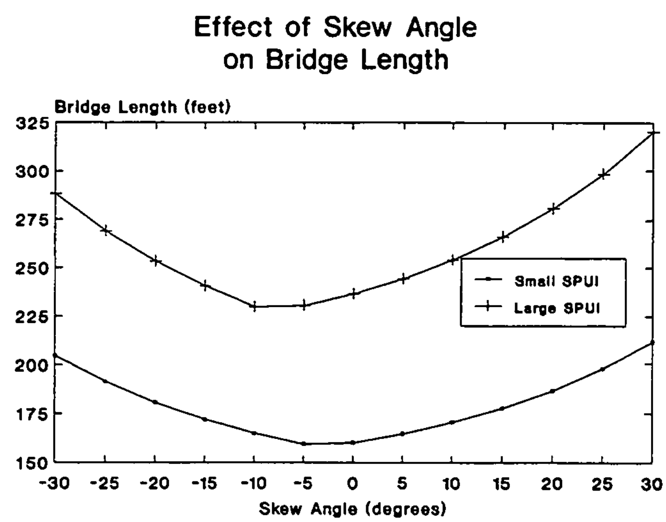


Figure 73. Effect of skew angle on bridge length.

Sensitivity Analysis of Bridge Length to Geometric Variables

The results of the sensitivity analysis of bridge length to median width and skew angle are depicted in Figures 72 and 73. Figure 72 shows the effect of median width on bridge length. Bridge length is observed to increase linearly with increasing median width. The trend for both the large and small SPUI scenarios suggests that bridge length increases about 0.6 ft for each 1-ft increase in median width.

Figure 73 illustrates the effect of skew angle on bridge length. In general, bridge length was found to increase with increasing skew angle. It was also found that a skew angle of about -5.0 deg would result in the shortest bridge length if ramps had the same circular radius. This is the result of an asymmetric physical relationship between the cross-road left-turn path, the off-ramp left-turn path, and the major road. This relationship can be envisioned as a "scissors-effect" wherein the off-ramp and cross-road left-turn paths that begin and end, respectively, on the same side of the cross road act as the blades of a scissors that "cut" (or constrain) the bridge abutment. However, the blades are not of equal length and the hinge point is not at the exact center of the intersection. As a result, rotation of the cross road (relative to the major road) through a negative skew angle would take advantage of this asymmetry and lead to the shortest bridge length. Slight variations in the left turning radii from the same equal radius can also accommodate this small asymmetry and reduce bridge length.

The general trend existing between bridge span and spacing between the ramps, as measured at the end of the bridge, is depicted in Figure 74. This figure was developed from two real-world case studies of actual SPUIs and shows the scissors-effect trends for small differences about the nominal bridge length values of about 200 ft. If proposed bridge lengths are reduced, resulting ramp separation distances, as defined in Figure 11, will be significantly larger. Figure 74 suggests that shorter bridges only come at the expense of more mainline right-of-way for a given set of design speeds and cross road features. The understandable design option is to break the back of the horizontal curves of the higher design speed off-ramp left turns or choose unreasonably low design speeds; however, these design options are not considered operationally desirable.

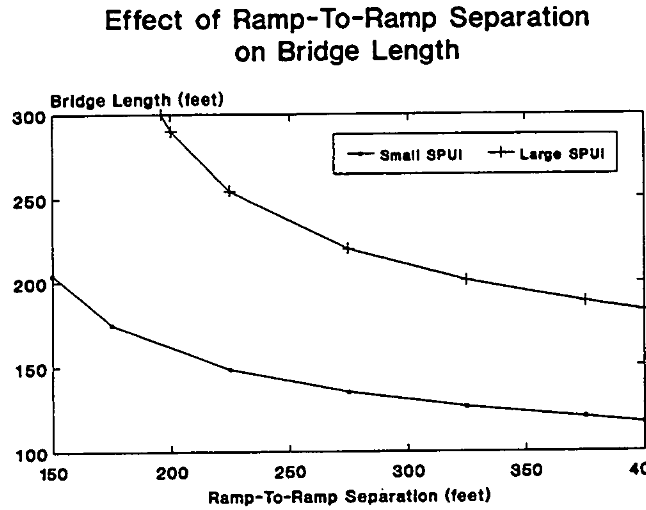


Figure 74. Relationship between bridge span and ramp spacing at bridge abutments for two case studies.

CHAPTER SIX

TRAFFIC CONTROL DEVICE APPLICATION GUIDELINES

Unfamiliar motorists operating through a SPUI for the first time are faced with a new driving experience that can be safely and efficiently handled when all devices in the Traffic Control Plan (TCP) are carefully planned and efficiently applied. The TCP would include the application of all traffic control devices, including all signs, signals and markings, together with the coordinated application of special devices and roadway lighting to provide an optimized driving environment. All traffic control devices should be in harmony and consistently delineate the travel paths of the single-point urban interchange in both day and night visibility conditions.

TRAFFIC CONTROL DEVICES

This chapter presents recommended application guidelines for all traffic control devices typically used at SPUIs. These guidelines are based on the field observations made at more than 20 operational SPUIs around the country, discussions with local traffic engineering personnel, and the 1988 *Manual on Uniform Traffic Control Devices* (*28*). The 1988 MUTCD is considered the controlling standard for application of all traffic control devices in the United States. A combination of text, graphics, and pictures is used to convey the recommended guidelines. Several options, too numerous to identify, would exist for guide signing depending on the site-specific route numbering system. Complementary guidelines for special pavement marking lights and roadway lighting also are provided to complete the system.

Development of Traffic Control Plan

The interchange TCP should be responsive to the class of mainline highway (freeway, expressway, or conventional road) and cross road. The size of the devices used should provide the target value and legibility needed for the expected operating speeds, degree of driver unfamiliarity, and size of the facility. Driver unfamiliarity would likely be higher along interstate freeways, near major airports, and popular tourist areas. Unfamiliar motorists would have greater information needs than would local traffic once the local traffic becomes familiar with the SPUI.

There are significant differences in the design requirements of the TCP depending on whether the SPUI is a mainline overpass or an underpass. The general visibility of the mainline underpass SPUI design is superior to the overpass design, as Figure 75 illustrates. However, shadows will remain at the intersection with any overpass design and should be treated with a high level of room ceiling style lighting underneath the bridge. As this example illustrates, each design type has special design needs. Field observations strongly suggest that the TCP for an overpass SPUI should be developed as if the SPUI is more like a high-type directional interchange rather than like a regular intersection to best serve the traffic.

Mainline Traffic

The mainline traffic has several critical factors that should be considered. Firstly, the interchange signing should be consistent

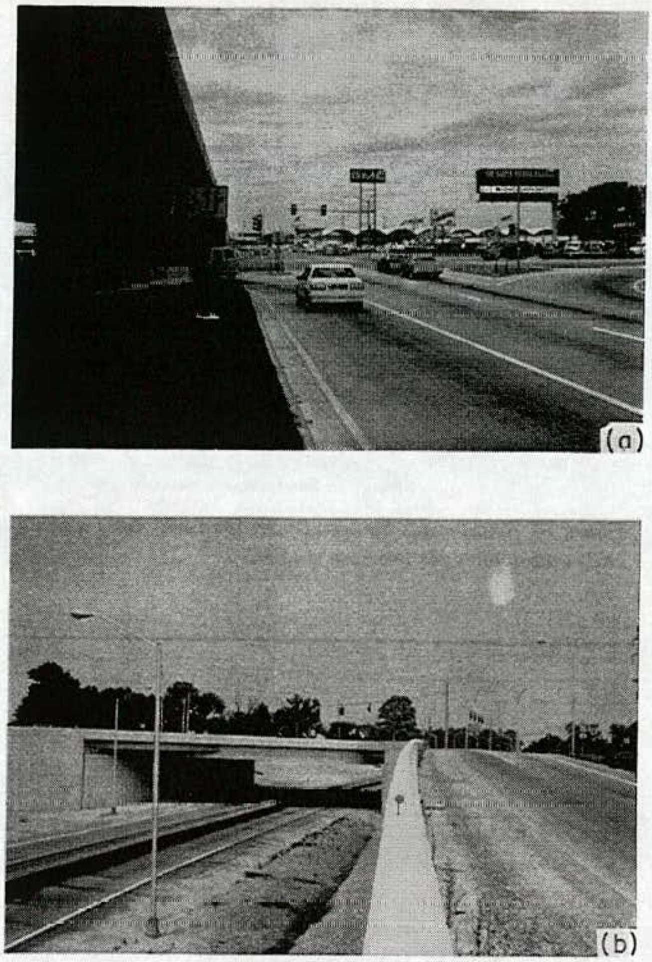


Figure 75. Driver views along off ramp to overpass and underpass: (a) overpass; (b) underpass.

with the class of mainline operation and signing. Mainline traffic exiting from a freeway to the off-ramp may be traveling at a high speed compared to local traffic on a grade-separated city street. Exit ramp traffic operating on downgrade ramps need even more visibility, deceleration distance, and larger traffic control devices. Neither mainline nor cross-road traffic can easily discern the form of the interchange until near or at the signal.

Mainline overpass designs tend to hide the true identity and special geometric features of the SPUI until the exit ramp traffic is near the intersection. Some improvement is provided by open multispan bridges. Traffic signs placed under the overpassing bridges will be difficult to discern in the daytime as Figure 75(a) shows.

On the other hand, mainline underpasses provide a good view of the exit ramp and slower ramp speeds, but the view of the cross-street's geometric features is restricted along the off ramp. If raised islands are used within the intersection area, off-ramp vehicle turning paths also may be difficult to discern when the cross street is also elevated above normal ground level in a crest vertical curve. In this case, left-turn traffic may experience some operational problems.

Cross-Street Traffic

The TCP for the cross street should also be responsive to the special features and informational requirements of the SPUI. The plan should reflect the functional nature of both intersecting highways in the urban network. Again, overpasses and underpasses present different visual cues to the motorists and, likewise, require a different set of operational responses that should be considered when preparing the TCP. Consider the comparable driver views shown in Figure 76.

For mainline SPUI overpass designs, one unusual cross-road operational feature that must be effectively addressed is the preparation and execution of left turns from the cross street to the mainline highway prior to arriving at the SPUI overpass structure. Cross road left turns normally must be made at least 300 ft in advance of the bridge into dual left turn lanes. This advance-turn maneuver is not typical of those made at TUDIs or parclos where the left turns are normally made beyond the overpass. This advance-turn design creates the need for highly visible guide signing having expressway quality legibility on the cross-street approaches to the SPUI. Overhead guide signing is recommended for the cross-street approach. This high-type design is highly recommended when the cross street is six lanes and serves a large number of unfamiliar drivers. Erratic maneu-

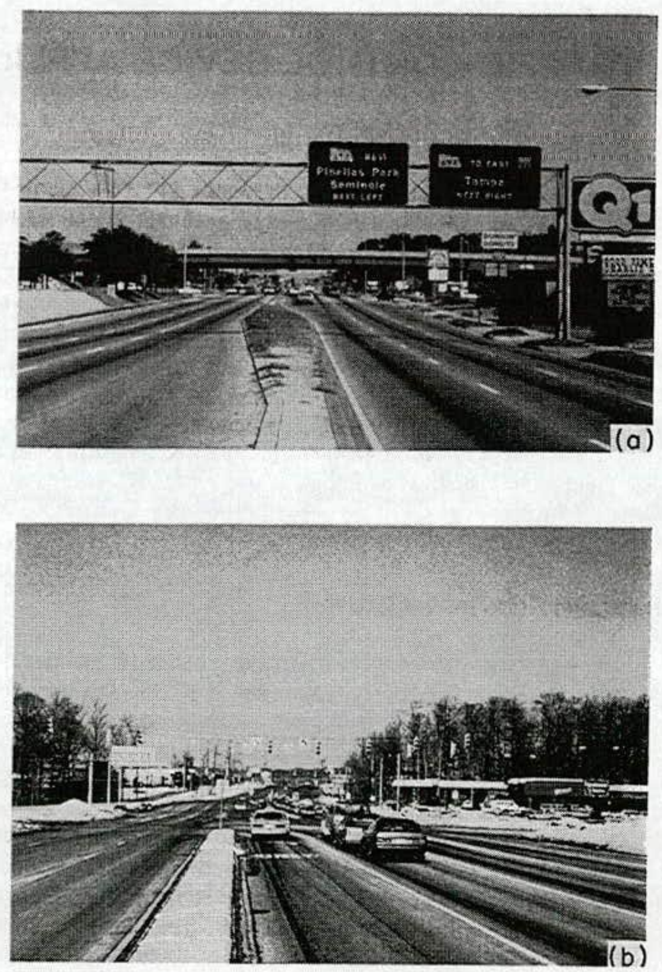


Figure 76. Cross street views along approach to overpass and underpass: (a) overpass; (b) underpass.

vers have been observed at several similar locations at otherwise well-designed SPUIs where conventional road ground-mounted guide signing was used on the cross street to guide approaching left turns.

For mainline underpass designs, the intersection area of the SPUI is very visible to cross-street traffic when the street is not on a crest vertical curve. However, the intersection area does appear void of traditional intersection cues to approaching cross-street traffic, especially when the cross street has only painted left turn markings and when span wires are used to mount the traffic signals. This result is also partly due to the absence of traditional cross street traffic lanes and divisional traffic islands. Thus, the cross road traffic lanes will need a good pavement marking system to identify stopping positions, turning paths, and exit points.

The guide signing, pavement markings, and traffic signal arrows should be designed to provide consistent visual cues that clearly define the desired routes and travel paths for both overpass and underpass designs. Figure 77 presents good examples for both types of interchanges. While overhead lane use assignments would be helpful at the intersection in both cases, overhead guide signing is not required for underpass SPUIs as there are no false cues produced by an overpassing bridge.

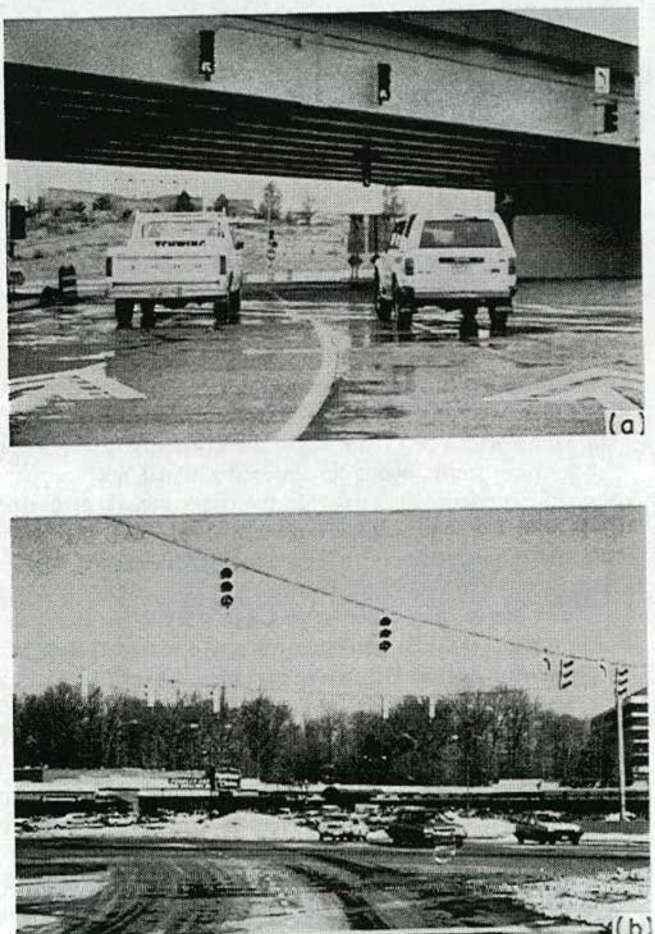


Figure 77. Examples of consistent visual cues provided by traffic control devices for overpass and underpass: (a) overpass; (b) underpass.

Island Treatment

The number of traffic control devices that may be used on a traffic island in a SPUI can be numerous and must be coordinated with all other features on the islands. Some islands may be very large whereas others may be small, depending on the size of the interchange and skew of the highway alignments. Agencies that have a strong, clear zone policy may desire to select geometric designs and alternative TCPs that minimize the number of ground-mounted signs in these islands while still efficiently providing the necessary guidance functions. All traffic control devices should be kept clear of pedestrian and bicycle facilities.

Some designers prefer to use a divisional center island in SPUI design, more frequently with overpass designs. These islands are used primarily to protect traffic signals hung from the overpassing bridge so that reduced ground clearances to the bottom of the signal can be used (thereby reducing bridge height by perhaps 3 ft or more). Signs that define turning paths for left-turn movements may also be placed on the center island. Some divisional separation of the opposing left turns is also provided by the island, but the actual need for this delineation function has not been demonstrated by field studies. Dual cluster signals located over the center island are recommended only for off-ramp left turns, as shown in Figure 78.

The following recommended application guidelines are provided to assist the engineer in preparing a formal TCP for a SPUI as quickly as possible. They are provided to complement and support all MUTCD and local agency traffic control requirements. An overall traffic control plan is graphically provided for the SPUI as a whole and windows are provided on an island-by-island basis to more clearly illustrate the recommended devices. Signing is addressed first followed by signalization and markings.

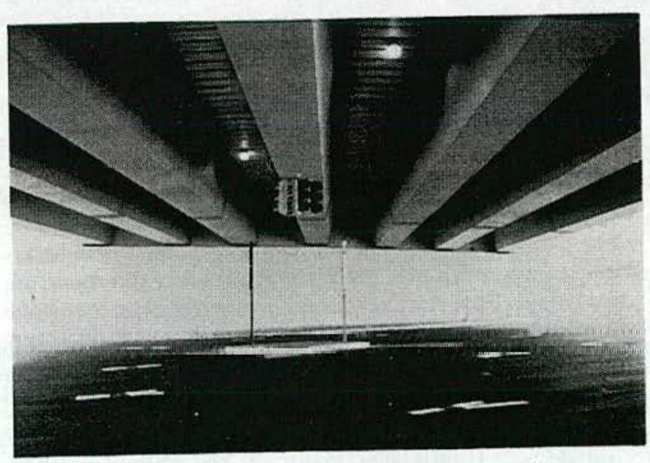


Figure 78. Signal cluster design for off ramp left turn for overpass.

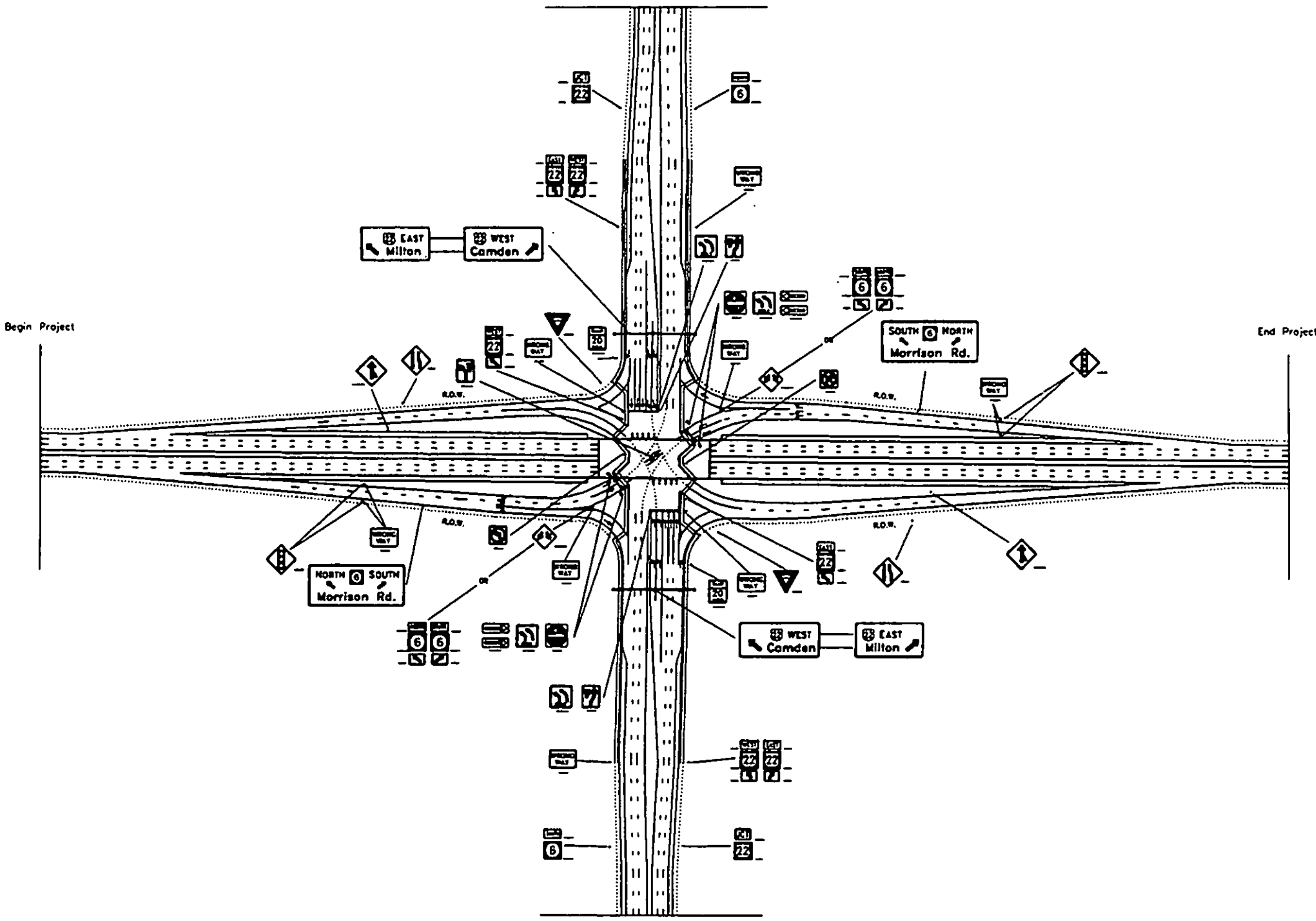


Figure 79. Overall traffic control plan for overpass SPUI3.

Traffic Control Plan

A mainline overpass SPUI3 is used as the mode for presenting the recommended TCP applications. Figure 79 provides the overall plan where it is assumed that the mainline highway U.S. 22 intersects cross arterial S.H. 6, also known locally as Morrison Road. Some options to this plan will be noted as they occur.

Signing

Mainline Exit Ramp. Traffic guide sign applications on these exit ramps must be consistent with the prior mainline signing plan. In addition, motorists may not expect a split exit ramp design prior to reaching the intersection; therefore, the cross street guidance information of route designation (if numbered), local street name (if previously signed), cardinal directions, and destinations (if appropriate) should be provided prior to reaching the interchange gore. SPUIs having dual left-turn lanes or one-way frontage roads should also have highly visible lane use assignment signing provided. Overhead signing is recommended with dual-dual turning lanes or frontage road systems which should have U-turn lanes. Figures 51 and 66 provide examples of good lane use signing along the off ramps.

Additional mainline exit ramp signing should be provided to warn exiting traffic of the approaching signal ahead and warn wrong way operating vehicles. The SIGNAL AHEAD sign, W3-3, should be placed on both sides of the exit ramp to better warn left turning traffic about an upcoming signal not adequately visible. Experience suggests that the traffic signals at overpass SPUIs with frontage roads are usually visible and, therefore, these warning signs may not be needed (see, for example, Figure 51).

At least one set of WRONG WAY signs, R5-9, should be used to sign the exit ramp against wrong way operations. Two signs should be used, one on each side of the ramp, because prohibited movements could enter the ramp from turning roadways on each side of the ramp. These signs are placed upstream of the DO NOT ENTER signs, R5-1. WRONG WAY signs are often placed on the back side of other signs needed for traffic control of the normal exiting traffic, as suggested in Figures 79 and 80, for the dual W3-3 signs. Additional signage may be used across the gore of the large refuge islands described below.

Island 1—Exit Ramp Refuge Island to Cross Street. Figure 81 presents a graphic window on the recommended TCP for this island and surrounding area. A signing issue is: Does the cross street divisional island protect the right-turn lane against wrong-way entry from the cross street left turns? In any case, flush

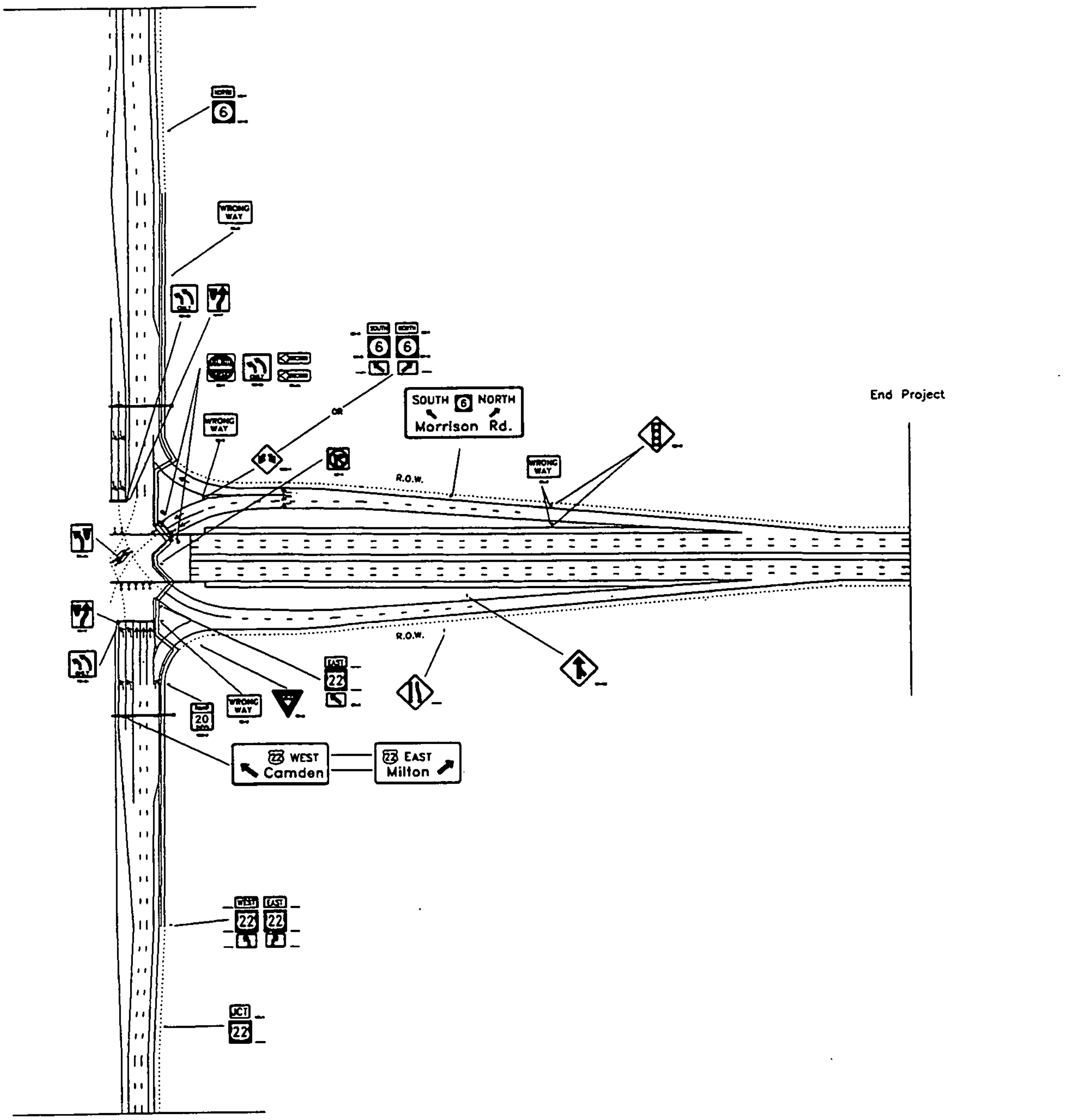


Figure 80. Segment of traffic control plan.

islands will require wrong way entry protection using R5-1 signs. Another key issue is: Will the right-turn movement be a "free right turn" or will it be signalized? Higher traffic capacity of the SPUI depends on having free right turns. Typically, recommended regulatory, warning, and guide sign applications include the following:

1. Regulatory signs: DO NOT ENTER, R5-1; two signs should be used to guard left-turn lane; and possibly two signs to guard the right-turn lane, depending on the cross street divisional island and Island 1 designs. LANE-USE, R3-8a; may be used to define dual left turn lanes and travel path, and to permit or restrict U-turns. These are usually placed on the back of R5-1 signs. Upward pointing arrows should be consistent with all other arrow indications, including signal lens arrows for left turns. ONE-WAY ARROW, R6-1; should be used with R5-1 or under post-mounted signals. Use alone on one-way frontage roads (Figure 46(b)). WRONG WAY, R5-9; a minimum of two signs should be used to guard exit ramp. These may be conveniently located on the back of dual W3-3 signs. One may be added to the W12-1 sign at the nose of the large island. If overhead lane assignments are used on the off ramp (Figure 66), then they may be added to the backsides.

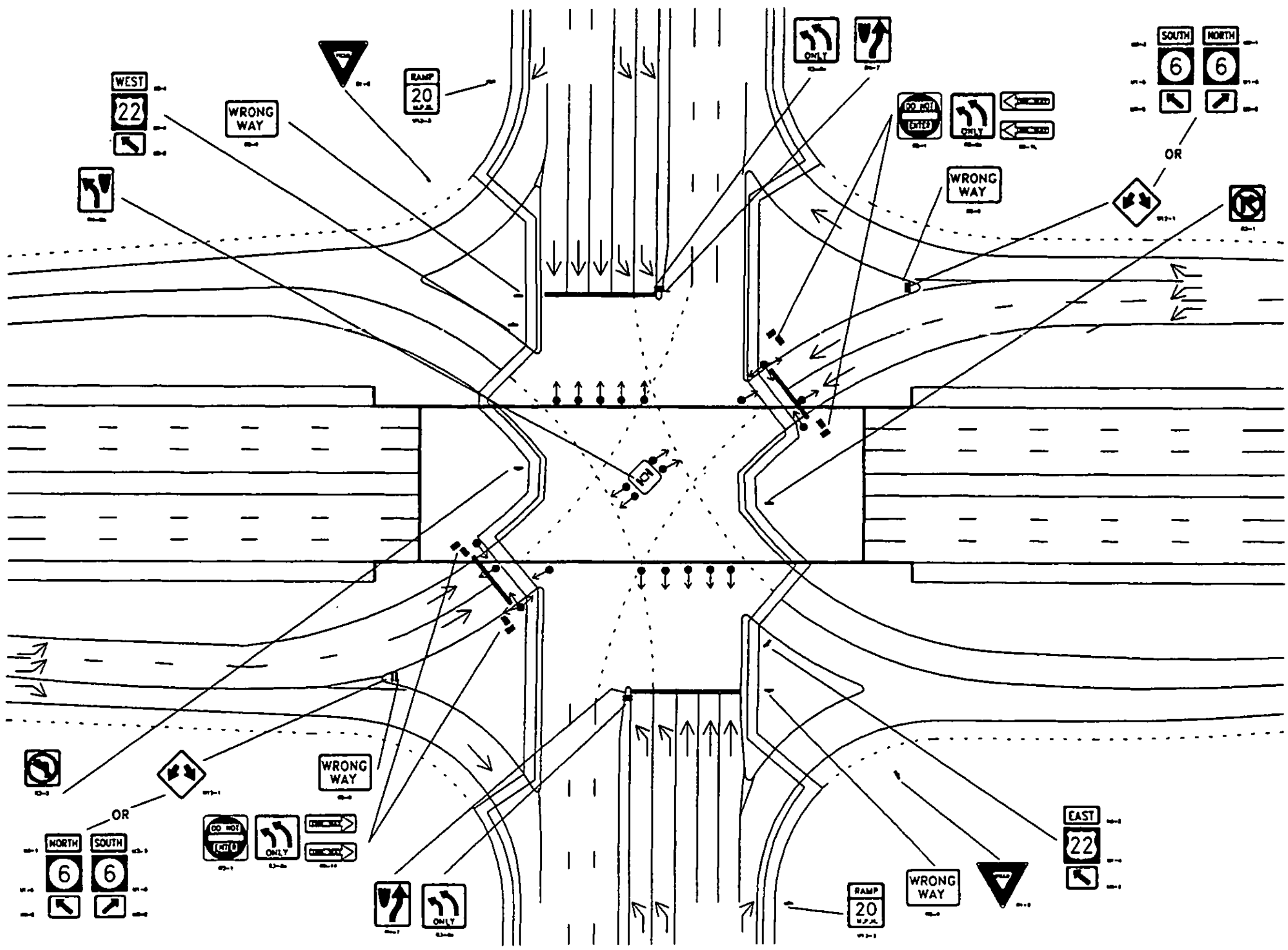


Figure 81. Traffic control plan for central area.

2. Warnings signs: DOUBLE ARROW, W12-1, + OBJECT MARKER OM-1 may be used to define a large refuge island gore points to approaching left and right turns. Field applications suggest that a conventional road or cross street guide sign assembly also appears to provide equivalent gore delineation plus guidance. The W12-1 sign and a route turn assembly should not be used together because their arrow pointers are not consistent. One points down to warn and the other points up to guide.

3. Guide signs: Either a route directional assembly, M1-4 + M6-2, or directional sign, D-1 or D-3, depending on the route designation of the cross street, has been effectively used to both delineate the bifurcation of the exit ramp and to provide useful guidance information. A WRONG WAY sign has been used on the backside of these signs.

Island 2 — Interior Mainline or Bridge Parapet Nose. A true island exists only when U-turn lanes are present. This area serves to delineate the exit-ramp left-turn path into the SPUI and the cross-street left-turn path onto the mainline entrance ramp. Its size depends on the size of the mainline highway. If one-way frontage roads and a U-turn lane are present, additional signing will be required. Any sign placed under a large bridge will be somewhat difficult to discern by cross-road traffic. Green guide signs are more difficult to read in shadow and should not be used underneath the bridge.

1. Regulatory signs: DO NOT ENTER, R5-1; protects off-ramp left turns from wrong way entry. Place one on each side of the off-ramp left-turn lane at the stop line. LANE-USE, R3-8a; may be used to define dual left turn lanes and travel path, and to permit or restrict U-turns. These are usually placed on the back of R5-1 signs. Upward pointing arrows should be consistent with all other arrow indications, including signal lens arrows for left turns. ONE WAY ARROW, R6-1; generally used with R5-1 or signal. NO RIGHT TURN, R3-1; may be used on the nose to restrict cross street right turns from wrong way entry into the left-turn approach.

2. Warning signs: No need evident.

3. Guide signs: None recommended in this area for overpasses. For underpasses, may consider using for guide signs on bridges.

Island 3 — Cross Street Right Turn Refuge Island. This island serves to define the outer paths of the cross street right turns and the outbound left turns from the opposing direction. It is usually clear of the overpassing bridge but its surface may not be readily visible to the far side cross street left turns in underpass designs.

1. Regulatory signs: None usually recommended. Some designs providing large islands have used a wrong way R5-9 sign beyond a left turn exit directional assembly at the nose to protect cross street approach traffic from wayward exit ramp left turns.

2. Warning signs: MERGE, W4-1 or ADDED LANE, W4-3, is used as appropriate for exit to on-ramp geometric design at junction of left-turn and right-turn lanes. Additional warning signs are used along the entrance ramp to identify lane drops, if any, and merging traffic.

3. Guide signs: Some guidance device for cross-street left turns should be provided at the nose facing the far-side cross-street left turns. The device may be a left-turn route directional assembly or a left-turn traffic signal. Use a 45 deg upward pointing directional arrow in the direction of the turn, consistent with all other turn indications. Directional turn assemblies for cross-street approach-traffic left turns and right turns have been used at the nose of large islands, but they are not generally recommended in urban areas because of their ineffectiveness.

Island 4—Cross Street Median Divisional Island for Left Turns. This island is the common extension of the cross street median along the approach to the SPUI. For divisional islands of appropriate size, it should contain, at most, only two signs, back to back, at the nose.

1. Regulatory signs: LANE USE CONTROL, R3-8a; may be used on the divisional median island to define the desired left turn usage. The sign provides positive guidance for dual left turns. It is normally a backside sign to the following KEEP RIGHT sign (see Figures 20 and 77). The R3-8a sign diminishes the need for additional lane use signing for the cross street placed in the intersection, either on the bridge facia or on span wires with the traffic signals. In any use, all arrows should have consistent left turn directional orientation. STOP HERE ON RED, R10-6; may be used instead of R3-8a sign at an underpass to clearly identify the desired left turn stopping position. KEEP RIGHT SIGN, R4-7, should be used on the outbound nose of this island to define departure paths on the cross street. It is not used with narrow medians or undivided roadways where overhead lane use and directional turn assignments are provided by structures, mast arms, or span wires at these SPUIs (see Figures 44 and 45).

2. Warning signs: None normally used.

3. Guide signs: May be used to support overhead guide sign structures. May have a ground-mounted guide sign for left turns on very wide medians. Good designs are shown in Figures 49(a) and 49(c).

Island 5—Center of SPUI Intersection Island. Sometimes a center island is used to protect an overhead signal cluster on overpasses and to better delineate left-turn paths for all designs. The center island does not appear to be needed for underpass designs and should have at least 100 sq ft surface area and be well lighted in overpass designs where used.

1. Regulatory signs: KEEP LEFT, R4-8a, signs are more frequently used for off-ramp left turns and sometimes for cross-street traffic. Recent designs have slightly modified the R4-8 sign to more clearly denote the desired left-turn path so that the arrow points toward the exit and not toward the approaching traffic (compare Figure 47(a) with Figure 52(b)). While this is a slight modification to the R4-8, it appears to be a desired improvement for SPUIs. It is much more consistent with the other directional pointers. Other informational signs have included LEFT TURNS KEEP LEFT or FOR LEFT TURNS ONLY. Object markers have been added to the sign with unknown success.

2. Warning signs: None used. As noted above, object markers (OM-1) have been used to improve visibility of existing signs.

3. Guide signs: None used.

Structures—Various Structures or Supports. These are available for supporting signing, depending on the type of SPUI. Most notably, the overpassing bridge structure is normally used to mount most traffic signals, as many of the previous figures show (see Figures 47 and 55, in particular). Lane use signing has been added to some cross street structures, but never for the off ramps, as shown in Figure 44.

For underpass designs, signals are usually supported from mast arms, span wires or small structures as shown in Figures 57 and 61. Lane use signing may be added to the display to provide a clearer definition of the intersection and travel paths.

Signalization

The signalization used depends on whether it will be a SPUI3 or a SPUI4 interchange, that is, one without or with one-way frontage roads. Most designs probably will be a SPUI3 in the future. An additional parameter to the design will be whether the signal will be coordinated into an existing cross arterial progressive signal system. Additional complications can arise due to local emergency (fire, medical, police) and railroad signal preemption needs, as illustrated in Figure 82. Typical phasing

Figure 82. Special signalization may be required.

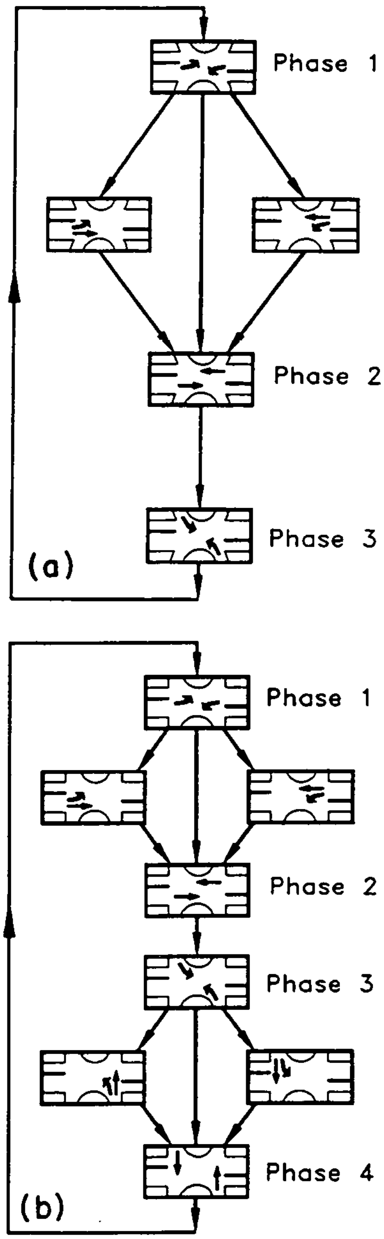


Figure 83. Typical SPUI3 and SPUI4 signal phasings of protected movements: (a) typical SPUI3 signal phasing; (b) typical SPUI4 signal phasing.

plans used for SPUI3 and SPUI4 interchanges are presented in Figure 83 (also refer to Figures 34 and 35 for other SPUI4 examples).

The SPUI3 phasing sequence shown in Figure 83(a) is almost always used. In two cases, the lead-lag phasing sequence has been used. The recommended traffic controller unit (timer) is a programmable "eight-phase, dual-ring, microprocessor-based, full-actuated" timing unit. Phases not used are simply deleted using programmed omits. Modern closed-loop or Type 170 signal systems can be selected, as desired, for their remote monitoring and control capabilities.

There are several topic items in SPUI signalization that merit special attention. These include timing the red clearance interval, control of off-ramp right turns, and the issue of pedestrian crossings of the cross arterial street. Field observations suggest that most other aspects of the signal timing are rather routine traffic-actuated control requirements. One exception would be if airport runway lights were added to complement the conventional pavement marking system.

A critical and difficult signal timing task for SPUI3s (and SPUI4s) is the task of programming the red clearance intervals. The "from-to" phase red clearances should provide safe clearance between traffic movements legally traveling through the control area of the intersection and the next conflicting controlled movement. The trade-off is between longer clearances to provide safer clearances and the need to minimize phase lost time to provide higher capacity. Because the turning paths of the SPUI are often more than 150 ft, red clearances longer than for normal intersections will probably have to be used. See Refs. 17 and 28 and Chapter Three of this text for further guidance on the subject of signal change intervals and red clearances.

The issue of signalizing the right turn from the off ramp to the cross street is also important for several reasons. If pedestrians are present, some agencies may have to signalize the right-turn movement to provide controlled passage. The field survey in general did not detect the operational need for this treatment. In fact, in one case signalized right turns were removed because of their negative impacts on cross-street right-turn capacity and overall interchange capacity. The apparent concern is that a signalized right turn greatly expands the legal definition of the intersection control area, thereby significantly increasing the red clearance legally required and traffic capacity lost. These losses in capacity can be observed in Figure 63 where roughly another 100 ft of intersection clearance would be added to the separation distance scale because of the signalized right turns.

A potential operational problem related to the question of signalizing the off-ramp right turns is possible inadequate capacity, particularly with single, right turn lane designs. As discussed in Chapter Three, the off-ramp right turns do not have a parent phase to provide protected entry, whereas the cross street does. This may produce a situation where the off-ramp right turns simply do not have adequate YIELD-entry merging capacity during high-volume conditions. In addition, the erratic nature of the entry may produce a high accident profile in the downstream merging and weaving area. Rear-end accidents at the merge may also be a problem.

Should this operational problem arise, it can be treated without signalizing the off-ramp right turns using the following control techniques. A queue detector is located in the off-ramp right-turn lane and connected to the adjacent cross-street left-turn phase. This delayed-call queue detector should be located perhaps 50 ft upstream from the stop line (to detect the presence of the second or third vehicle stopped in queue). A delayed call of perhaps 6 sec would be adequate for normal 6 ft by 6 ft inductive loop detector design. If the queue remains over the loop for 6 sec or more during the cross street left turn red, a call is placed for the left turn phase to provide "protected" right turns. If the left turn phase is already green, the "delay inhibit" or defeat feature of the detector-controller system (sometimes called the Pin J feature monitoring the On state of the left turn phase) should be used to turn off the delay feature during green, so that the right turn calls are immediately recognized to extend the cross-street left-turn phase until gap out. These features will provide additional movement capacity only when needed by just monitoring the queueing status of the right turn. Single vehicles stopping in line to make a right turn will still enter under YIELD control.

Some consideration needs to be given to the problem that pedestrians may have in crossing the arterial at the SPUI3 (see Figure 42 for a previous discussion of this issue). Basically, the three-phase SPUI3 does not provide a through phase on the ramps for which to conveniently overlap a pedestrian crossing phase. The design of a pedestrian crossing phase is complicated by the fact that all six SPUI3 movements will conflict with a total pedestrian crossing movement from shoulder to shoulder. In this case, no traffic phase could simultaneously operate. Pedestrian signal control only between the refuge islands is less undesirable because it requires smaller crossing time and provides the possibility of driving one nonconflicting left turn traffic phase. Use of a separate pedestrian signal phase overlapped with a blank through phase is suggested.

Only one SPUI3 in the United States is known to have the above pedestrian crossing signal for the cross street at the intersection. Most designs are simply silent with no crosswalks provided for the cross street but they are usually used parallel to it. Pedestrian signals may or may not have been used parallel to the cross street.

The physical location of the traffic signals depends primarily on whether the SPUI is an overpass or underpass. Overpass designs of SPUI3s usually place the cross street signals on the bridge, as depicted in Figures 55(b) and 56(a). In earlier SPUI3 designs, one signal was usually placed on the bridge over the off-ramp left-turn lane(s), but more recent designs place one signal over each left turning lane (see Figures 2(a) and 55(b). A dual signal cluster for the off-ramp left turns is still usually hung over the center island to provide redundant forward signal visibility from the stop line. In one restricted design case, a state mounted, dual, off ramp left turn signal faces horizontally on the lower bridge flanges. Other sites have avoided placing any signals to the underside of the bridge by using pedestal-mounted signals (for example, see Figure 47(b)). For the overpass SPUI3, other signals may be desired depending on site visibility along the off ramps and turning lanes. One leading signal head has been placed in the divisional island of the off-ramp to provide advanced visibility. Where used, it should be placed to the left of the centerline of the off-ramp, preferably near the left side of the island. If pedestrian signals are used, the signal may be combined with the pedestrian signal and the R 6-1 signs in a neat assembly, as shown in Figure 55(b). Some designers have added a backside left turn signal for the opposing left turn when no central cluster signal is used. Longer yellow and reduced red clearance intervals may be desired for these far side left turn signals through the use of external timers to minimize the chances of a late arriving motorist stopping within the intersection.

Underpass designs typically have used much simpler signal designs. Most have used some form of span wire pattern with dual signal indications as a minimum, usually one for each lane. Figures 3, 18, and 45 show typical field examples. However, in some urban areas where span wire designs are seldom used, pedestal or post-mounted signals are used. Far side signals will be required to control cross-road and off-ramp left turns when no center of intersection left turn signal is present.

All SPUI4s observed in this study had one-way frontage roads and were all overpass designs. Almost every site had a different signal design scheme, as noted in Figures 33 to 35. Most were not affixed to the bridge, but some were. Local designers apparently made their own decisions and no consensus was noted. The large steel bridges often used may have had some influence on the final signal design. A controlling parameter is that the one-way frontage road through movements cannot be signalized from the bridge; therefore, some form of external support is needed, whether it is span wire or not. If pedestrian signals are used, steel poles with mast arms probably will be used to simplify the design. The bridge may still be used for signalizing the cross street approaches (see Figure 48(a)) and some form of left turn signals will be needed for the off-ramp left turns, either cluster signals (see Figure 54), horizontally mounted heads, or far side arrangements. Be advised that no phasing should ever suggest that passing motorists should stop within the intersection control area. Some SPUI4 phasing sequences (see Figure 35) were claimed to be superior to others in this regard and to have lower clearance intervals. However, these claims were not examined in this study. To be sure, lead-lag split phasing will separate any opposing left turn conflicts but at some loss in interchange capacity. This may be desirable with restricted left turn geometrics.

The designer of signal systems should always be aware of the MUTCD guidelines for signal systems given in Part 4-B therein (*28*). The MUTCD states that the primary consideration in signal face (head) placement shall be visibility. In particular, general visibility requirements of signal faces are provided as related to 85 percentile approach speed (Table 4-1 in MUTCD). The MUTCD notes that dual left turn off ramps, as at SPUI3s, should be treated basically as through lanes for visibility analysis, requiring at least two signal faces of which both should be visible for the distances given in Table 4-1, therein, as related to approach speed. Where suitable visibility requirements of Table 4-1 cannot be met, a suitable sign(s) shall be erected to warn approaching traffic. MUTCD Part 4B-11-12.a should not apply in restricting the desired use of up-left pointing, left-turn arrows on an advanced left turn signal mounted nearby on the right-side refuge island at SPUI3s because no through traffic flow (or lanes) occurs on the turning lanes that might become confused, an implied possibility in the 12.a restriction. The desired spatial locations of signal faces are related to intersection geometrics and approach alignment (Figure 4-2 in MUTCD). One important design control arising here is the 40-ft minimum distance required from the stop line to the primary signal faces that control the movement. A minimum offset to cross street bridge facia supporting through lane traffic signals is implied. Signal face height requirements are related to their location on the roadway. Signal faces not mounted over a roadway shall be at least 8 ft but not more than 15 ft above islands; whereas, the bottom of the housing of a vehicle signal face suspended over a roadway shall be at least 15 ft but not more than 19 ft above the pavement grade at the center of the roadway. Signals placed underneath an overpassing bridge over an adequate center island are governed by the former mounting height, normally 14 to 15 feet. Other signal design requirements pertinent to SPUI applications are given in Part 4-B of the MUTCD.

Pavement Markings and Embedded Pavement Marking Lights

As many of the previous figures demonstrate, a wide variety of pavement marking designs have been used to mark the desired turning paths of vehicles traveling through the SPUI. The wide open space underneath an overpassing bridge is the principal area of concern, primarily for the left turns. Dual left turn lanes offer greater need for positive guidance of turning paths. In spite of initial concerns expressed by local engineers, lane tracking problems associated with SPUI pavement markings have not been as big a potential safety problem as have been restricted geometric designs of the turning lanes and use of conventional road signing in high volume urban conditions.

To alleviate some understandable concerns by decision-makers with the first two SPUIs ever built, airport runway lights were embedded in the pavement to ensure that everything had been done to make them work safely when opened to traffic. There is no objective evidence that these lights were ever needed or have improved the overall safety of the interchange. In several cases where they have been installed, they have been allowed to become inoperative, with no noticeable increase in traffic accidents reported. Field observations reveal that even the newest pavement marker lights are nominally ineffective in the daytime (see Figure 52(b) for an early pavement marker application). Further design details of these devices will be provided later in this section.

The main guidelines that can be offered for pavement markings is to use the system that field experience has shown works best in the local area for similar circumstances. SPUIs generate considerable traffic volumes over a concentrated area coming from several directions; therefore, frequent maintenance should be expected to retain a reasonable percentage of the initial markings' visibility.

Some observations on the marking of left-turn lanes are drawn from the field survey. About every conceivable marking design for left-turn paths has been tried, from solid wide lane lines on all sides to only a dotted line for the center line of dual turn lanes. At least at the opening of the SPUI, it would appear that there should be at least one lane line marking each left-turn lane throughout the central area. With single left turning lanes, the inside lane line should receive priority for marking; with dual left turning lanes, the center lane line should receive highest priority. There does not appear to be a need to mark the outer lane lines of well-designed turning paths, although most agencies apparently do. Too many lane lines in the central area may confuse passing motorists. However, wide stop lines are encouraged to define the starting points of the turning maneuvers.

Pavement marking lights have been used in SPUIs to delineate primarily the paths of the left turning movements. The pavement marker lighting systems all used the Federal Aviation Administration's standard runway marker lights. Systems used FAA L-868 bases and L-852 B light fixtures. Costs were about $1,000 per unit installed with a typical installation having about 50 lights embedded in the pavement with a large number of backup spares. Only left turn lanes have been lighted. Figure 84 shows an example of the first ones installed in Clearwater, Florida.

The most recent product specifications for the highway guidance lights, now made almost exclusively by Crouse-Hinds Inc., follow. The newest lampholder assembly is 12 in. in diameter and holds a 62-W prefocused quartz-halogen projector lamp with a rated life of 1,000 hours. Lens screens are available in green, red, or yellow to delineate lines in a variety of colors. All SPUI operations use the green filter. The green color does not meet MUTCD standards for white lane lines (*28*) but the color is noticeable at night.

The new lampholder fits on top of a 16-in. base buried beneath the pavement in a 16-in. diameter hole. Enclosed in this base may be a 120/10 V electrical transformer and related conductors to operate the lamp. Openings in the base allow conduit entry so that all lamp wiring may be drawn underground to a common service cabinet. Timed dimmer switches may be devised to adjust voltages and sequence the display following green onset. The transformers apparently can be moved to a roadside cabinet if the timed serial onset of the individual lights is not used following green onset. All lamps are turned off at the end of the phase.

Figure 84. Embedded pavement marking lights.

Most early users had difficulty keeping their pavement marking lights operating. Invading moisture was a constant problem. Some newer designs provide a small drip current in the lamps to keep them warm and dry. Recent solid-state designs have wattage control timing that has improved the overall reliability. However, pavement debris still routinely covers many lights because of the recessed design (to protect from snowplows) of the lampholders. Street sweepers routinely fill the cavity. Snow cover and snowplowing operations are also other considerations in northern climates. Thus, providing diligent maintenance has been a challenging task made somewhat hazardous by the fact that the markers are firmly embedded in the pavement in the middle of a busy intersection, which cannot be easily barricaded for service.

Several transportation agencies have considerable experience with the lights and consultation with them is encouraged to obtain the benefit of their expertise. The City of Clearwater, Florida, has gone through two generations of these systems and has the longest experience record with them. They recently rebuilt their initial system and seem to be happy with it. Colorado DOT and the City of Colorado Springs have had cold weather experience with the lights and have made several modifications to the design. CDOT removed the electrical transformers normally found in the mounting base to a roadside cabinet. Suggestions by several users to eliminate the base entirely and use only a covered lampholder assembly specially designed to minimize dirt and debris retention have been seriously proposed. Other users encourage the use of portland cement concrete (PCC) pavement under the bridge to minimize the need for frequent repaving and improve brightness under the bridge. This spot PCC paving should be considered in urban areas.

No studies have been conducted as to the cost effectiveness of these lighted marker systems. They are known to be relatively expensive and difficult to maintain by inexperienced technicians, however. They should not be considered, when used in combination with high wattage wall-packs, as replacements for good lighting underneath an overpass in the central area of the SPUI. They may be beneficial in guiding motorists along broken-back curves (small-large-small radii) sometimes used on off-ramps where economics dictate that restricted geometric features must be used at that particular location. Our observations and discussions with operating agencies do not indicate that pavement marking lights are needed for SPUIs having nominal design features, standard pavement markings, and good roadway lighting.

ROADWAY LIGHTING GUIDELINES

Roadway lighting is normally used at SPUIs to identify the physical features of the interchange at night and to provide a safe and efficient operational environment. Most engineers have used conventional roadway lighting systems without the use of pavement marking lights. Only 31 percent of the SPUIs in this study's data base have pavement marking lights installed in any form. Conventional roadway lighting is normally used with special emphasis given to the central area of the SPUI and to route junction points. High-mast (tower) lighting is also in use at a few overpasses. Area lighting underneath overpassing bridges is considered an important part of the roadway lighting design.

Design Considerations

Lighting design for a SPUI should be sensitive to the special aspects and requirements of this interchange and to local practice. For example, some states do not presently light urban freeways or interchanges using roadway lighting by policy to save energy and money. Special lighting of a SPUI would be inconsistent with this policy. However, anyone who has seen a well-lighted SPUI would understand the apparent benefits provided and the trade-offs made.

Existing roadway lighting also affects lighting options. The field survey reveals that a new SPUI should be lighted if the mainline (freeway) is lighted for both overpasses and underpasses. If the SPUI were not lighted, the existing overpass lighting would cast a dark shadow under the bridge at the most critical operational point. Existing underpass lighting cannot efficiently light the pavement surface of an elevated bridge deck. The light sources (luminaires) in both cases would produce glare and an undesirable backlighting of traffic signs. Underpass lighting is particularly prone to excessive glare sources unless high-mast (tower) lighting is used. Thus, the SPUI should be lighted if either the mainline or cross street is lighted.

Most SPUIs surveyed were well lighted both on the mainline and cross-road. Good lighting was also typically provided on all ramps and junction points. Average maintained levels of horizontal illumination in the range of 1 to 2 foot-candles (ft-c) were measured at 10 sites using a digital photometer. Variability in light intensity (illumination) in both horizontal and vertical planes was the principal problem observed at the sites. Lighting under the bridge using only wall-pack units, as shown in Figure 85, create hot-spots (5 to 6 ft-c) and significant glare to off-ramp traffic and, unless highly diffused and combined with ceiling lighting attached to the bridge, is not judged desirable for SPUI applications.

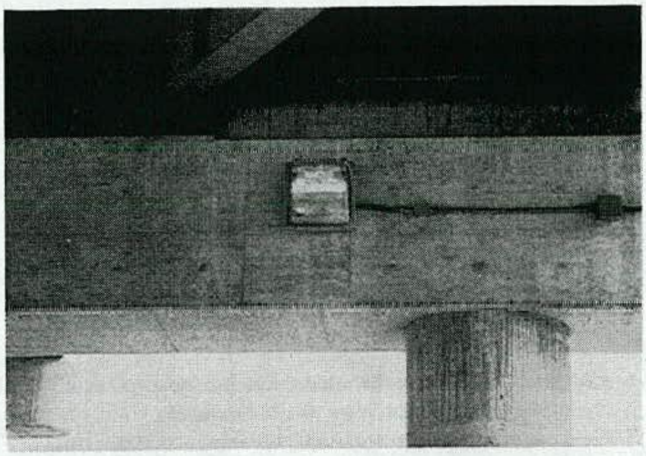


Figure 85. Example of wall-pack units that may produce glare.

Uniform distribution of light within the intersection area is a desirable objective for a SPUI lighting system. This is much more difficult to achieve with mainline overpass designs. Here, the overpass bridge casts a shadow from the mainline and cuts off light emitted from nearby cross-road light sources directed toward the central area of the SPUI underneath the bridge. It should be noted that the central area of the SPUI is the most critical area of the interchange, unlike traditional tight diamond designs, where this area is mainly a connection between intersections.

Design Guidelines

The following lighting design guidelines are provided to aid lighting engineers in providing an optimal driving environment for SPUIs. Technical knowledge of roadway lighting systems and standard design practice is assumed; however, it is also recognized that many engineers may not be familiar with all aspects and issues of SPUI design.

In all cases, the most important lighting design principles are uniformity of light and minimization of glare. The level of horizontal-plane illumination should be consistent with the class of the interchange facility and surrounding environment (see Table 8 of FHWA's "Roadway Lighting Handbook" for details (*29*)). Field measurements of illumination indicate that engineers are providing average maintained levels of horizontal illumination in the range of 1 to 2 ft-c external and 0.6 to 1.2 ft-c underneath the bridges of overpasses. These levels of illumination appeared to be adequate, given that reasonable uniformity of light distribution is provided. However, it appears that some engineers have tried to use the wall-pack units to make up the difference in illumination and to add more vertical-plane illumination under the bridge, perhaps for pedestrians. These units, as typically used, may produce severe glare to off-ramp left turns at night.

For overpasses, the crossroad lighting system should be designed such that the central area underneath the bridge is well lighted. A dark shadow band paralleling the center line of the bridge should not be permitted to occur on the pavement underneath the bridge. A good design can be achieved with some

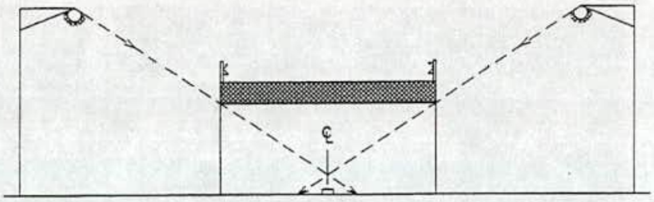


Figure 86. Design objective for roadway lighting under bridge.

special attention given to the selection and location of the lighting fixtures located on both sides of the bridge along the cross road.

Figure 86 illustrates the design problems and objectives. The luminaires should be positioned both horizontally and vertically such that light rays from the closest light will travel at least 10 ft beyond the center of the bridge (measurement is made along the center line of the cross road). This is achieved by carefully analyzing the effective cutoff angle to the light ray created by the lower flange of the exterior bridge girders. A 20-ft overlapping area, as a minimum, is desired for the closest light fixtures. A box layout of four 250-W high-pressure sodium luminaires centered along the cross road on each side of the bridge seems to provide the desired illumination. The closest luminaires would be located within the ramp refuge islands, if the islands are of adequate size. Again, location of the luminaires and their mounting height affect the resulting light distribution cast underneath the bridge.

Side-fire luminaires strategically positioned behind the ramps along the right-of-way line also enhance this design. These light fixtures should illuminate an area at least 30 ft along the center line beyond the center of the SPUI. In this case, the bridge flange and the vertical wall of the exterior bridge abutment form cutoff points of the light rays. The secondary luminaires can be designed to light the signs on the center island and the vertical walls of the bridge abutments. This careful attention to lighting detail was noted at the SPUI shown in Figure 66 where illumination is good under the bridge, yet no lighting whatsoever is used under the bridge, no pavement marking lights are used, and the nighttime driving environment is excellent. The light beige color of the SPUI also enhanced its nighttime visibility.

Use of "wall pack" lighting units along the vertical walls of SPUI bridge abutments, as shown in Figure 85, should be discouraged. To be sure, these units have been routinely and effectively used by engineers to light overhead signs and roadways within overpasses and tunnels. The survey also indicated that wall-pack units are also frequently used at SPUIs with mixed results. A SPUI has one operational difference that is significant in this case. The four inverted left turns have to directly face the bridge abutments and, consequently, drivers stopped on the off-ramps waiting for the traffic signal would have to stare directly into the glare of the wall packs for a considerable period of time. If wall packs are used, they should be highly diffused for pedestrian benefits and used only with area lighting under the bridge and external roadway lighting systems. They should not be used alone as the sole source of illumination.

Finally, the lighting engineer should work with the bridge engineers to prepare a quality ceiling lighting system under the bridge. Illumination levels under the bridge should be according to desirable agency standards, or Ref. 29, but on the order of 1 ft-c horizontally. Provide as much overlap between lighting elements placed between the bridge beams without introducing direct glare to motorists. Consider the cross section of the bridge beams and panels in locating the lamps. Align the lamps directly over the through lanes in a consistent pattern along the lanes. Field observations suggest that the walls and under side of the bridge should be painted a uniform pastel color to enhance the driving environment. Ensure that all exterior electrical conduit on the bridge is painted the same pastel color. Figure 87 provides examples that follow these guidelines.

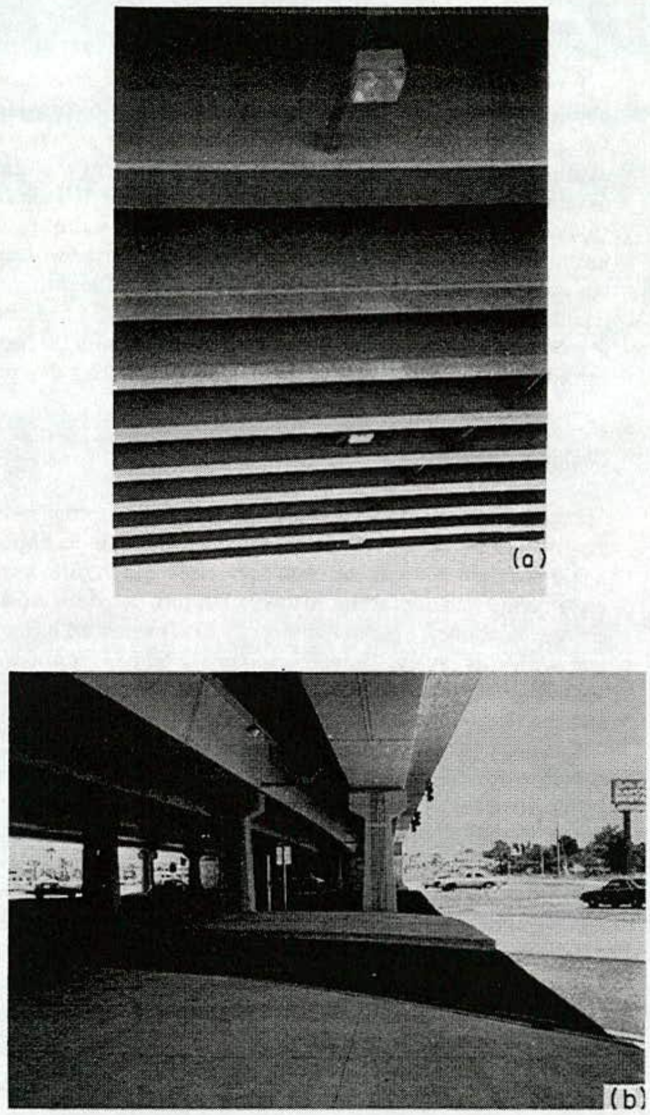


Figure 87. Lighting design underneath the overpassing bridge: (a) lighting units attached to mainline girders; (b) lighting units attached to bridge deck.

CHAPTER SEVEN

COST EFFECTIVENESS ANALYSIS GUIDELINES

This chapter describes a recommended cost effectiveness analysis process for single point urban interchange (SPUI) applications. It provides general guidelines for assisting the planner in selecting the most desirable interchange in the project development stage of the planning process. This cost effectiveness analysis has a twofold concern. First, the process focuses on estimation of: (1) project costs, (2) right-of-way acquisition costs, and (3) operation and maintenance costs. Second, user costs associated with traffic operations are estimated. These user costs provide a basis for assessing the operational benefits of a particular design. Benefits and costs are used together to determine whether or not the proposed design is a better selection economically than a competing alternate design.

The following sections address key aspects of cost estimation and operational benefits analysis in the context of cost effectiveness. Critical design factors that impact cost estimation of SPUIs are discussed. Primary and secondary cost components are identified and their relationship to total project cost described. Key user cost elements are presented and a typical approach to quantify them is discussed.

The cost effectiveness analysis process is illustrated using the case study approach. An at-grade intersection (AGI) is compared to a SPUI with the major road over the cross road as the competing design. The potential impact of alternate designs compared to the SPUI is considered. The intent is to present a qualitative assessment of criteria that are considered critical discriminators when evaluating a SPUI compared to other feasible competing designs, possibly even including other SPUI design options.

The cost effectiveness analysis process should be implemented during the early stages of project development when evaluating different design solutions. As a consequence, the cost estimation process and the analysis of user costs presented are applicable to broad-based project development and not to detail design. The process and relevant techniques presented are generally limited to the conceptual planning phase of project execution where basic design decisions are made.

KEY COST ISSUES

Highway planners are faced with a variety of issues when evaluating the relative merits of alternate designs to handling traffic flow through an intersection. The relative merits of an alternative must be quantified in terms of total cost of the alternative versus the benefit it provides to the user. The comparison of several alternatives should consider the incremental cost and associated incremental added value to the user for each alternative. While this decision process is complex, it is essential to selecting the best solution for either new construction or reconstruction of an existing interchange.

The research indicates that the SPUI is a likely design solution in four distinct situations. It is a viable solution when an at-grade urban intersection is at or near its practical capacity and fully developed within the physical constraints of its existing location. In this case, a two-level interchange is the only real alternative to achieve a sufficient increase in capacity over a reasonable design life to warrant the sizable cost of a new interchange. SPUIs also are effectively used in new freeway construction where the freeway intersects with a major arterial or cross street. When an existing interchange on a freeway requires significant modification or rehabilitation, a SPUI is also a possible solution. Finally, when an arterial is upgraded to expressway standards, the SPUI is an interchange which may serve the need.

In all of the foregoing situations, there are critical factors that influence the selection of the type of interchange such as site topography and existing conditions, traffic volumes, and interchange layout. The range of candidate choices includes the SPUI, the tight urban diamond interchange (TUDI), and possibly the partial cloverleaf (parclo).

For the SPUIs studied in this research, the diamond interchange would probably be the primary alternate design concept. Where congestion relief is the primary driving factor for considering an urban intersection upgrade, then perhaps two alternate design concepts should be evaluated. The alternatives would be a SPUI and a TUDI.

Both of these two-level interchanges require a substantial initial investment. They also can provide a 50 to 100 percent increase in capacity over a 20-year design life in comparison to the existing at-grade condition. This large increase in capacity is necessary to generate sufficient user benefits to justify the sizeable cost of either interchange. In most cases, the benefits of the grade separation solution will be much higher than the associated initial construction costs when compared to a highly congested AGI, thereby easily demonstrating the economic viability of either the SPUI or TUDI. This relationship is schematically illustrated in Figure 88.

The graph in Figure 88 shows an increasing incremental benefit-to-cost ratio on the vertical axis and increasing construction cost on the horizontal axis. A fully built out and congested AGI (existing condition), and two competing design solutions, a TUDI and SPUI, are plotted in their probable relative positions on the graph. According to Figure 88, the investment in construction cost is large for both the TUDI and SPUI compared to the AGI. However, the incremental increase in benefits due to significant increases in capacity also will be large and would more than likely easily justify the cost of construction of either design in the congested AGI conditions set forth above. Thus, both the TUDI and SPUI will have an incremental benefit and cost ratio that is greater than one as shown on the graph. The actual B/C ratio is likely to be on the order of eight-to-one for this case.

Figure 88 makes one other significant point. SPUIs are typically more costly to construct than TUDIs (*9, 30, 31*). Thus, selecting the SPUI over the TUDI would require that the addi-

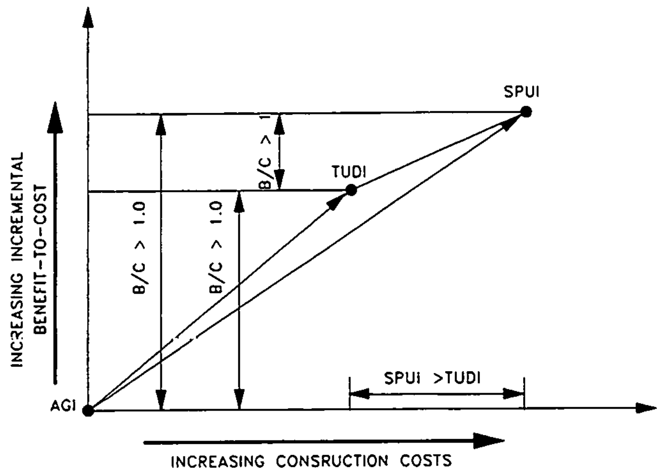


Figure 88. Relationship between incremental benefit/cost to construction costs for various interchange design types.

tional construction expenditure be justified through either an increase in operational benefits via reduced user costs, i.e., the SPUI handles a greater traffic capacity, or a reduction in right-of-way cost, or a combination of both. The net result must be an incremental B/C ratio greater than or equal to one to select the SPUI over the TUDI. This is shown on the graph as a double arrowed line connecting the horizontal line through the TUDI and the horizontal line through the SPUI. In the final analysis, the selection of the SPUI may be considered as a trade-off between the increased cost of the bridge structure and a reduction in right-of-way and user costs. The reduction in user costs will be reflected in the potentially higher capacity of the SPUI compared to the TUDI.

The design team should not automatically select one interchange solution without considering the other option whether it be a SPUI or TUDI. The choice should be based on the best B/C ratio for the design that meets the projected traffic capacity requirements and fits the site conditions in which the interchange will be built. Both should be considered competitive alternatives.

The intent of this chapter on cost effectiveness is to provide guidelines that would assist the design team in evaluating the SPUI as one alternative design solution. It is assumed that most readers are familiar with and have considerable knowledge of the more common tight urban diamond interchange. Therefore, the majority of the sections are limited to discussions solely related to cost estimation and the analysis of user costs for SPUIs. The illustrative case study, however, will be based on upgrading an at-grade intersection with a SPUI as the design alternative. The techniques and guidelines presented are generally applicable to evaluating other designs. All competing designs should be considered by the design team in the context of Figure 88.

Cost-effectiveness analysis is one structured methodology for selecting the appropriate interchange. This approach most often compares alternatives. There is usually a base case, i.e., as an existing condition, and one or more viable alternatives that could replace the existing facility. The relative worth of each proposed alternative can be determined by using the benefit/cost (B/C) method. This method provides the basis for selecting the best alternative, given the estimated user benefits and total cost associated with each alternative as their designs dictate.

An evaluation of this nature normally occurs in the early or conceptual planning stages of a project. At this stage planners are developing alternate design concepts. These concepts are converted into a sufficient level of design detail to estimate both user costs and initial project costs. Thus, the context of the discussions herein is from this conceptual planning perspective.

The primary purpose of this chapter is to describe a methodology for evaluating the cost effectiveness of Single Point Urban Interchanges. To achieve this purpose several objectives are delineated as follows: (1) to identify critical design and location parameters that impact the cost estimation and benefits analysis of SPUIs; (2) to understand how these key parameters influence cost estimation for both evaluating total project costs and user costs during conceptual planning; (3) to illustrate the cost effectiveness methodology using a case study approach; and (4) to discuss qualitative selection criteria that might discriminate between feasible competing designs and the SPUI. The sections to follow will address each of these objectives.

PROJECT DEVELOPMENT AND COST EFFECTIVENESS

Two principal components of the cost effectiveness process are the estimation of initial project costs and user costs. At the conceptual planning level of project development, the estimation process is correlated to the design information available, accuracy desired, and the constraints imposed by the site location.

Two types of estimates are commonly used during early project development. They are the preliminary or "order of magnitude" estimate and the comparative estimate (*32*). The order of magnitude estimate attempts to bracket the probable cost within a rather large range. This estimate is based on limited data and only on a general definition of project scope. Reference projects that have similar scope and site conditions are used to approximate the cost of the new project. This type of estimate produces the lowest degree of accuracy.

It is often difficult to meaningfully compare alternatives using the order of magnitude cost estimate. Moreover, because of the unique nature of the SPUI design, it may be difficult to locate projects with similar characteristics. For those agencies whose history with SPUIs is limited such an estimate process may not be feasible or desirable for comparing alternatives.

The comparative estimate is best suited for cost-effectiveness analysis. This type of estimate requires both quantities and unit costs for key parameters that describe the project. These key parameters are linked to specific design factors and site conditions. In comparative estimating, an increased level of design information is required. The accuracy of the estimate improves and the opportunity to compare alternate designs is enhanced. It is important, therefore, to understand the relationship between critical design elements and key parameters needed for cost estimation.

In developing the SPUI conceptual design, there are several key factors that predominately influence the characteristics of the design and, hence, costs. They are: (1) the right-of-way available, (2) the topography and existing conditions at the interchange site location, (3) the geometrics of the SPUI, (4) the traffic volume, and (5) the structural features of the bridge. These factors were discussed in detail in Chapters Two through Six. The interrelationships among the design factors, key estimating parameters, and cost are described in the next section.

MAJOR INITIAL COST COMPONENTS

The design, construction, and operation of any project can generally be characterized by its major facility components. One characterization that is particularly useful for cost estimation during conceptual planning is the relative weight of each component in terms of its contribution to total cost. The components that have the largest impact on cost are given the closest scrutiny when developing a specific conceptual design and estimating costs.

At the conceptual planning level, most major facility components are estimated based on a single key facility parameter, such as surface area of bridge deck. The nature of each key parameter is specified so that sufficient design information is developed to quantify the parameter. The estimated cost of a facility component derived from a parameter must encompass all aspects of constructing the component.

The following section describes major cost components that an agency should evaluate in the cost-effectiveness analysis process. Three major cost components are covered and include right-of-way, construction, and engineering and design and contract administration. Specifics of each component are described in detail with some indication of the likely range of expected costs as relevant to SPUIs. Potential variations in significant cost components are discussed including probable causes of these variations. Unique construction problems associated with SPUIs that influence cost also are highlighted.

In this section reference will be made to different elements of cost. When discussing total project cost this total covers all project costs for right-of-way, engineering and design and contract administration, and construction. Construction costs cover those items of work typically incorporated into a contractor's bid.

Right-of-Way

The amount of existing right-of-way available may restrict the design. It may determine the boundary limits of the interchange in both directions. If the proposed interchange cannot be built within existing right-of-way, the area of new right-of-way taken is a critical estimate parameter. The magnitude of this area is determined by the proposed geometric design of the interchange.

The cost of acquiring new right-of-way is a significant factor in the planning of any interchange. The cost of a partial taking of open urban land for an interchange can vary from as little as \$1.00 to \$2.00 per square foot to as much as \$50.00 per square foot in a highly traveled and populated urban area. If buildings or other properties are in the potential right-of-way area, the cost per square foot may be much higher than \$50.00 per square foot and can exceed \$1,000 per square foot in special cases where major buildings are taken. Clearly, cost of right-of-way can be the controlling factor in the general economic feasibility of the proposed project.

Cost per square foot is only one element, however. A second critical element affecting total right-of-way cost is the additional right-of-way parcels required beyond the existing right-of-way. The parcels are dependent on specific site location and are influenced by the geometrics of the SPUI. Specifically, the cross road right-of-way of the arterial includes the width of traveled way plus a 10-ft easement on each side of the cross road. The total cross road right-of-way is then a function of the number of through lanes and left turn lanes as specified in the design.

The major road right-of-way is a function of the separation distance between the SPUIs on ramps and off ramps. The larger the separation the more right-of-way area required and, hence, a possible increase in cost for acquiring the land. Finally, the use of retaining walls will impact the area of right-of-way needed. Retaining walls will reduce right-of-way area at the expense of added cost for retaining walls versus the use of sloped embankments.

Area of right-of-way is further influenced by factors that may constrain design layout such as: requirements for traffic maintenance during construction, access to business in and around the interchange, topography, and whether or not a complete parcel must be purchased. Moreover, the SPUI may require new right-of-way in all four quadrants. Therefore, all of the preceding factors should be considered in each quadrant of the interchange.

Because most of the SPUIs described in this report are constructed in an urban setting, it is likely that the cost of new right-of-way acquisition will be a significant determinant of overall cost effectiveness. This cost component has high variability and can be as much as 40 percent of the total cost of some urban projects. In those cases where the SPUI can fit into the existing right-of-way, its impact on project economics is not as significant.

Construction

Construction costs typically include those costs for labor, material, equipment, and support, including overhead and profit, required to build permanent facilities. In this study, cost data were gathered and analyzed to identify key construction cost components of SPUIs. Estimates prepared by consultants and bid sheets with costs from actual construction projects were gathered and reviewed. Several site visits were made. This analysis resulted in the identification of approximately ten major components of SPUIs most frequently evaluated when preparing a conceptual construction cost estimate. The specific components identified are the bridge deck and its support structures; pavement for the major road, cross road and ramps; earthwork; retaining structures; and other items such as signalization, lighting, drainage, traffic control, mobilization, signing and so on.

The construction cost data gathered were dissected by major components. Percentages of total construction cost by each component were determined. Because the percentages varied rather drastically, a percent range was developed by major component. These results are delineated in Figure 89. As shown in Figure 89, the primary construction cost components are the bridge, pavement, earthwork, and retaining structures (if used). These four critical components account for 65 to 75 percent of the total construction cost of the SPUI.

Another significant point gleaned from Figure 89 is the apparent wide variation in the cost percentages for each component. This variation is a function of the specific design selected, materials of construction, and site location characteristics. For in-

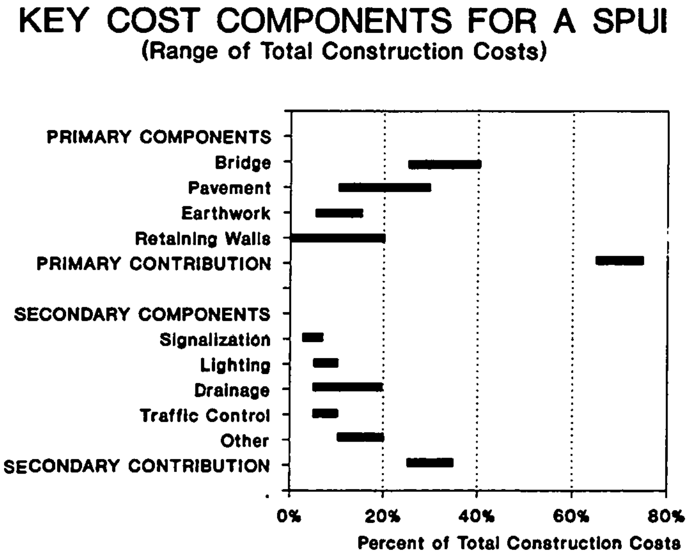


Figure 89. Key construction cost components for SPUIs.

stance, if a sloped embankment is used in lieu of retaining walls, the percent for retaining walls is naturally zero while the percent weight for the other components increases. Additionally, the percent construction cost associated with the bridge could vary, depending on bridge length and whether concrete or steel is used.

On the basis of the data presented in Figure 89, the conceptual estimating process should focus on the four critical components that likely will determine 65 to 75 percent of the total construction cost. Since the design features that influence these four key cost components also impact right-of-way, it follows that right-of-way area and cost should be a fifth key cost when evaluating total project cost. It also should be emphasized that the percentages given in Figure 89 could change markedly, depending on actual site conditions such as the impact of a major utility relocation and other existing conditions.

Bridge

Bridge cost is most often the single largest construction cost component. It can vary from between 25 to 40 percent of the total construction cost. The structural features of the bridge significantly influence other design features of the SPUI. As a consequence, the bridge is perhaps the most important SPUI cost component, especially in relation to other two-level grade-separated interchanges. Bridge cost, when expressed as a percent of total construction cost, is likely to be higher for the SPUI than for the TUDI, being significantly more costly for overpasses.

The bridge cost component for conceptual estimating typically includes the bridge deck, superstructure, substructure, and all miscellaneous items attached to it, such as guard rails, wing walls, and concrete traffic barriers. The key estimating parameter is the square area of bridge deck. Unit costs for SPUI bridges can vary from $60.00 to $100.00 per square foot of surface area depending on bridge depth, bridge length, use of a single or multispan bridge, the type of bridge (I-beams or trapezoidal box girder), and whether concrete or steel construction is specified. An appropriate design, the selection of construction materials, and an accurate cost analysis are crucial to properly evaluate bridge costs for SPUIs.

Bridge configuration depends on whether the major road is over or under the cross street. The existing site conditions will then determine which configuration is selected. In new construction, site topography is the key determinant. Alternatively, if the SPUI replaces an existing interchange, the new design will generally follow existing grade profiles.

If the major road is over the cross road, the bridge can either be single span or multispan. The center span can be large and can vary between 180 to 280 ft for single span bridges. Total bridge length for multispan bridges can reach more than 500 ft. The long center span of the SPUI correspondingly increases bridge thickness. This increase creates potential structural fabrication and construction problems that might have a significant impact on construction costs. Careful attention to construction sequencing, material delivery, and traffic control must be evaluated and the estimate adjusted to local conditions as appropriate. Compared to an overpass TUDI the cost of an overpass SPUI bridge is likely to be at least $10.00 per square foot greater and is a major cost difference between the two interchanges.

When the major road is under the cross road, the bridge center span decreases markedly because a center bent can be used. Total bridge length of between 90 and 194 ft was found in this research for underpass designs having one or two spans. The largest single open span was 194 ft. Because of the inverted left turn lane configuration of the design, bridge construction for this type of SPUI increases in complexity. As an example, flared ends at the left turn movements off the exit and entrance ramps have a curvature that impacts the main supporting girders. These girders are more difficult to fabricate or construct because of their curvature in comparison to an underpass bridge for the tight diamond (see Figure 61 for an example of a SPUI underpass design under construction). This requires special consideration when estimating bridge cost for the underpass SPUI and should be reflected in the unit cost. A larger area simple platform bridge design may be cheaper than a smaller but more complex shape.

Four types of bridge designs are common to SPUI construction: (1) steel plate girder, (2) steel trapezoidal box girder, (3) concrete cast-in-place box girder, and (4) steel or precast concrete I-Beam.

The first three are most often used on overpass designs, while the last type of bridge is more commonly associated with underpass designs. Generally, the concrete box girder is the least costly of the first three types and on the order of 10 to 20 percent less than the steel bridge. This percentage differential will vary with economic conditions, site location, required maintenance of traffic, suppliers of material and other related factors. Finally, the steel trapezoidal box girder is often used to reduce bridge depth. However, this type of steel bridge is more costly to fabricate, transport to the site, and install in the field because of its shape and manner of connection. Figure 90 shows a steel trapezoidal box girder under construction for a multispan SPUI bridge with frontage roads.

Because of the criticality of the bridge component, the design team may want to develop rough drawings of the girder system. This would allow for more detailed pricing of individual bridge components. It also may provide a better comparative base if concrete and steel are both viable construction materials. This approach requires more engineering time and cost to create the requisite design information. If little historical data are available on SPUI-type bridges, developing additional bridge design and construction data may be a wise investment.

Research suggests that the type of bridge structure selected is often a function of material availability, demonstrated local construction expertise, and preference of the agency building the interchange. As an example, most SPUIs constructed in the Phoenix area are concrete box girders using a soffit fill method of construction. This technique requires an earthfill under the proposed bridge deck. The fill is compacted up to just below the bottom of the bridge deck. Typically, a 3-in. to 4-in. lean concrete layer is poured on top of the fill. This provides a flat working surface for constructing the concrete box girder. After the bridge is completed the fill and lean concrete layers are removed. This method is believed economical locally because of an ample supply of good fill material plus the expertise of local contractors well versed in using soffit fill construction. The availability of excellent aggregate for concrete also is a factor and is dependent on geographical location.

Construction access and traffic control are two other factors influencing bridge type selection, especially in a reconstruction scenario where traffic flow must be maintained through the intersection. If the area around the interchange has sufficient available right-of-way, temporary "frontage" roads can be built in such a way as to divert traffic around the area of bridge construction. This may allow full access to the bridge during construction as opposed to building the bridge structure in stages in order to maintain traffic flow through the major and cross roads. Temporary roads for traffic control are costly. They are required if concrete box girder construction is used. If steel is used proper staging of bridge construction can minimize the impact on traffic, particularly when an existing bridge is present. Traffic can be controlled to allow bridge construction to continue through its entire erection sequence with the exception of a minimal interruption to set the steel girders which may require road closure over a weekend.

Careful analysis of the traffic control problem is necessary because of its potential impact on cost. Traffic control also influences the construction schedule, depending on the particular circumstances of the site and type of bridge selected. A traffic control plan should be developed early in the conceptual planning stage if greater precision is desired in cost estimation.

Pavement

Pavement costs for the major road, the cross road, and the entrance and exit ramps represent a second significant cost component. This cost component can vary between 10 and 25 percent of the total construction cost. At the conceptual estimate level, pavement cost includes the preparation of subbase, the paving material itself, i.e., asphalt or concrete, and medians.

Pavement costs are estimated as a function of the total square feet of road surface area and the unit cost per square foot of surface area. The square foot of pavement area is determined by the width of the road, less curbs, and the length. The unit cost would cover subbase and all paving construction. Unit costs are likely to vary depending on the thickness of the pavement, the type of paving, i.e., jointed concrete or continuously reinforced concrete paving, availability of materials and volume of current construction. A typical range of unit costs for pavement construction can vary from $1.75 to $6.00 per square foot.

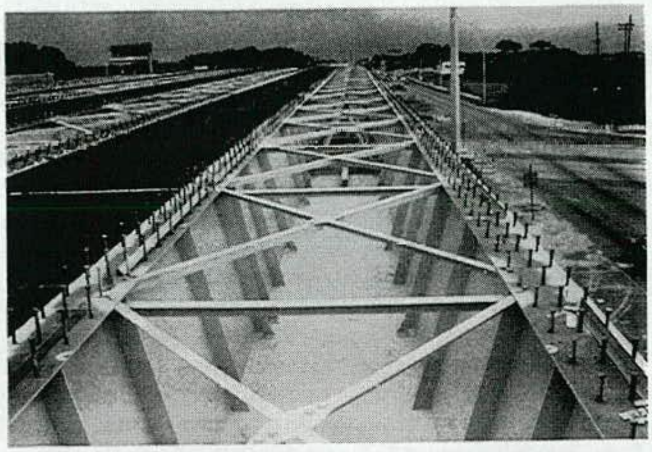


Figure 90. Steel trapezoidal box girder for SPUI bridge under construction.

Pavement costs may be somewhat less for SPUI compared to other two-level grade-separated interchanges because of a fewer number of lanes on the cross street. This decrease would be offset somewhat by an increase in ramp pavement due to the curvature required of SPUI ramps.

Earthwork and Retaining Structures

The remaining two primary cost components are earthwork and retaining structures associated with the major road. These two components can comprise between 10 and 20 percent of the total cost of construction. Earthfill is used to prepare the incline ramp for the major roadway from where the major road deviates from existing grade to the beginning of the bridge structure on either side of the bridge. The cost of earthfill will be greater for the single span SPUI compared to tight diamonds due to an increased vertical alignment, as noted in Figure 65. If the major road passes under the cross road, earthwork consists of excavating existing soil and removing it from the site. Variations in costs also are due to the type and availability of materials, soil characteristics, haul distances, location of fill and volume of soil required. Unit costs range from $2.00 to $14.00 per cu yd.

The major road is often bordered by a concrete retaining structure unless sufficient right-of-way is available to use sloped embankments. Retaining walls can be cast-in-place or precast reinforced earth panels. Cast-in-place walls are more costly than precast walls but provide more flexibility for future utility work.

Earthwork costs are comprised of purchased fill or soil removed, the hauling of the materials to and from the construction site, and the compaction of the material in place. Earthwork costs are based on cubic yards of soil moved.

The quantity of fill will increase substantially if sloped embankments are used in lieu of retaining walls. In this case, the cost of earthwork increases. Consequentially, more right-of-way area will be required which can increase project cost. However, these cost increases are offset by eliminating the retaining wall. The net impact on cost is influenced by the unit cost associated with right-of-way acquisition, assuming sufficient right-of-way is available to allow this alternative. In many urban settings, aesthetics may be the dominant factor in selecting a retaining wall even when the retaining wall is more costly. Conversely, large elevated designs may be rejected because of their perceived negative impacts on aesthetics.

If retaining walls are the only option, because of cost prohibited right-of-way acquisition, the combined cost of earthwork and retaining walls is likely to be closer to 30 percent of the total construction cost. In comparison to tight diamonds, the use of retaining walls for overpass SPUIs would increase construction cost. This is due to the SPUIs increased vertical alignment, which adds height to the retaining wall, and correspondingly increases the roadway fill material required in relation to the tight diamond.

At the conceptual planning phase of project development, the estimated cost of retaining walls is based on the outside surface area of the wall. This area is a function of wall height and the length of the wall from its starting point to the bridge abutment structure. The maximum height is driven by vertical alignment, as determined by minimum vertical clearance, bridge depth and design speed, previously shown in Figure 65. Retaining wall unit costs range from $15.00 to $45.00 per square foot of wall area, depending on site location, type of wall (precast or cast-in-place), height of wall, and wall thickness.

Other Construction Items

Many other project items will influence the cost of SPUI. These additional items represent the remaining 25 to 35 percent of the total initial construction cost. The major items include: signalization; lighting; curbs and gutters; drainage; traffic control; mobilization; minor items such as signing, stripping, landscaping; and demolition. Individually, these items represent a small percent of the total construction cost (see Figure 89); however, together they can have a major cost impact.

Signalization includes the signal heads, signal supports electrical distribution, controllers, and pavement markers. Primary power and a transformer are required at each intersection to operate the various signals. Signalization is often estimated based on the number of heads with an allowance included for primary power and the transformer. The latter allowance depends on whether there is existing power available such as in the case of reconstruction. Signalization could be as much as 1 to 4 percent of the total construction cost.

Lighting includes light poles, all luminaires, electrical distribution, lighting panels, and connections. This cost category is frequently estimated based on the number of light poles. Lighting typically runs between 2 and 5 percent of total construction costs. Lighting for SPUIs would probably be more costly when compared to other similar interchanges. This is due in part to the lighting required under the bridge. The longer and deeper bridge girders make it more difficult to light the conflict area under the bridge.

Curbs and gutters are typically referenced in terms of height of the curb and width of the gutter such as 6 in. by 24 in. The cost of curb and gutters is estimated on a lineal foot basis to include forming, reinforcing steel, and concrete. This cost category is 1 to 2 percent of the total construction cost.

Drainage covers those elements required to remove storm water from the bridge and roads. This cost category can vary significantly from as low as 3 to 5 percent to as much as 20 percent of the construction cost. It is dependent on several critical design and location criteria. If, for example, the project is considered new construction, the design could require installation of a major storm sewer system including main headers and feeders from various collection points. This could be further compounded if either the major road or the cross road is depressed below grade. In this latter case, pumping may be necessary to lift storm water into a main header system. There also could be existing storm sewer and other utility lines that may have to be relocated. A design may prove totally infeasible if it requires major utility relocation.

If the project is considered reconstruction, a new storm sewer system may or may not be required, depending on whether the existing system can carry the additional needed capacity due to the larger surface area for water runoff. The problem of existing storm sewer lines would have to be examined closely to determine if they can accept added demand. Similar problems would occur if either road was depressed. This would necessitate relocation or installing new sewer lines and perhaps the addition of a pump station.

Drainage is a site-specific item and must be evaluated closely to determine the appropriate historical percentage used to estimate its cost. Because the percentage can vary substantially, other estimating techniques might be warranted depending on site circumstances. As an example, a preliminary conceptual design could be developed to better scope the problem. This design could provide rough quantities for estimating purposes.

Generally, drainage cost would be similar for the SPUI in comparison to competing designs. However, if the cross road must be depressed because of the desire to maintain an existing profile grade line and increased bridge thickness, drainage costs for the SPUI would increase and probably would be higher than for a tight diamond design for the same situation.

Traffic control during construction is another category that can vary depending on the design and specific site location. This cost category includes the construction of temporary roads, signs, signals, and so on, if a SPUI is changing an existing intersection to a grade-separated interchange. The extent of traffic control is a function of the type of bridge, method of construction, and right-of-way available. This cost category is typically estimated as a percent of total construction cost, normally 6 to 8 percent. However, if traffic control is extremely complicated the percentage can be much higher. Depending on the situation, a traffic control plan may be developed to better estimate the cost. This may be advisable for the SPUI case due to its unique feature of the long center span.

Mobilization covers those costs incurred by the contractor to set up the site to commence construction. These costs are typically estimated as a percent of the total construction cost (between 3 and 5 percent).

Minor items such as striping, signing, sidewalks, and landscaping are generally included as a percent of total construction costs (1 to 3 percent). Two large overhead guide signs on the cross road may be needed only for the SPUI because of its special advance left turn operations. Discretionary use of pavement marking lights would add approximately $100,000 to the project cost. While minor in terms of their impact on total construction costs, signing and striping can have a significant impact on traffic operations of the SPUI. Careful attention to the design of these two items is important in the conceptual planning phase to properly evaluate user costs.

Demolition is required if the project involves reconstruction. This may include the removal of existing structures or breaking pavement and disposing of it. The cost of demolition will vary with specific site conditions and the distance required to haul the materials to a disposal site. Pavement demolition can be estimated on a square area basis, while bridge demolition can be estimated using a per ton or square area approach. Careful analysis of the existing conditions must be made to estimate a reasonable cost for this category. As with drainage, any major differential in the required demolition (or salvage value) of existing infrastructure that one design alternative accrues over another can be very important to the economic analysis and should be provided.

Contingency

Contingency is a cost category that covers the uncertainties associated with an estimate. The percent contingency added to an estimate during any phase of a project is related to the type and nature of the information available and the risks associated with the project. This includes the status of the design, the location of the project, and the historical cost data base used to convert the design into a reliable cost estimate.

Typically, conceptual planning estimates for highway construction can be as much as 15 to 25 percent higher than the contractor's low bid (*32*). This variation includes a high percent contingency. The percent contingency used on most conceptual estimates of the nature discussed in this report is between 15 and 20 percent depending on the information available about the site and supporting design details developed. It might be argued that the percent contingency added to the SPUI estimate should be higher when compared to other similar interchanges. This would result from the uncertainties associated with unique bridge construction and traffic control especially where an agency has little experience with the large center spans common to overpass SPUIs. The agency may not have historical data relevant to SPUIs, thus, adding more estimate uncertainty.

Engineering and Design and Contract Administration

The total initial cost of any project should include the cost to perform engineering and detailed design through the development of construction plans and specifications. Once the project is let out for bid and awarded to a general contractor, an engineering group must supervise and inspect construction. This also includes drawing interpretation, processing contract modifications, verifying quantities placed and so on. At the conceptual planning phase of a project, these two cost components are estimated as a percent of total construction cost. This percentage is typically between 8 and 10 percent.

The nature of the SPUI design process combined with bridge complexity would likely increase the engineering and design costs in relation to the tight diamond interchange. Because these costs are a percentage of the total construction cost and SPUI construction costs are customarily greater than the tight diamond, more costs would probably be estimated for engineering and detailed design of SPUIs.

ANNUAL MAINTENANCE AND OPERATING COST COMPONENTS

An integral part of cost-effectiveness analysis is the estimation of on-going agency costs once the SPUI is constructed and operational. Two categories of cost are considered, operating and maintenance. Operating and maintenance costs are recurring or annual costs. They continue over the estimated useful life of the project.

Operating costs cover electricity to operate the signals and luminaires. Maintenance costs include general maintenance of traffic signals and luminaires, signs, striping, curbs and landscaping, resealing of joints for concrete pavements, repair of guard rails, and painting of the bridge structure as required.

Operating costs can be estimated by determining the kilowatt hours of electricity used by the signals and lights. This can be approximated based on a count of the number of signal heads and luminaires used in the design. Based on the kilowatt-hour consumption by each item, the total anticipated use per year and the cost per kilowatt-hour, an estimate of operating cost can be developed. This estimate is an annual cost. It is likely that the SPUI has higher operating costs compared to other interchanges because of the additional lighting requirements and marking wear.

Maintenance cost can be estimated based on past history for the type of bridge and road pavement materials specified. Assumptions must be made on frequency of repair and cost at the time of repair. Past history is the best possible source for these cost data. Historical data may not be available on SPUIs since they are a relatively new type of interchange. It may be necessary to use similar types of interchanges for the purpose of estimating maintenance costs. Maintenance costs are estimated on an annual basis.

Often operating and maintenance costs for conceptual estimates are derived as a percentage of initial construction cost. This percent may vary between 5 and 10 percent. This approach provides the total cost of operating and maintaining the interchange on a present worth basis. The percentage method is suitable provided there are no unusual features that may distort these costs.

MAJOR USER COST COMPONENTS

The major user cost components that are usually included in a benefit-cost analysis include operational time costs, vehicle operating costs, and accident costs. Each of these costs is typically calculated for an existing (or do-nothing) alternative and for one or more improvement alternatives. Benefits are calculated as savings in user costs as compared to the existing or do-nothing alternative which typically performs poorer with time due to increasing volumes.

Time Costs

Total travel time is calculated as the sum of the time that it would take a vehicle to go through an intersection or interchange at a uniform speed (the mid-block or approach speed) plus the signal delay for vehicles that are required to stop. The signal delay includes the added time that vehicles take to stop and accelerate back to the mid-block speed plus the time actually stopped (also called idling time). Following the 1985 "Highway Capacity Manual" (*22*), it is assumed that the total signal delay equals 1.3 times the stopped or idling time. The recommended value of time per vehicle-hour in 1990 dollars is $12.69 for passenger cars (using $9.76 per person-hour and an assumed occupancy rate of 1.3 persons per auto) and is $23.02 for trucks, based on updated figures of time recommended values from NCHRP Project 7-12 (*33*); also see Ref. 34.

Vehicle Operating Costs

The operating cost equations used for highway segment and intersection calculations were estimated from Zaniewski (*35*), updated to 1990, and are given in the following.

Vehicle Operating Costs for Idling. Vehicle operating costs for idling are given for the two vehicle types, passenger cars and trucks, in cost per hour of idling time:

Idling Costs, Passenger Car = $0.94/vehicle-hour
Idling Costs, Truck = $0.97/vehicle-hour

Vehicle Operating Costs for Stops. The vehicle operating costs for passenger cars and trucks are estimated as the excess costs of stopping from and returning to a uniform speed as compared to traveling the same distance at the uniform speed. These stop-start cycling costs are estimated as functions of the uniform speed from which the stop is made. The equation for passenger cars is:

$$PCYC = 1.2206 + 0.14948 * S + 0.01028 * S^2$$

where $PCYC$ = excess passenger car cost, over the amount required to travel at the uniform speed, for stopping and returning to running speed S, in dollars per 1,000 stops ($/1000 stops); and S = passenger car approach speed in miles per hour.

The equation for trucks is:

$$TCYC = -9.8845 + 3.3657 * S + 0.09396 * S^2$$

where $TCYC$ = truck cycling cost from speed S to speed zero in dollars per 1,000 stops ($/1000 cycles); and S = truck approach speed in miles per hour.

Vehicle Operating Costs for Speed Changes Other Than Stops. Some analysts may want to calculate the cost for slowing down when traveling through an intersection, for those vehicles that do not stop. For calculating the vehicle operating cost associated with these speed changes, equations for excess cost for making 10-mph speed changes are included. The excess cost of making 10-mph speed changes for passenger cars is:

$$\log(PCYC1) = 0.9869 + 0.0324 * S - 0.0001 * S^2$$

where $PCYC1$ = excess passenger car cycling cost for a 10-mph speed change ($/1000 cycles), and S = uniform speed prior to making the speed change in miles per hour and is usually assumed to be the approach or mid-block speed.

The excess cost for making 10-mile per hour speed changes for trucks is:

$$\log(TCYC1) = 3.0784 + 0.0562 * S - 0.0004 * S^2$$

where $TCYC1$ = excess truck cycling cost for a 10-mph speed change ($/1000 cycles), and S = uniform speed prior to making the speed change in miles per hour and is usually assumed to be the approach or mid-block speed.

Vehicle Operating Costs at Uniform Speeds. Two vehicle operating cost equations are provided for estimating vehicle operating costs for travel at uniform speeds. The equation for passenger cars is:

$$\log(PVOC) = 5.6370 - 0.02750 * S + 0.00033 * S^2$$

where $PVOC$ = passenger car running costs per 1,000 vehicle miles, and S = uniform speed in miles per hour and is usually assumed to be the approach or mid-block speed.

The equation for estimating the vehicle operating cost for trucks operating at uniform speeds is:

$$\log(TVOC) = 6.7904 - 0.03464 * S + 0.00041 * S^2$$

where $TVOC$ = truck running costs per 1,000 vehicle miles, and S = uniform speed in miles per hour and is usually assumed to be the approach or mid-block speed.

Accident Rates and Costs

Although an attempt was made to evaluate accident rates at existing SPUI interchanges, several difficulties were encountered as applied to this study. Accident records are not available for an extended period for the limited number of SPUIs. Also, many of the existing SPUIs differ either in design or operation from the specific designs recommended in this report. Therefore, it was decided that no specific recommendations could be made on accident prediction for SPUIs as compared to conventional diamond interchanges. At this time, it is recommended that accident rates for all interchanges be estimated from the same standard rates, as discussed below.

Accident costs are calculated by multiplying the accident rate times the cost per accident. Accident rates for highway segments are taken from the Highway Performance Monitoring System Analytical Package (*36*). Accident rates for intersections and interchanges are based on a study of accidents in Texas from 1981 to 1986 (*37*). It was not possible to distinguish among interchange configurations because of the way the data are coded and the small number of accidents at interchanges. Costs per accident were taken from a study on accident costs by Rollins and McFarland (*38*). The accident rates and costs are given in Table 8.

The formula for using the rates for intersections and interchanges is given below. Note that the equation predicts the number of accidents per year even though the traffic is stated in daily terms, because the estimates were developed in this form.

$$AY = ACR * (ADT/1{,}000)/LN$$

where: AY = number of accidents per year, ACR = accident rate from Table 8 for a given type of intersection or interchange, ADT = total average daily traffic entering the intersection or interchange, for all directions of travel, including turning traffic, and LN = number of main lanes (not including separate turning lanes) from all directions for the intersection or interchange.

Table 8. Accident rates and costs for intersections and interchanges, by location (*37, 38*).

Type of Accident	PDO Accidents (ACR)	Injury Accidents (ACR)	Fatal Accidents (ACR)
Accident Rates per Intersection per [(ADT/1,000)/#lanes]			
Urban			
At Grade Stop	0.9393	0.5165	.0102303
At Grade Signal	0.4648	0.2145	.0020001
Interchange	0.0879	0.0518	.0014806
Rural			
At Grade Stop	0.8374	0.5484	.0306748
At Grade Signal	0.8655	0.3598	.0075463
Interchange	0.0694	0.0406	.0046282
Cost per Accident (updated to 1990)			
Urban			
At Grade Stop	$1,378	$14,434	$986,088
At Grade Signal	1,378	14,434	986,088
Interchange	1,312	13,646	952,235
Rural			
At Grade Stop	1,903	24,537	1,101,427
At Grade Signal	1,903	24,537	1,101,427
Interchange	2,034	22,832	1,187,136
Type of Highway	Freeway	Divided	Undivided
Accident Rate, Number per 100 Million Vehicle Miles			
Urban Highway	244	565	616
Rural Highway	93	261	248
Cost per Accident (updated to 1990)			
Urban Highway	$13,384	$12,597	$9,185
Rural Highway	29,688	37,134	36,740

COST-EFFECTIVENESS METHODOLOGY

This section illustrates the cost effectiveness methodology as applied during the conceptual planning stages of a project. Because benefit/cost analysis requires a base case to compare with an alternate design, an at grade intersection (AGI) is selected as the base case. A SPUI with the major road over the cross road is selected as the alternate design. Assumed design and traffic operations data are developed and described for each case.

The purpose is to use the case study design to demonstrate the cost effectiveness evaluation process. This process includes estimation of initial project costs, on-going agency costs, and user costs. These data are key inputs for the benefit/cost analysis. The concept continues to focus on the key cost components that influence cost effectiveness with particular emphasis on SPUIs. The reader of the report will be further sensitized to the critical nature of the key factors and their impact on cost effectiveness. This analysis process can be readily applied to other types of interchanges such as the tight diamond, or TUDI.

Design Definition for Case Study

The base case for the illustrative study is an AGI of two major arterials in a developing suburban environment of a large city. It is assumed that the AGI is an existing signalized intersection, has already been upgraded to almost its maximum practical size, and the traffic demand is approaching the capacity of the upgraded intersection.

A plan view of the existing AGI is presented in Figure 91. The major road includes six lanes of through traffic and dual left turn lanes. The minor cross road has four lanes with left-turn bays. Existing traffic volumes have been recorded for the AGI for all 24 hours. The total entering volume in 1990 was about 82,000 vpd. The 1990 p.m. peak-hour traffic movements are depicted in Figure 91. Table 9 provides 1990, 2000, and 2010 projected traffic volumes for all movements for the 7 to 8 a.m., 12 to 1 p.m., 5 to 6 p.m., and 9 to 10 p.m. hourly time periods. Traffic growth was assumed to be about 3 percent per year. The assumed traffic volumes would cause significant congestion in the afternoon peak since the average volume-to-capacity ratio on the existing intersection is about 1.02. The morning rush will also experience major delay, but is slightly undersaturated with a v/c ratio of about 0.95.

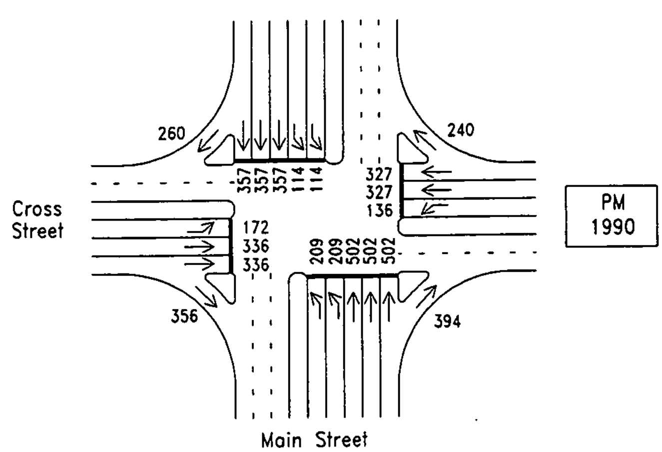


Figure 91. Existing at-grade intersection (AGI) with 1990 P.M. Traffic volumes.

Table 9. Traffic volume estimates for 4 hours for 1990, 2000, and 2010.

DATA USED FOR TRANSYT - 7F RUNS FOR 1990

END HOUR	NORTHBOUND			SOUTHBOUND			EASTBOUND			WESTBOUND			CYCLE LENGTH	
	L	T	R	L	T	R	L	T	R	L	T	R	AGI	SPUI
8	350	1035	137	228	1334	193	242	727	237	360	695	232	105	80
13	246	812	172	156	800	156	142	483	205	143	492	159	95	75
18	404	1458	381	236	1072	268	173	691	285	142	709	240	100	80
22	139	460	98	88	453	88	80	274	130	90	270	90	90	75

DATA USED FOR TRANSYT - 7F RUNS FOR 2000

END HOUR	NORTHBOUND			SOUTHBOUND			EASTBOUND			WESTBOUND			CYCLE LENGTH	
	L	T	R	L	T	R	L	T	R	L	T	R	AGI	SPUI
8	470	1390	184	307	1793	260	326	964	319	467	934	311	115	90
13	331	1091	231	209	1075	209	190	650	276	192	661	213	105	85
18	543	1959	512	318	1440	360	232	929	383	191	953	323	110	90
22	187	618	131	118	609	118	108	368	175	121	362	121	100	80

DATA USED FOR TRANSYT - 7F RUNS FOR 2010

END HOUR	NORTHBOUND			SOUTHBOUND			EASTBOUND			WESTBOUND			CYCLE LENGTH	
	L	T	R	L	T	R	L	T	R	L	T	R	AGI	SPUI
8	502	1869	247	412	1808	349	394	989	428	483	1160	419	125	100
13	444	1467	311	281	1445	281	256	873	370	258	889	287	115	95
18	729	2633	689	427	1936	484	312	1248	515	256	1281	434	120	100
22	252	830	176	159	818	159	145	494	236	162	487	162	110	95

SATURATION FLOWS USED FOR THE MOVEMENTS

	NORTHBOUND			SOUTHBOUND			EASTBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
AGI	3271	5091	1800	3271	5091	1800	1866	3555	1866	1866	3555	1866
SPUI	3650		1800	3650		1800	3612	3280	1800	3323	3280	1800

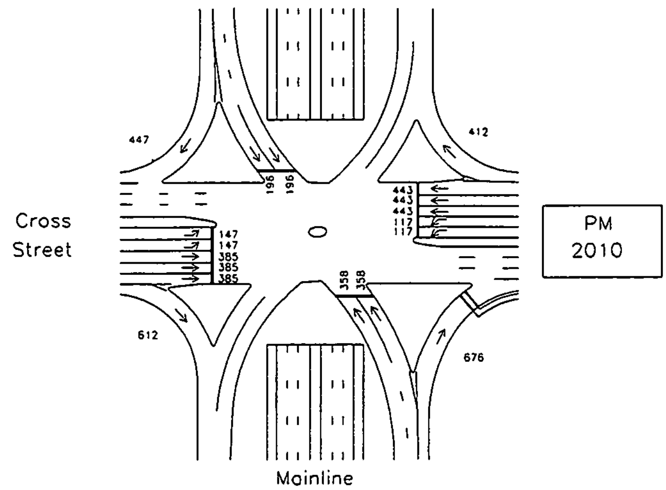


Figure 92. Proposed high capacity 6-2-2-1 SPUI with 2010 P.M. traffic volumes.

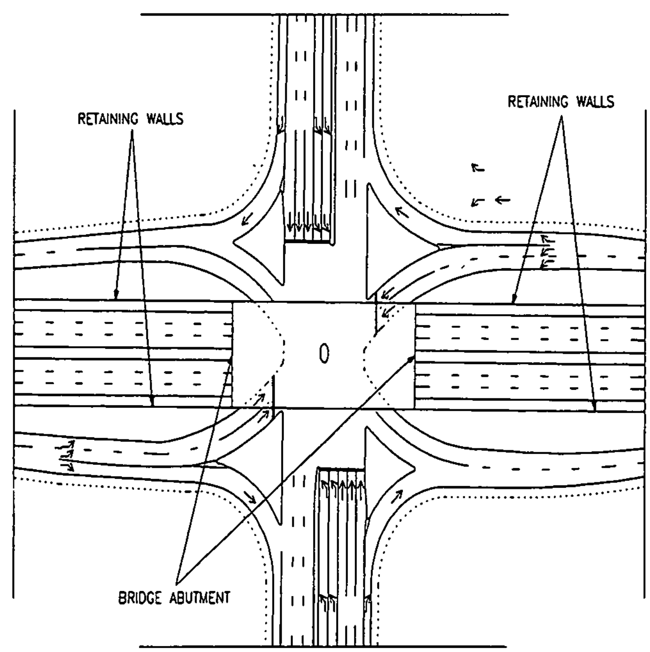


Figure 93. Proposed 6-2-2-1 SPUI construction details.

A plan view of the proposed new 6-2-2-1 SPUI design is shown in Figure 92. As this is a high-growth area of the city, an efficient, high-capacity design is proposed. Consequently, the major road over the cross street will have six lanes for through traffic. Likewise, the minor cross road will be upgraded to a six-lane facility regardless of the type of interchange selected. Dual left turn movements onto and off of the access ramps are incorporated into the design to provide sufficient capacity for design year 2010. Single-lane right turns are shown for entrance and exit to and from the access ramps. Traffic volumes have been forecasted for the SPUI for each hour of the design year, four hours of which are presented in Table 9, for the 3 percent traffic growth rate.

The SPUI configuration selected represents a composite of desirable features found in those existing SPUIs analyzed in the field survey. The design is considered to be realistic. The conceptual design of the SPUI, as shown in Figure 93, is expanded to include boundary limits and a profile of the major road as delineated in Figure 94.

The design sequence to derive the SPUI configuration shown in Figure 94 is as follows. The bridge length is first determined. As noted previously, bridge length is a function of the number and width of turning lanes, median width of the cross street, separation distance of the entrance and exit ramps and sight distance. These design variables are used to calculate the bridge length for the SPUI presented.

Bridge depth together with the minimum vertical clearance height of the structure from the cross street is used to determine the profile grade line of the major road. The thickness of the bridge is a function of its length, construction materials, and design loads. A quick and simple method to estimate bridge depth is to use Figure 65 from 1989 AASHTO bridge design manual (*25*). This is not precise but may be sufficient for conceptual estimating. It was the approach used to determine bridge depth for the SPUI in Figure 94.

The new profile grade lines (PGLs) for both the main highway and the arterial are then established using standard vertical curve equations and an appropriate design speed of 60 mph in this case. The point where the distance between the bridge and the arterial is the smallest is the critical point that controls vertical alignment and, hence, the profile grade line. In the SPUI shown in Figure 94, this point is located at the edge of the major road and the center of the arterial.

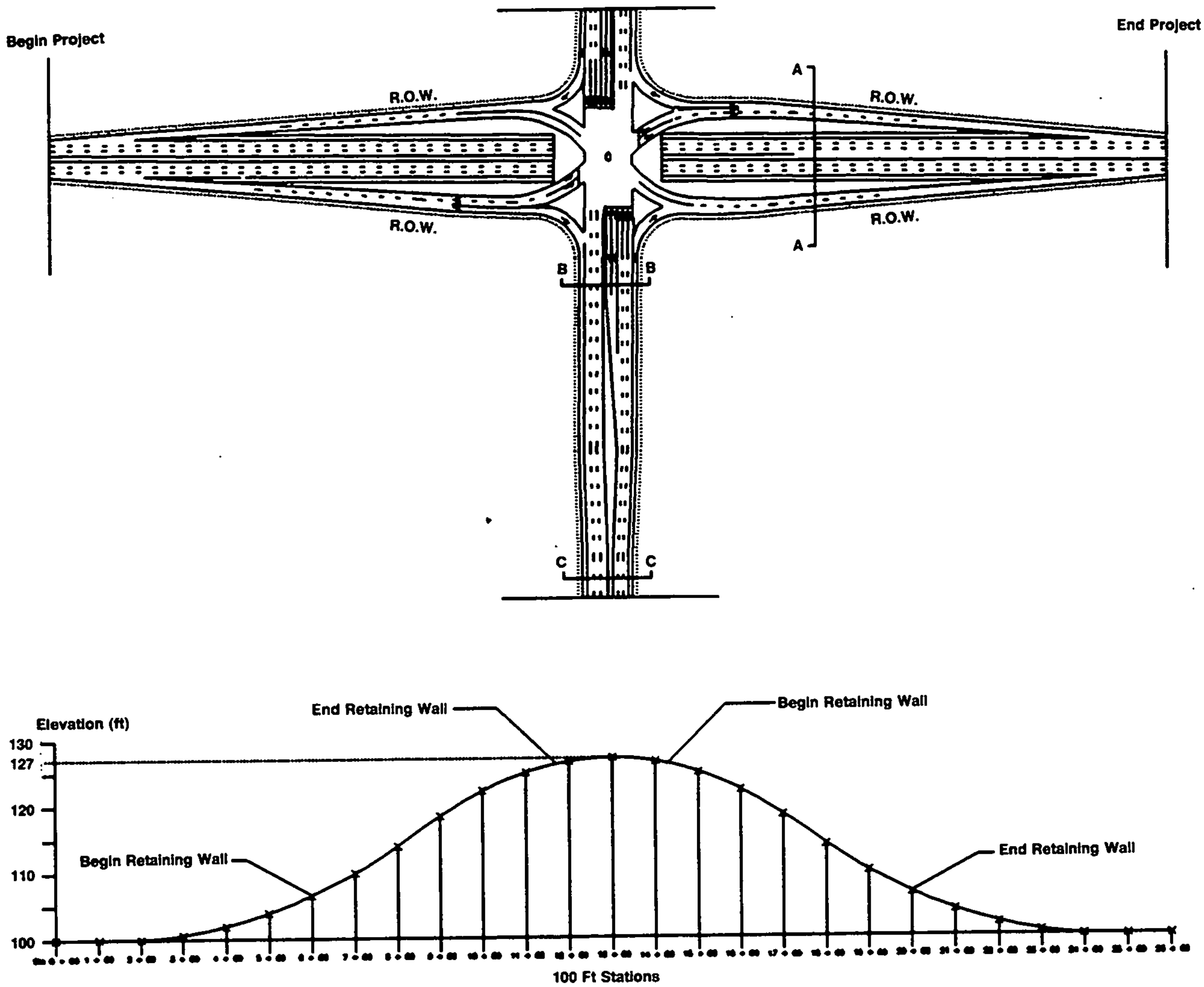


Figure 94. SPUI boundary plan and profile gradeline.

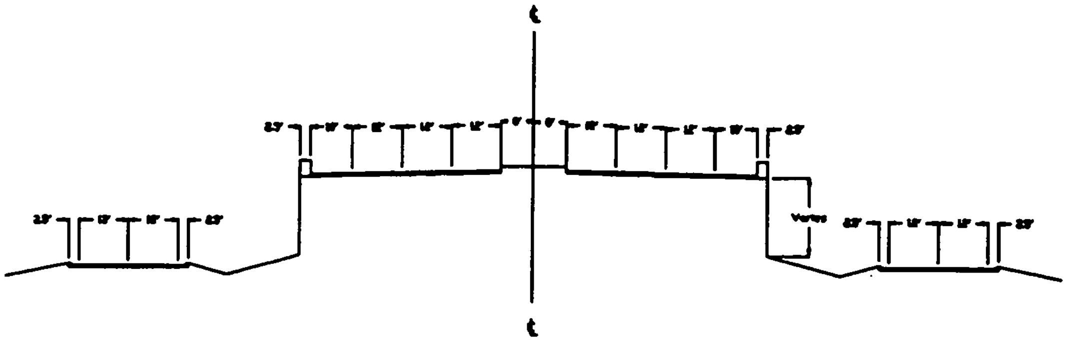
The new profile grade lines, the additional road area needed to accommodate projected traffic volumes, horizontal transitions necessary to tie into existing pavements, and the maintenance of traffic all interrelate to define the boundary limits of the SPUI. These limits may be confined by existing or new right-of-way required and the cost of right-of-way. In this example, right-of-way was determined by the location of the ramps and cross streets. It is assumed that sufficient right-of-way can be acquired. This may not be the case in actual practice.

As delineated in Figure 94, the match lines for the major road and the cross-road arterial are provided on the plan. The distance from where the new realigned major road deviates from the normal at grade section until it returns to the normal at-grade road is approximately 2,600 ft. The cross street distance for the SPUI estimate is 1,820 ft from where the cross street changes to six lanes and returns back to the current lane configuration of the existing cross street. Retaining walls are incorporated into the design as shown on the profile in Figure 94. The maximum height of the major road is approximately 27 ft. This height is comprised of a 17.5-ft clearance and a bridge thickness of 9.5 ft. The bridge structure for the SPUI consists of a single span steel plate girder design with an approximate span of 240 ft. The maximum grade of the major road is assumed to be 4 percent based on a 60-mph design speed for freeways. The right-of-way boundaries are shown on the plan as dotted lines.

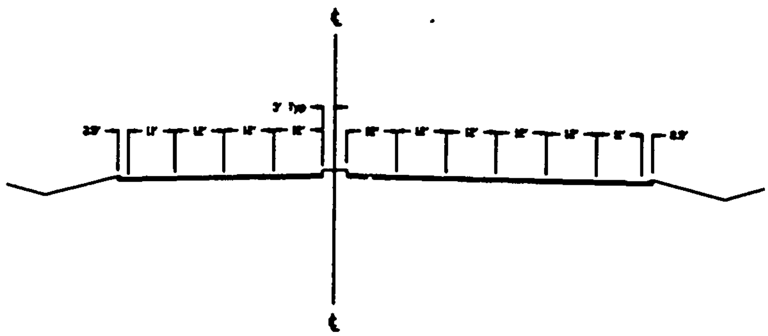
Figure 95 provides typical cross sections of both the major road and the cross-road arterial. These sections give dimensions for lanes, medians, shoulders and so on. The location of the cross section is noted in Figure 94. Pavement construction is continuously reinforced concrete for the major road and jointed concrete for the cross road and ramps.

Estimation of Initial Costs

The cost estimation process that is illustrated provides a basis for the comparative analysis of alternative designs during the conceptual planning stages of a project. The conceptual estimate is an estimate that is developed with limited design information. As a result, key parameters are employed to derive project costs. The type of information available at this stage of the decision-making process normally includes existing topography (terrain, existing roadway alignment, drainage, utilities), forecasted traffic volumes, existing available right-of-way, and the level of commercial development in adjacent areas.

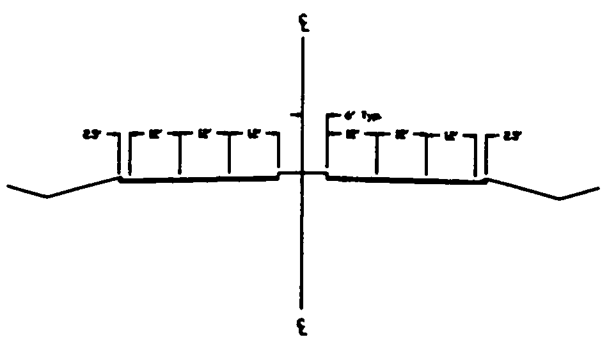


Cross-Section A-A (Major Road)



Cross-Section B-B (Cross Road)

(Before Taper)



Cross-Section C-C (Cross Road)

(After Taper)

Figure 95. SPUI major and cross-road cross-sectional views.

A dependable cost estimation of any project cannot be performed in isolation. Existing conditions, traffic volumes, and the physical characteristics of a site must be understood so that an accurate forecast of the future needs of the intersection are obtained. Conceptual estimating should involve several professionals. The team should include a designer, estimator, and someone with practical construction experience or someone with a combination of these experiences. Construction experience is important, especially when estimating SPUIs, because first-hand knowledge of the SPUI may be limited.

The conceptual estimate will produce results that are within 15 to 25 percent of the actual costs. This is a typical accuracy range for a parametric-type estimate based on conceptual designs. It is desirable for the estimate to be on the high side. The reason that the estimate should be higher than the actual construction costs is to ensure that sufficient financing is available to complete the project. This range may seem unreasonable, but given that each design alternative will be evaluated in the same manner and that different designs are compared to each other on an incremental basis, any systematic error will have a minor effect on the net result. Each feasible design will be compared to a standard base design. This base case design could be either an existing condition or another alternative design. In the example presented, an at-grade-intersection is the base condition and the proposed SPUI is the alternate design.

Key Parameter Evaluation

Figure 89 identifies key cost components that are considered when estimating the construction cost of a SPUI. They will be used to illustrate the conceptual estimating process applied to the SPUI shown in Figure 94. The major components include: (1) bridge; (2) pavement for the highway (major road), ramps, and arterial (cross street); (3) earthwork; (4) retaining walls; (5) signalization; (6) lighting; (7) curbs; (8) mobilization; (9) traffic control; (10) drainage; (11) minor items such as striping, signing, sidewalks, and landscaping; and (12) demolition. In addition to construction costs, right-of-way, engineering and design, contract administration and contingency costs are estimated. The sum of all cost components represents the total project cost.

The first seven components are estimated based on a single key parameter that is quantified and priced from an appropriate unit cost representative of the type, time, and location of construction. The next five components (items 8 through 12) are estimated on a percentage basis where the percent is derived from historical data such as provided in previous sections. Demolition, when required, is quantified using the appropriate parameter. Right-of-way is always quantified because of its critical impact on total project cost and design features. The remaining three components are estimated as a percent of total construction cost (items 1 through 12). Specific techniques and example calculations are described in the section to follow.

The Estimation Process

The estimated project costs presented are considered typical for open shop construction in the Southern United States. These costs are estimated in first quarter 1990 dollars. The estimate also assumes that site restrictions and typography will have a minimal effect on costs, that is, there are few unusual site characteristics that would inflate the unit cost of any of the construction components. This infers that the unit costs used in this estimate are mid-range costs. Costs for any given site could vary in either direction from the mid-range costs that are incorporated into this estimate. Another basic assumption is that the interchange is being built as part of other ongoing reconstruction, therefore, unit costs would reflect some volume discount.

At the conceptual level, the final component cost, i.e., the quantity times its unit price, encompasses all of the costs associated with that component. For instance, the bridge quantity is defined as the horizontal surface area of the bridge deck. The quantity and the respective unit price are used to estimate the cost of the total bridge including the cost of the superstructure and substructure.

Quantities for the key components of the SPUI, along with a few minor items, are calculated using the design information in Figures 93, 94, and 95. The results are presented next.

Bridge. The cost of the bridge structure is estimated using the horizontal area of the bridge deck, in square feet, and a unit cost that reflects steel plate girder construction. The horizontal area is derived by measuring the adjusted length of the bridge. This adjusted length is the length from bearing seat to bearing seat plus 4 ft. A typical dimension for a bearing seat is 1.75 ft. The total width of the bridge out-to-out is determined by summing the width of all traveling lanes, medians, shoulders, and two 1.75 ft sections for concrete traffic barriers. The adjusted length and the width are multiplied together to calculate the horizontal area in square feet.

The adjusted bridge length is 242.75 ft for the SPUI case. The out-to-out width of the major road is 111.5 ft. This results in an adjusted bridge area of 27,000 sq ft. The estimated unit cost for a steel bridge of this size and type is $85.00 per square foot. The potential range of unit costs for bridges of this class can vary from $70.00 to $100.00 per square foot depending on bridge span, depth, type (plate or trapezoidal box girders), location, construction access, and other related factors. The final item cost reflects all elements of costs associated with the bridge.

Because the bridge is a critical component in SPUI cost estimation, it would be prudent to determine whether a concrete box girder is a feasible alternative. Unit costs for these types of bridges can range from $60.00 to $75.00 per square foot in some locations.

Pavement. The cost of pavement is estimated using square area of pavement. The horizontal lengths and widths are measured from the conceptual design drawings and appropriately multiplied together. This produces the quantity of pavement in square feet. The pavement is separated into three unique categories: the highway (major road), the ramps, and the arterial (cross road). This enables pricing to be based on different pavement designs. The pavement design for the highway is assumed to be a 12-in. continuous reinforced concrete pavement (CRCP). Nine-in. jointed concrete pavement (JCP) is assumed for the arterial and ramps.

The quantity analysis resulted in an estimate of 266,000 sq ft of highway, 190,000 sq ft of arterial, and 134,000 sq ft of ramps. The potential range of unit costs for pavement is between $1.75 to $6.00 per square foot. The unit costs used in the estimate are $4.00 and $2.00 per square foot for CRCP and JCP, respectively. The final item cost reflects all the costs associated with the

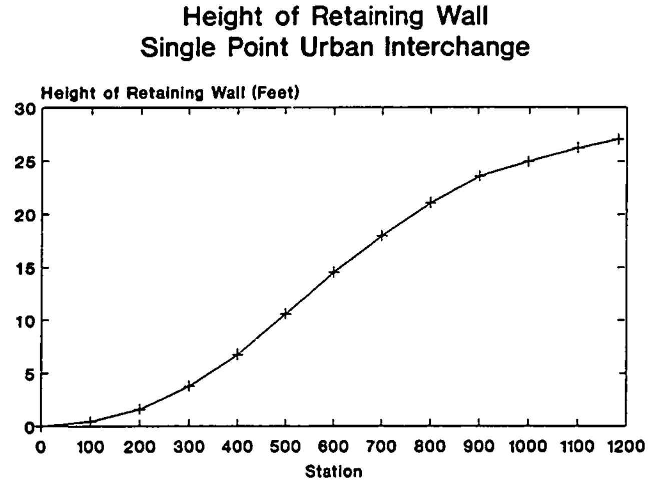


Figure 96. Height profile of retaining wall for SPUI.

pavement, i.e. forming, rebar placement, subgrade preparation, and so on.

Earthwork. The cost of earthwork is determined by multiplying the volume of soil compacted in place by the respective unit price. Because an at-grade intersection is used as the base case and no irregularities are assumed to be present, which typically would not be the case, the quantity of earthwork in this particular case is tight. Variations that would be present in the actual construction of the SPUI are accounted for in the contingency category of the estimate.

The end volume method is used to calculate the volume of soil. Quantity takeoff assumptions are: (1) 0.02 ft/ft cross slope of the roadways; (2) the highway would be 12-in. continuous reinforced concrete pavement; (3) 3/4-in. bond breaker is used; (4) 20 percent additional soil is required to account for compaction; and (5) the haul distance is 20 miles because of the urban setting. The compaction shrinkage of 20 percent is assumed for purposes of this illustration. The compaction rate is dependent on actual soil characteristics specific to each site. Due to the symmetry of this case, the volume of soil in one quadrant is calculated and then multiplied by four to estimate the total quantity of earth fill required.

The quantity of earth fill is 129,000 cu yd. The unit cost is $9.00 per cubic yard based on select fill and the assumptions given above. The unit costs for earthwork can vary from as low as $2.00 per cubic yard to as much as $14.00 per cubic yard. The unit price depends heavily on the type of fill specified, availability of materials, haul distances, and volume of fill required. The final item cost reflects all the costs associated with earthwork, i.e., compaction, hauling a typical distance for an urban setting, dozing, blading, scarifying, and so forth.

Retaining Walls. The cost of retaining walls is estimated by multiplying the surface area of the retaining wall by the appropriate unit price. Figure 96 shows the height of the retaining wall along the highway. Precast retaining walls are used as opposed to a cast-in-place approach.

The area of the retaining wall along one side of the highway is estimated by calculating the average height every 100 ft and then multiplying it by 100 ft. All 100-ft segments are summed to derive the square area of wall surface. The square area of both sections of retaining wall under the bridge is determined by the overall width of the highway times the height of the retaining wall at the bridge.

Because of the symmetry of the design, retaining wall area of one quadrant is first calculated and then multiplied by four. The two sections under the bridge are added to derive the total quantity of retaining wall area.

The total area of the retaining wall is 67,000 sq ft. The unit price for precast retaining walls ranges from $15.00 to $25.00 per square foot. The unit price used in this estimate is $20.00 per square foot. Cast-in-place retaining walls are generally more expensive and can be as much as $45.00 per square foot. The final item cost reflects all the costs associated with erecting retaining walls, i.e., placement of the footers, attachment of the wire mesh, and compaction of the cement stabilized sand.

Signalization. The cost of signalization is calculated by multiplying the number of signal heads that the project requires by the appropriate unit price. Based on the inspection of other SPUIs 12 heads are deemed to be appropriate. This quantity is then multiplied by the unit price to produce the final item cost. This final item cost reflects all the costs associated with construction of the signal system, i.e., foundation, pole, wiring, transformer, detectors, signals. The unit price estimated for the traffic actuated signalization is $10,500 per head.

Lighting. The cost of lighting is estimated based on the number of poles that a SPUI might typically require and the appropriate unit price. A typical height of the light poles would be between 40 and 50 ft. Based on the inspection of other SPUIs 34 poles are determined to be appropriate. This quantity is determined by that portion of the project that would typically be illuminated and the area each light pole illuminates. This quantity is multiplied by the unit price to produce the final item cost. This final item cost reflects all the costs associated with construction of the lighting system, i.e., foundation, pole, wiring, and so on. The unit cost for lighting is based on $5,700 per pole.

Curbs. The cost of curbs is calculated from the lineal feet of curb and the appropriate unit price. The quantity of curb that is required is measured directly from the conceptual design drawings. A total of 12,000 lineal feet is calculated for the curbing. It is assumed that 6-in. by 24-in. curbing is used. The final item cost reflects all the costs associated with construction of the curb. Curb unit costs can vary from $8.00 to $12.00 per lineal foot. Ten dollars a lineal foot is used in this estimate.

Mobilization. This category covers the costs associated with the contractor moving equipment and personnel onto and off of a project. Four percent of the sum of the previous seven cost elements is used.

Traffic Control. The degree of additional paving, signing, striping, maintenance of detours, and traffic handling are all reflected by traffic control costs. Traffic control costs are affected by the area of right-of-way available, the type of bridge selected, whether the intersection is at capacity, the number of phases that the project requires, and whether complex turning movements are necessary. Seven percent of the sum of the first seven cost elements is considered adequate. Since steel bridge construction is used, it is assumed that proper staging of construction can occur without the need for a significant detour system. It might be prudent to devise a bridge erection plan to confirm such an assumption in actual practice, especially when there is limited familiarity with SPUI construction.

Minor Items. Typical items contained in this category include striping, signing, sidewalks, and landscaping. Three percent is determined to be appropriate for this SPUI. This percent is applied to the sum of all previous cost elements.

Drainage. Drainage may include the installation of storm sewers, pump stations, inlets, and all related work that is needed to carry water away from the interchange area. It is assumed that the drainage for the project would be a gravity flow type and that the existing storm sewers will be able to handle the added demand. Ten percent is used for this illustration. This percent is applied to the sum of all previous cost elements exclusive of minor items. Because of the site-specific nature of drainage requirements the percent selected can be determined only after careful study of the project site conditions.

Demolition. Demolition of existing pavement is required for the case study example. The cost of demolition includes breaking existing pavement and removal to an existing dump. The total area of pavement to be removed is estimated at 409,000 sq ft. A cost of $1.25 per square foot is applied. This assumes a haul distance of approximately 20 miles. Demolition is project specific and generally applicable to reconstruction projects. This category should be analyzed closely in actual practice. This particular case study assumes that additional utility line relocation is not required. Every site should be carefully evaluated to detect if utility lines must be relocated.

Engineering and Design. The detailed design and initial survey of the project are included in this cost. Nine percent of total construction cost is used for engineering and design.

Supervision and Administration. This category includes the cost to the organization or state agency that is responsible for inspecting the construction of the project and the reporting of progress and scheduling. Nine percent of total construction costs is used for supervision and administration.

All items that are estimated on a percentage basis assume that the case is average in every way. Unfortunately, this rarely occurs in practice as each individual project will have its own unique characteristics. An appropriate percent should be used from an agency's own data base of projects.

Right-of-Way. The area that is needed to construct the case study SPUI is taken from the conceptual design drawings. This area is superimposed onto the area currently being used by the AGI. The difference between the two is calculated. This difference includes a 10-ft easement and is the incremental amount of right-of-way that is needed to implement the SPUI design. The AGI is assumed to be at the existing available right-of-way. The total right-of-way for the SPUI and AGI is 867,000 and 481,000, respectively, given a 386,000-sq ft difference.

The incremental right-of-way area calculated is considered a clean quantity, which will never be the true case. The quantity is based on square feet and the cost is derived on a dollar per square foot basis. The cost of land is highly dependent on the location, area required, and specific economic viability of any given parcel of land. A typical price of open urban land varies anywhere from $1.00 to more than $50.00 per square foot. This can cause the viability of any design to wane and a sensitivity analysis often is warranted. The unit cost for right-of-way is $7.00 per square foot, and this assumes moderate land cost without significant problems in obtaining the area needed. This is seldom the case in practice, thus, local knowledge of right-of-way costs and availability of land at a given site should be investigated and used in specific project estimates.

Contingency. This item covers those unforeseen items and other uncertainties associated with the development of the conceptual design where there is lack of specific details. The percent contingency selected for the case is 20 percent of all project costs including right-of-way. The summary of all project cost items is presented in Table 10 based on the quantities calculated and the respective unit cost presented. The estimated total project cost for the SPUI is $16,304,000.

Table 11 provides the percent weight by cost of each category to the total cost of construction (see column three). The four primary cost categories (bridge, pavements, earthwork and retaining walls) represent 70.6 percent of the total cost of construction with the bridge having the highest percentage at 25.0 percent. These data compare favorably with the ranges presented in Figure 89. This type of comparison is a typical methodology for verifying an estimate against known standard ratios. Table 11 also shows the percent weight of all items to total project cost, excluding contingency, in the last column. The cost of additional right-of-way represents 19.8 percent of the total. This percentage appears reasonable.

Cost Impact of Other SPUI Design Considerations

The SPUI estimated in this case study reflects one set of design features. Because project site conditions differ depending on location, alternate SPUI designs may be considered in practice. Table 12 attempts to capture the impact different design considerations might have on project costs. Seven major design considerations are identified in Table 12 as the major roadway, bridge type, bridge span, retaining walls, ramp separation distance, drainage,

Table 10. Project cost estimate for 6-2-2-1 overpass SPUI.

Project Components	Total Quantity	Units	Unit Price	Total Cost
Bridge Structure	$ 27,000	SF	$ 85.00	$ 2,295,000
Pavements				
Road Over	266,000	SF	4.00	1,064,000
Arterial	190,000	SF	2.00	380,000
Ramps	134,000	SF	2.00	268,000
Earthwork	129,000	CY	9.00	1,161,000
Retaining Walls	67,000	SF	20.00	1,340,000
Signalization	12	Heads	10,500.00	126,000
Lighting	34	Poles	5,700.00	194,000
Curbs	12,000	LF	10.00	120,000
Subtotal				6,948,000
Mobilization			4%	278,000
Traffic Control			7%	486,000
Subtotal				7,712,000
Minor Items			3%	231,000
Drainage			10%	771,000
Demolition	409,000	SF	1.25	511,000
Construction Cost				9,225,000
Engr/Design			9%	830,000
Constr Admin/Inspct			9%	830,000
ROW	386,000	SF	7.00	2,702,000
				13,587,000
Contingency			20%	2,717,000
Total Project Cost				$ 16,304,000

First Quarter 1990, Southern U.S. Construction Costs

Table 11. Analysis of key cost components for the SPUI.

Project Components	Total Cost	Percent Construction Costs	Percent Total
Bridge Structure	$ 2,295,000	25.0%	16.9%
Pavement			
Road Over	1,064,000	11.5%	7.8%
Arterial	380,000	4.1%	2.8%
Ramps	268,000	2.9%	2.0%
Earthwork	1,161,000	12.5%	8.6%
Retaining Walls	1,340,000	14.6%	9.9%
Signalization	126,000	1.4%	0.9%
Lighting	194,000	2.1%	1.4%
Curbs	120,000	1.3%	0.9%
Subtotal	6,948,000	75.3%	51.2%
Mobilization	278,000	3.0%	2.0%
Traffic Control	486,000	5.3%	3.6%
Subtotal	7,712,000	83.6%	56.8%
Minor Items	231,000	2.5%	1.7%
Drainage	771,000	8.4%	5.7%
Demolition	511,000	5.5%	3.8%
Construction Cost	9,225,000	100.0%	67.9%
Engr/Design	830,000		6.1%
Constr Admin/Inspct	830,000		6.1%
ROW	2,702,000		19.8%
	13,587,000		100.0%
Contingency	2,717,000		
Total Project Cost	16,304,000		

First Quarter 1990, Southern U.S. Construction Costs

and vertical alignment. Six major project cost components (right-of-way, bridge, retaining walls, earthwork, drainage, and pavements) are evaluated for each design consideration in terms of the potential impact on total project cost. This assessment is intended to be qualitative in nature and to provide an indication of the possible direction of change in project cost due to a change in a design consideration. It is offered as a guide to help identify potential cost impacts because of design changes made to the base case SPUI. Table 12 does not include the effects of changes in user costs as a result of a change in design. This impact also must be considered.

The SPUI described in the case study is defined as the base case in Table 12 and is denoted as "B". The intersection of a design consideration and a key cost component is described either by more cost ("+") or less cost ("−") or similar cost ("S") when compared to the base case design. The key cost components are ordered from left to right by the relative magnitude of impact on total project cost. Thus, a change in the right-of-way and bridge cost components will have the greatest potential impact on total project cost.

One application of Table 12 is illustrated with ramp separation distance. This is a design consideration that influences the geometric features of the SPUI. Its proper evaluation is crucial to operational efficiency, bridge configuration, and right-of-way acquisition. The critical measure is the separation distance between the SPUIs on ramps and off ramps as determined by the longest out-to-out dimension between ramps. This distance may be constrained by existing right-of-way available along the major road. If right-of-way is plentiful and relatively inexpensive, ramp separation distance may be wider. A more robust ramp radius of curvature will result in a shorter bridge center span with potentially lower bridge costs. A shorter bridge span reduces bridge thickness and, hence, vertical alignment. This will reduce earthwork and retaining wall cost. A wider ramp separation distance also may improve operational efficiency because of better sight distance for drivers making left turns off of the ramps.

Table 12 provides a qualitative assessment of the project cost impact of a change to ramp separation distance. The base case SPUI has an ideal separation distance. According to Table 12 if the separation distance is wider, i.e., increased, the estimated cost of right-of-way would correspondingly increase, bridge costs would decrease and retaining walls, earthwork, drainage and pavement costs would increase. The overall impact would likely be a net increase in total project cost. However, the magnitude of the increase will depend on quantities and unit costs for each item and, in particular, the unit cost of right-of-way acquisition when new right-of-way must be acquired.

Table 12 also can be used to assess the potential effect of changes to several design factors. As an example, assume the base design is changed to: (1) a trapezoidal steel box girder, using sloped embankments in lieu of retaining walls; and (2) a narrow ramp separation distance. With these design adjustments, right-of-way costs increase when sloped embankments are specified, but decrease as ramp separation distance narrows. The net impact on project cost may be offsetting. A narrow ramp separation distance increases the bridge span and thickness and the corresponding cost of the bridge. The trapezoidal box design might reduce bridge thickness, but the fabrication and construction of the trapezoidal box elements are assumed to be more costly. The net impact on bridge cost is a potential increase. Retaining wall cost is deleted and replaced with increasing earthwork cost. Narrowing the ramps would reduce the earthfill required. The net impact of these changes to earthwork and retaining wall design is likely to be a decrease in cost, because construction of retaining walls is usually more costly in relation to earthfill operations. The overall impact of all three changes to the base design may be a slight increase in total project cost. This would have to be verified by quantity evaluation and adjustments to the appropriate unit costs.

If there is an increase in total project cost, there should be a reduction in user cost associated with the design change as reflected in greater capacity. This reduction in user cost translates into increased benefits due to improved operational efficiency. This incremental increase in benefits should be greater than the incremental increase in cost to warrant the change to a wider ramp separation distance or other changes to key design considerations.

Several general comments can be made about the information in Table 12. At the conceptual planning level, four design considerations should be critically evaluated. They are bridge type and materials of construction, bridge configuration, retaining wall requirements, and ramp separation distance. The four design considerations are interrelated and their impact on project cost is complex. The direction of change in project cost due to changes in design should be understood and carefully evaluated. The best approach is to develop quantitative data to analyze the magnitude of the impact of design changes related to the four key considerations. Moreover, the design team also should consider the benefits derived from such changes. Increases in project costs should be justified by corresponding increases in user benefits, i.e., benefit/cost greater than one. This approach will pro-

Table 12. Qualitative assessment of the impact of key design considerations and key cost components for SPUIs.

DESIGN CONSIDERATIONS	KEY PROJECT COST COMPONENTS: RIGHT OF WAY	BRIDGE	RETAINING WALLS	EARTHWORK	DRAINAGE	PAVEMENTS: HIGHWAY	ARTERIAL	RAMPS
MAJOR ROADWAY								
OVER	B	B	B	B	B	B	B	B
UNDER	S	-/S	-	S	+	S	S	S
BRIDGE								
TYPE								
I-BEAM								
STEEL	B	B	B	B	B	B	B	B
CONCRETE	S	+/-	+	+	S	S	S	S
TRAPAZOIDAL								
STEEL	S	+/-	-	-	-/S	S	S	S
CONCRETE	S	-	-	-	-/S	S	S	S
SPANS								
SINGLE	B	B	B	B	B	B	B	B
MULTI	S	+/-	-	-	S	-	S	S
RETAINING WALLS								
PRECAST	B	B	B	B	B	B	B	B
CAST-IN-PLACE	S	S	+	S	S	S	S	S
NONE	+	S	-	+	+	S	S	+
RAMP SERPARATION DIST.								
IDEAL	B	B	B	B	B	B	B	B
WIDE	+	-	+	+	+	S/+	+	+
NARROW	-	+	-	-	-	S/-	-	-
DRAINAGE								
CAPACITY AVAILABLE	B	B	B	B	B	B	B	B
NO EXISTING	+/S	S	S	S	+	S	S	S
AT CAPACITY	+/S	S	S	S	+	S	S	S
VERTICAL ALIGNMENT								
CHANGE HIGHWAY ONLY	B	B	B	B	B	B	B	B
CHANGE ARTERIAL ONLY	S	S	+/-	S	+	S	S	S
CHANGE BOTH	S	S	+/-	S	+	S	S	S

Legend

B =	Base Case 6-2-2 SPUI
S =	Similiar Costs
+ =	Costs More
- =	Costs Less

duce the most cost-effective SPUI design for the given site conditions. The techniques described in this chapter should provide a method for making such evaluations.

Estimation of Annual Costs

The estimation of operating and maintenance costs was derived from data collected from several state transportation department. These data are considered approximate in nature and are believed to be representative of operating and maintaining SPUIs. The operating cost estimate includes the cost of operating the signals and lights. The electricity consumed by the lights and signals is the primary operating cost. Maintenance costs include: the signals and lights (changing light bulbs, burned out components, etc.); resealing pavement joints; bridge painting and replacement of guard rails; and upkeep of signs, pavement markers, and striping.

Operating costs are estimated by determining the kilowatt-hours of electricity used by the signals and lights. This is approximated based on the number of signal heads and luminaires for a given interchange design. Based on the kilowatt-hour consumption of each item, the total anticipated use per year and the cost per kilowatt-hour, an estimate of annual operating cost is determined.

Maintenance costs are estimated based on empirical data collected from similar bridge designs and pavement designs that are available. The frequency of repairs and cost at the time of repair for the pavement is assumed to occur at 10, 20, and 28 years. The costs at the time when maintenance occurs is determined and

Table 13. Average daily traffic for different traffic movements and roadway, by year.

	Traffic Movements			
	Through	Left-turn	Right-turn	Total
Year 1990				
Arterial	33,733	8,127	6,242	48,102
Cross Street	18,902	6,288	8,117	33,307
Year 2000				
Arterial	45,334	10,922	8,389	64,645
Cross Street	25,403	8,451	10,909	44,763
Year 2010				
Arterial	60,862	14,667	11,359	86,888
Cross Street	34,140	11,358	14,665	60,163

converted to a net present value. These costs are then annualized using a 4 percent discount rate over the duration of the maintenance period. Maintenance of the signals and lights is estimated using historical data of average costs of similar interchanges. Maintenance of the bridge includes painting the structure every 10 years. Repair and upkeep of the signs, pavement markers, and striping is determined based on past history. All of these costs are represented as annual costs.

The total annualized amount of on-going costs is estimated to be approximately $47,000 for the SPUI and $32,000 for the AGI. The net present value of the on-going cost for the SPUI is $640,000 based on a 20-year life and a 4 percent discount rate. This represents about 7 percent of the total construction cost. The net present value of the on-going cost for the AGI is $440,000.

Summation of the Initial Costs

The initial costs required for the benefit/cost analysis are summarized as follows for both cases:

Base Case—At Grade Intersection	
Project Costs	Not Required
Operating and Maintenance Cost	$440,000
Alternate Case—SPUI with Major Road Over	
Project Costs	$16,304,000
Operating and Maintenance Cost	$640,000

These initial costs reflect 1990 costs.

Estimation of User Costs

Assumptions. It is assumed that all vehicles are traveling at 35 mph for the approach or mid-block speed, i.e., S = 35mph. It is assumed that the proportion of trucks for calculating weighted vehicle operating costs and values of time is 10 percent.

Average daily traffic. The average daily traffic for 1990, 2000, and 2010 for the different movements are as given in Table 13.

Estimation of delays and stops. The amount of delay and the number of stops can be estimated by several different methods. The TRANSYT-7F computer program was used in this study to estimate delay and stops for the intersection and the SPUI. TRANSYT-7F has several advantages, including the ability to evaluate many different types of design alternatives. TRAF-Netsim and Highway Capacity Software could have been used.

Table 14. Hours of delay and number of stops per day for each alternative, by year.

	At-Grade Intersection	Single-Point Urban Interchange
Hours of Delay		
Year		
1990	567	176
2000	1,056	265
2010	2,519	436
Number of Stops		
Year		
1990	52,315	22,569
2000	75,059	31,235
2010	131,297	43,221

Table 15. Distance traveled in study area per vehicle by type of movement and alternative.

At-Grade Intersection

	Distance Traveled Per Vehicle, Feet		
	Through	Left-turn	Right-turn
Arterial	2,600	2,203.5*	2,116.5**
Cross Street	1,820	2,212.1***	2,116.5****

Single Point Urban Interchange

	Distance Traveled Per Vehicle, Feet		
	Through	Left-turn	Right-turn
Arterial	2,602	2,247.7*	1,901.5**
Cross Street	1,820	2,235.0***	1,946.5****

* Turning left from arterial to major cross street.
** Turning right from arterial to major cross street.
*** Turning left from major cross street to arterial.
**** Turning right from major cross street to arterial.

The amount of delay and the number of stops was estimated for the example problem for each alternative at three points in time, for the years 1990, 2000, and 2010 and these totals are given in Table 14. It is assumed that the ADT values for these 3 years apply to the beginning of the year 1990, the beginning of year 2000, and the beginning of the year 2010; therefore, there are 10 years between successive estimates. Amount of delay and number of stops per day are multiplied by 365 to obtain annual values (rates per year at the three points in time).

Care should be taken not to overestimate the amount of delay predicted by the computer program for overloaded traffic movements. As a guideline, a projected hourly turning movement volume should not exceed 1.2 times the capacity of the phase to serve that movement. Motorists are presumed to divert to other facilities if congestion becomes too severe. The values given in Table 14 were constrained by this guideline.

Estimation of distances traveled. The overall length of the study area is 2,600 ft along the main route arterial and 1,820 ft along the cross street. The distances traveled through the study area per vehicle for each vehicle movement are estimated by measurement from the preliminary design dimensions and are given in Table 15.

Table 16. Distance traveled per day, for all vehicles, by type of movement and alternative.

At-Grade Intersection	Daily Vehicle Miles of Travel, vm			
	Through	Left-turn	Right-turn	Total
Year 1990:				
Arterial	16,611	3,392	2,502	22,505
Cross Street	6,515	2,624	3,254	12,393
Total Daily Vehicle Miles				34,898
Year 2000:				
Arterial	22,324	4,558	3,363	30,245
Cross Street	8,756	3,541	4,373	16,670
Total Daily Vehicle Miles				46,915
Year 2010:				
Arterial	29,970	6,121	4,553	40,644
Cross Street	11,768	4,759	5,878	22,405
Total Daily Vehicle Miles				63,049

Single Point Urban Interchange				
	Through	Left-turn	Right-turn	Total
Year 1990:				
Arterial	16,624	3,460	2,248	22,332
Cross Street	6,515	2,662	2,992	12,169
Total Daily Vehicle Miles				34,501
Year 2000:				
Arterial	22,341	4,650	3,021	30,012
Cross Street	8,756	3,577	4,023	16,356
Total Daily Vehicle Miles				46,368
Year 2010:				
Arterial	29,993	6,208	4,188	40,389
Cross Street	11,768	4,808	5,406	21,982
Total Daily Vehicle Miles				62,371

The distance traveled in the study area in vehicle miles per day is calculated by multiplying the average daily traffic by the above travel distance in ft for each movement and dividing by 5,280 ft per mile and is given in Table 16.

Cost of delay. The hours of delay per day given previously are multiplied by 365 days per year and the average time cost per vehicle-hour to obtain the annual delay cost, which are summarized by year in Table 17.

The weighted average cost per vehicle is calculated using the assumed 10 percent trucks and 90 percent passenger cars with the values of time per vehicle-hour given previously for each vehicle type, $12.69 for passenger cars and $23.02 for trucks.

Vehicle operating cost of idling. The hours of idling time are equal to the hours of signal delay divided by 1.3. To obtain the annual cost of idling, given in Table 18, the daily hours of idling are multiplied by 365 days and the weighted cost for idling of $0.943 per vehicle-hour.

Vehicle operating cost for stops. The excess cost of stopping per year is calculated by multiplying by 365 days per year and by the weighted average cost of stopping of:

Excess cost of stopping from 35 miles per hour:

Passenger cars = $19.05 per 1,000 stops
Trucks = $223.02 per 1,000 stops
Weighted average = $39.45 per 1,000 stops or $0.0395/veh stop

The number of stops per day given previously is multiplied by 365 and by $0.0395/veh stop to obtain the annual cost of stopping shown in Table 19.

Table 17. Annual cost of delay by alternative and year.

Study Year	Annual Time Cost of Delay: At-Grade Intersection	Single-Point Urban Interchange
1990	$2,840,000	$ 882,000
2000	5,289,000	1,327,000
2010	12,617,000	2,184,000

Table 18. Annual vehicle operating cost for idling by alternative and year.

Study Year	Annual Cost of Idling: At-Grade Intersection	Single-Point Urban Interchange
1990	$150,000	$ 47,000
2000	280,000	70,000
2010	667,000	115,000

Table 19. Annual vehicle operating cost for stopping, by alternative and year.

Study Year	Cost of Stopping Per Year: At-Grade Intersection	Single-Point Urban Interchange
1990	$ 754,000	$325,000
2010	1,082,000	450,000
2010	1,893,000	623,000

Table 20. Daily vehicle-miles, by alternative and year.

Alternative	Daily Vehicle Miles, by Year: 1990	2000	2010
At-grade Intersection	34,898	46,915	63,049
Single-point Urban Interchange	34,501	46,368	62,371

Vehicle operating cost at uniform speeds. The total daily vehicle miles for each alternative for the years 1990, 2000, and 2010 are summarized in Table 20.

The vehicle operating cost per year for operating at uniform speeds through the study area is calculated by multiplying the above miles per day by 365 days per year and by the vehicle operating cost per mile for operation at 35 mph, the assumed mid-block or approach speed, which is $188.23 per 1,000 miles/

Table 21. Annual vehicle operating cost at uniform speeds, by alternative and year.

	Annual VOC at Uniform Speed, by Year		
Alternative	1990	2000	2010
At-grade Intersection	$2,398,000	$3,223,000	$4,332,000
SPUI	2,370,000	3,186,000	4,285,000

Table 22. Accident rates and annual number of accidents, by alternative and year.

	Total ADT(000)/ Total Lanes	Rates PDO	Injury	Fatal	No. of Accidents PDO	Injury	Fatal
At-Grade Intersection							
Year 1990	8.1399	0.4648	0.2145	0.00200	3.7834	1.7460	0.0163
2000	10.9408	"	"	"	5.0853	2.3468	0.0219
2010	14.7051	"	"	"	6.8349	3.1542	0.0294
Single-Point Urban Interchange							
Year 1990	6.7833	0.0879	0.0518	0.00148	0.5962	0.3514	0.0100
2000	9.1173	"	"	"	0.8014	0.4723	0.0135
2010	12.2543	"	"	"	1.0771	0.6348	0.0181

Table 23. Accident cost per accident, for intersections and interchanges, by accident severity.

	Cost/ Accident, Urban Area, by Severity		
Alternative	PDO	Injury	Fatal
Intersection	$1,380	$14,400	$986,000
Interchange	1,310	13,600	952,000

1,000 miles, or $0.18823 per mile. The results are given in Table 21.

Accident costs. The number of accidents is calculated using the equation and rates presented earlier, and the results are given in Table 22. Table 23 presents 1990 costs per accident for intersections and interchanges in urban areas, by severity.

Multiplying these accident costs by the number of accidents per year from the preceding table gives the annual accident costs given in Table 24.

For each of the two alternatives, the different types of user costs are added to obtain total annual motorist costs, as summarized in Table 25.

Salvage value. There is very little good information on the salvage value that should be used for highway projects such as highway interchanges. However, the Highway Performance Monitoring System has developed weighted average service lives to be used in calculation of life-cycle costs. These weighted average service lives range roughly from 40 to 70 years in length for different types of urban highway improvements (*39*). With straight-line depreciation, this would indicate that from 50 percent to 71 percent of the value of the facility would remain at the end of a 20-year analysis period. In this example, the bottom range of these values is used and it is assumed that the salvage value for an interchange at the end of a 20-year analysis period

Table 24. Number of accidents and annual accident costs, by alternative and year.

No. of Accidents Per Year: Year	PDO	Injury	Fatal	Accident Cost Per Year: PDO	Injury	Fatal	Total
At-Grade Intersection							
1990	3.7834	1.7460	0.0163	$ 5,000	$25,000	$16,000	$46,000
2000	5.0853	2.3468	0.0219	7,000	34,000	22,000	63,000
2010	6.8349	3.1542	0.0294	9,000	45,000	29,000	83,000
Single-Point Urban Interchange							
1990	0.5962	0.3514	0.0100	1,000	5,000	10,000	16,000
2000	0.8014	0.4723	0.0135	1,000	6,000	13,000	20,000
2010	1.0771	0.6348	0.0181	1,000	9,000	17,000	27,000

Table 25. Summary of total annual user costs by type of cost, by alternative and year.

	Annual Motorist Costs, by Year		
	1990	2000	2010
At-Grade Intersection			
Time Cost	$2,840,000	$5,289,000	$12,617,000
Vehicle Operating Costs:			
Idling	150,000	280,000	667,000
Stopping	754,000	1,082,000	1,893,000
Uniform Speed	2,398,000	3,223,000	4,332,000
Accident Costs	46,000	63,000	83,000
Total Costs	$6,188,000	$9,937,000	$19,592,000
Single-Point Urban Interchange			
Time Cost	$ 882,000	$1,327,000	$2,184,000
Vehicle Operating Costs:			
Idling	47,000	70,000	115,000
Stopping	325,000	450,000	623,000
Uniform Speed	2,370,000	3,186,000	4,285,000
Accident Costs	16,000	20,000	27,000
Total Costs	$3,640,000	$5,053,000	$7,234,000
Annual Benefits for Interchange	$2,548,000	$4,884,000	$12,358,000

is 50 percent of initial cost. This is discounted to the beginning of the analysis period to obtain the present worth of the salvage value.

Present Worth of Total Benefits. Given benefits for any years the benefits for a period can be estimated using a nomograph or formula given in the 1977 AASHTO Manual (*40*). Also, following this manual, which recommends a discount rate of 3 to 5 percent when costs and benefits are calculated in constant dollars (future costs and benefits are not inflated), a discount rate of 4 percent per year is chosen for use in this report. The AASHTO Manual uses the following formula for calculating benefits for n years, given the benefits for the first year and last year of the period of length n years.

$$f = (e^{(r-i)n} - 1)/(r - i)$$

where: f = the factor that is multiplied times the first year's benefits to estimate benefits for the entire period of n years; n = the length of time being considered; $r = \ln(\alpha)$, where ln indicates the natural logarithm and α is the ratio of the benefits in the last year being considered to the benefits in the first year considered; and i = the discount rate, which is assumed to be 0.04 (or 4 percent per year).

To estimate benefits for this example problem, this formula is used separately for the first and last 10 years. For the first 10 years, the following calculations are made:

B_1 = first year benefits = \$2,548,000
B_2 = second year benefits = \$4,884,000
n = 10 years
α = \$4,884,000 /\$2,548,000 = 1.9168
$r = \ln(\alpha)/10 = 0.065066$
$f = (e^{(0.065066 - 0.04)10} - 1)/(0.065066 - 0.04) = (1.284873 - 1.0)/ 0.025066 = 11.365$

Benefits for the first 10 years = $f * B_1$ = 11.365(\$2,548,000) = \$28,958,000.

For the second 10 years, the following calculations are made:

B_1 = first year benefits = \$4,884,000
B_2 = second year benefits = \$12,358,000
n = 10 years
α = \$12,358,000 / \$4,884,000 = 2.5303
$r = \ln(\alpha)/10 = 0.0928338$
$f = (e^{(0.0928338 - 0.04)10} - 1)/(0.0928338 - 0.04) = 13.1755$

Benefits for the second 10 years = $f*B_2$ = 13.1755 (\$4,884,000) = \$64,349,000. The total benefits are equal to \$28,958,000 plus \$64,349,000, or *\$93,307,000.*

Benefit/Cost Analysis

The benefit cost ratio is calculated using the following formula:

$$B/C = B/(IC + MC - SV)$$

where: B = present worth of benefits over the analysis period of 20 years, which were calculated as \$93,307,000; IC = initial capital cost for the interchange, which was estimated as \$16,304,000; MC = present worth of maintenance and operating costs over the analysis period, which is the increase in maintenance and operating costs for the interchange as compared to the intersection, or \$200,000 (\$640,000 − \$440,000) for the 20-year period, in present worth terms, as calculated previously; and SV = the present worth of the salvage value for the interchange net of any salvage value for the existing intersection; as discussed previously, this is calculated by assuming that the salvage value of the interchange at the end of 20 years is 50 percent of the initial cost (\$8,152,000), which is discounted to the present by dividing by $(1 + 0.04)^{20}$ (= 2.191123), to give a present worth of the salvage value of \$3,720,000.

Using these assumptions, the B/C ratio for the interchange is:

$$\begin{aligned} B/C &= (\$93{,}307{,}000)/(\$16{,}304{,}000 + \$200{,}000 - \$3{,}720{,}000) \\ &= (\$93{,}307{,}000)/(\$12{,}784{,}000) = 7.30 \end{aligned}$$

This calculation indicates that the SPUI will provide 7.30 times as much benefit as cost over the 20-year period. The net benefit for building the SPUI is \$93.3 million minus \$12.8 million, or \$80.5 million.

ALTERNATE DESIGN CONSIDERATIONS

During the conceptual planning stages of the project, the design team will most probably analyze other alternate designs. As an example, the SPUI is often compared to the tight urban diamond interchange (TUDI). Generally, there are several key discriminating factors that impact the cost effectiveness of various designs when compared to each other.

The functionality of the intersection within the context of the overall system plan immediately discriminates between the potential options available. The key discriminating factor is traffic capacity. Capacity almost automatically specifies the general category of intersection a design team would consider for a given intersection. As an example, if capacity and the system plan dictate a freeway-to-freeway interchange, a cloverleaf or directional interchange is probably the only choice. The SPUI is not an option in this scenario. The same scenario can be applied to a SPUI compared to an AGI. Unless the AGI is fully built out, upgrading an existing AGI would likely be the best choice. As a consequence of functionality, the comparison of the SPUI with the TUDI or, perhaps, a parclo is really the only comparison of interest.

This section provides a qualitative assessment of critical discriminators between the SPUI and two alternate designs, the TUDI and parclo. This assessment portrays the impact these discriminators might have on cost effectiveness. A matrix has been developed to portray the qualitative assessment and is shown in Table 26. The left column reflects key selection criteria that influence the cost effectiveness of a project. Two types of feasible alternate designs are listed across the top (TUDI and parclo). The base case design for comparative purposes is the SPUI presented in the case study discussed in previous sections (see Figures 93 and 94). The intersection between the selection criteria and the other feasible alternate designs reflects a change in project or user cost to the base case SPUI when comparing it to the other two design choices.

In Table 26, the base case SPUI and the other alternate designs are assumed to have similar traffic handling capacity. Given that each design has a similar traffic handling capacity, the

Table 26. Qualitative cost comparisons between SPUI and other feasible design alternatives.

Selection Criteria	Feasible Design Alternatives: Tight Urban Diamond Interchange	Parclo
Capacity Handling	Similar*	Similar*
Right of Way	More	More
Bridge Structure Costs	Less	Similar
Earthwork	Less	More
Retaining Walls	Less	Less
Drainage	Similar	More
Pavement	Similar/More	More
Operating And Maintenance Costs	Less	Similar
Work Zone Traffic Control	Less	Similar/More
Signalization	Similar/More	More
Striping	Less	Similar
User Costs	Similar/More	More

*All designs are assumed to have similar capacity.

intersection between selection criteria and feasible design alternatives represents a directional change in cost of the TUDI or parclo compared to the SPUI. For example, given that the traffic capacity is the same for all three designs, the cost of right-of-way will be more for both the TUDI and the parclo when compared to the SPUI. The intent of Table 26 is to provide the designer with indicators of the potential impact of various selection criteria on cost. It is offered as a guide to the designer for gaining a better understanding of key differences between designs as related to cost. However, the impact must be quantified to verify true differences given specific designs and site conditions.

The two most significant discriminators between the SPUI and a comparable TUDI are right-of-way and bridge cost. SPUIs typically take less right-of-way area than the TUDI. Thus, if new right-of-way must be acquired, in a comparative analysis such as presented in this chapter, the net impact would be an increase in overall project costs for the TUDI, as shown in Table 26. The magnitude of the difference would depend on the additional incremental area of right-of-way taken and its associated cost. This is a very site-specific problem.

Conversely, bridge costs for the SPUI would increase in relation to the TUDI. Because of an increase in the vertical alignment of the SPUI bridge, other related costs will increase such as earthwork and perhaps retaining walls. Correspondingly, the cost for the TUDI bridge will be less. An incremental increase in SPUI construction cost must be offset by a decrease in right-of-way costs. If this incremental cost delta between the SPUI and TUDI is small, the SPUI must significantly reduce user cost to produce a sufficient benefit to make the SPUI economically attractive compared to the TUDI. As shown in Table 26, TUDIs typically have higher user costs.

Because right-of-way and bridge design are critical discriminators, careful analysis of these elements would be requisite for the specific location of the project. An analysis of user costs as derived from traffic capacity evaluations for both designs also would be critical. Thus, the approach to the design of operations and geometric features of the interchange will determine the potential differential in user costs. Finally, incremental benefits due to a reduction in user costs compared to incremental project investments will determine which design solution is economically the most attractive.

Although the parclo may be selected for other reasons, it may be best suited for the situation where only two of four quadrants have right-of-way available for developing an interchange. This can occur when access on two adjacent quadrants is constrained by either buildings or railroad tracks. Thus, the parclo is more often selected when it fits this unique site situation. Table 26 illustrates the qualitative impact that selection criteria have on cost when comparing a parclo to the SPUI.

CHAPTER EIGHT

CONCLUSIONS AND RECOMMENDATIONS

GENERAL CONCLUSIONS

1. This research was one of the first comprehensive studies of the Single Point Urban Interchange (SPUI) conducted in the United States. Consequently many of the study findings and conclusions are statements of discovery about various aspects of SPUIs. These results cover historical development, geometric and bridge design, traffic operations, and general applications. The study findings are based on the field survey of 36 SPUIs visited in the summer of 1989, subsequent capacity studies conducted in 1990, and a synthesis of the literature and related experiences gathered by the research staff to date.

2. In general, the SPUI interchange was found to be a safe and efficient interchange that probably cost somewhat more than a TUDI to construct. The SPUI is being widely implemented throughout the country with numerous local studies of its effectiveness underway to better quantify its features.

SPECIFIC CONCLUSIONS

Historical

1. The first SPUI built in the U.S. was completed on February 24, 1974, in Clearwater, Florida, by the Florida Department of Transportation. The primary planning and engineering for the project was provided by Greiner Engineering of Tampa, Florida.
2. The second SPUI built was completed on September 9, 1975, in Moline, Illinois, by the Illinois Department of Transportation. The preliminary engineering for the project was provided by DeLeuw, Cather & Company of Chicago, Illinois. Both of the pioneer interchanges are still operating today and providing good traffic service.
3. There are perhaps 50 SPUIs operational today in the United States in at least 18 states with many others under construction. As a result, engineering expertise with SPUIs is rapidly growing at many of the agencies and firms.
4. SPUIs have been built in several other countries, including Germany, Greece, and Canada by other organizations.

Applications

1. Two design types of SPUIs exist. The basic and more prevalent type is the SPUI3, which does not have frontage roads. The secondary type is the overpass SPUI4, which has been used while upgrading some arterials to expressway standards. This design provides continuous one-way frontage roads that are conveniently used as detour routes during bridge construction, and then leaves them in to provide local access to adjacent property in the final design. Alabama, Georgia, and Florida have used the SPUI4 in this manner.
2. A major application for SPUI3s (three-phase with no frontage roads) has been for congestion relief whereby the new SPUI3 provides up to twice the available traffic handling capacity as the congested at-grade signalized intersection it normally replaced, usually at the intersection of two principal urban arterials.
3. Other important SPUI applications have included capacity upgrades of outdated signalized interchanges, general usage with new freeway construction, and reconstruction of urban arterials along a corridor to expressway standards.
4. The SPUI is basically a two-level diamond interchange having a single signal for serving all four inverted left turns. From an evolutionary development viewpoint, however, it is best thought of as evolving from a congested high-type single signalized intersection where the mainline through-roadway has been transformed into a median flyover and where all mainline approach turns have been given a single right-hand-side exit while remaining anchored to their original single signal location and inverted left-turn operation.
5. A well-designed SPUI is a safe and effective high-capacity two-level signalized interchange. Its safety record suggests that it is as safe as a competitive tight urban diamond interchange (TUDI), and some evidence indicates that it may be safer. The SPUI3 appears to be as adaptable to variable traffic patterns as is a TUDI.
6. A well-designed SPUI contains adequate capacity to handle the projected traffic demands, reasonable design speeds for the turning roadways, adequate sight distances, an expressway level signing system, and good illumination for nighttime conditions.
7. The SPUI and the TUDI are viable design options for most types of capacity improvement projects for congested urban intersections and older urban diamond interchanges. A 6-2-2-2 SPUI3 should be the first overpass SPUI option considered, i.e., the cross street is a six-lane divided facility having dual left turn lanes and the off ramps have dual left turn lanes and dual right turn lanes.
8. The SPUI3's single traffic signal located at the center of the interchange, instead of having two signals each offset perhaps 150 ft along the cross street toward other nearby signals, can be an important aspect favoring the SPUI over a TUDI in some restricted arterial situations. However, sufficient distance is still needed to the next downstream signal along the cross arterial to provide adequate weaving distance for the off-ramp traffic to safely maneuver into the downstream left turn bay. A separation distance exceeding 1,200 ft from the merge point to the downstream signal appears desirable.

Design Attributes

1. There are two interchange bridging options — the mainline overpass and underpass. The overpass design is the more prevalent type. Many important design parameters depend heavily on the type of grade separation with bridge design being the principal one.
2. Two types of mainline overpass bridges are common. Typical design parameters include: (a) a single-span bridge about 220 ft long with a depth of about 9 ft, or (b) a three-span bridge about 400 ft long with a center span of 180 ft and a depth of about 7 ft. Steel plate girder, steel trapezoidal box girder, and concrete cast-in-place box girder designs are used in overpass bridge construction. Selection of bridge type depends heavily on local conditions.
3. One type of mainline underpass bridge is common: a two-span bridge about 140 ft long having twin 70-ft spans about 3 ft deep supported on a mainline median pier. Platform deck girder bridges with a flared deck on each end to serve the left turning roadways are commonly used. Complex, hourglass-shaped, cast-in-place concrete bridges have been used in some aesthetically sensitive urban areas.
4. Dual left turning lanes are generously applied. The percentages of SPUIs in the field survey that used dual left turning lanes on both approach legs of the off ramps and cross street for overpass and underpass designs were as follows:

SPUI	Overpass(%)	Underpass(%)
Off-ramps	67	67
Cross street	56	22

5. Nominal left-turn paths have an average radius of about 200 ft. The average left-turn radius, in feet, in the field survey for various types of SPUIs was as follows:

SPUI	Overpass(ft)	Underpass(ft)
Off-ramps	210	296
Cross street	204	195

6. Generous application of sight distance along the turning ramps using the latest design concepts presented in the AASHTO Green Book provide the best operating environment. Multispan (three or more spans) bridges offer more sight distance and visibility for a given center clear span with overpass designs.

Operations

1. The 200-ft nominal left turn radius at SPUIs promotes a high quality of turning operations. Triple trailer trucks can make the left turn, two abreast from a stop following green onset, without any noticeable effect on left-turn saturation flow or operational safety beyond those expected for single-lane operations.

2. The average left-turn saturation flow, S_{lt}, in passenger cars per hour green per lane, pcphgpl, for nominal, left turn lane widths can be estimated by the following equation, as a function of its average turning radius, R, in feet—$S_{lt} = 3{,}600 / (1.50 + 1.11/R^{0.245})$ pcphgpl—with a maximum value of 2,000 pcphgpl, occurring for radii exceeding about 200 ft. Saturation flow was observed to consistently increase with increasing left-turn radius up to this level. Saturation flow for 12-ft wide through lanes is about 2,000 pcphgpl for all interchange types.

3. The 95 percentile free turning speed in miles per hour, mph, of left turning vehicles on green may be estimated from the related average left turn radius, R, in feet as: $V95 = 4.53\, R^{0.357}$ mph. These estimated turning speeds at SPUIs are slightly higher than those expected from the AASHTO design criteria for nearly level cross sections and suggest that drivers may tolerate higher side forces and travel faster than the design speed of a turn lane.

4. The dominant traffic signal control observed was isolated traffic actuated operation. Conventional three-phase control was used for SPUI3 (no frontage roads) and variations of four-phase control was used for SPUI4 (frontage roads). Dual left turns leading was the dominant arterial phasing sequence used, although lead-lag phasing was observed.

5. The large open central control area of the SPUI impacts traffic operations to some extent. The following operations were noted: (a) A slight hesitancy sometimes occurs by through drivers not promptly moving into the intersection following green onset. (b) Reduced operational quality occurs when high-speed off-ramp vehicles are tracking a broken back left turn in the central control area, i.e., curves having a central left-turn radius of the compound curve exceeding 300 ft, when used with overpass designs. (c) Drivers sometimes have difficulty finding the on-ramp left-turn slots if the outbound roadway is not readily visible, especially for underpass designs. The ideal cross-road alignment for underpasses would have a slight sag vertical curve at the middle of the SPUI with slightly elevated ramp connections, i.e., a swale effect with corner drainage. (d) Some undesirable left-turn operations are being made by cross street motorists traveling through the SPUI. The incorrect maneuver typically begins by drivers not making the advance left turn bay maneuver, then trying to complete the left-turn maneuver from the median through lane during the latter portions of the through phase either in advance, over, or beyond the center island. Similar left turn bypass maneuvers have been observed with SPUI4 frontage road operations.

6. The wide open central control area of a SPUI does not permit TUDI-like phase overlaps. Thus, larger red clearances, on the order of 3.4 sec per phase, and related lost times per phase typically result. Bigger central intersection control areas result in larger lost times and corresponding reductions in average phase capacity per lane.

7. The average phase capacity per lane of a SPUI3 is probably higher than a TUDI when the average arterial stop-line to stop-line separation distance is 265 ft or less. Typical SPUI3 spacings range from 190 to 260 ft for six-lane overpasses, depending on the width of the mainline roadway, design speed of the turning ramps, and the center span of the bridge. Multispan bridges used with overpasses and underpass designs, in general, will have slightly shorter stop-line to stop-line separation distances because sight distance obstructions are less in the central control area.

8. Off-ramp right-turn operations require special design attention, especially where mainline off-ramp volumes are high and traffic patterns are nearly balanced. Off-ramp queueing of right turns can be a problem if the queueing overlaps and interacts with the adjacent left turns. Likewise, downstream right-turn operations may not have time to weave across a wide arterial into a left turn bay unless the distance to the downstream intersection is sufficient. While this general downstream access problem exists for all interchange forms, it is ironically exacerbated by output flow from the SPUI3's efficient three-phase signal operation.

9. The three-phase signal operation used at a SPUI3 does not have a protected cross street pedestrian phase. Consequently, almost no operating agency provides marked crosswalks here to suggest or encourage pedestrian crossings at the interchange parallel to the mainline highway. Crossing at the next downstream intersection is implied.

10. High-quality illumination exceeding 1.0 ft-c is desired under overpass SPUIs. Uniform light distribution is highly desired. Use of wall-pack units on overpass bridge abutments to provide some of this illumination typically produces excessive glare and is deemed undesirable for this purpose.

11. The novel use of airport runway lights, or variants therefrom, as pavement markers does not appear to be cost effective because of their high initial cost, potentially complex maintenance problems, and a lack of demonstrated effectiveness above conventional pavement marker systems used with nominal SPUI designs. Sand and other debris often block the recessed light sources. Ironically, street sweeper operations contribute to this problem rather than solve it. A redesigned lampholder assembly that has no recessed outer surfaces would improve its effectiveness at SPUIs.

Economic Aspects

1. SPUIs cost more than TUDIs to construct. Generally, the longer clear spans for SPUIs require greater bridge depth that results in additional earthwork, more extensive retaining walls and longer vertical realignment of the intersecting roadways. The right-of-way cost is generally less for SPUIs because of its geometric features and adaptability.

2. The validity of economic analysis may be questioned unless due care is taken to ensure that equivalent operational conditions are being compared between alternatives, or unless the incremental benefits provided by the added investment are appropriately credited to the design option. Because the SPUI will likely cost

more than the TUDI but may provide higher benefits, an incremental benefit-cost analysis is suggested.

RECOMMENDATIONS

A series of general application guidelines are recommended regarding SPUI use based on the results of this research. These guidelines address basic planning and design issues. A summary of SPUI location guidelines concludes the planning section. More specific application guidelines have already been provided in Chapters Four to Seven of this report.

Planning Guidelines

1. The SPUI3 and TUDI should be considered as viable competitors to provide major added capacity needed for congestion relief in growing urban areas at signalized at-grade intersections that have been fully expanded to their practical maximum size. Typical benefit-to-cost ratios exceeding 6:1 should be expected for a needed project using either alternative.
2. The SPUI3 and TUDI should be considered as viable competitors for all two-level diamond-type interchange projects on all new urban freeways without frontage roads.
3. A SPUI3 should be given special consideration when left-turning volumes exceeding 600 vph are expected, or when significant large truck operations are expected from off ramps having left-turn volumes exceeding 300 vph.
4. A SPUI4 and a TUDI should be considered as viable competitors when upgrading a high-volume urban arterial along a corridor to expressway standards. Further studies would be needed to justify expanding the scope of a SPUI4 to freeway applications.
5. A multidisciplinary team should be formed early to ensure that the preliminary design for a SPUI follows a systems approach that considers all pertinent planning, design, construction, and operations aspects identified in this report. Trade-offs between bridge length and traffic operations impacts should be carefully assessed before any final design decisions are made. Proposals to shorten bridge length for economic reasons to lengths that produce broken-back left-turn lanes or unreasonably low design speeds for the turning lanes can result in operationally deficient sight distances and reduced safety and, therefore, should be avoided where feasible.
6. The cost-effectiveness methodology in Chapter Seven, or a more refined method, should be used to guide the interchange selection process. All relevant life cycle costs should be included. Special care should be given to the estimation of the principal cost components in SPUI design to include: (a) right-of-way availability and acquisition cost; (b) bridge design options, fabrication and construction issues; (c) highway realignment requirements and impacts; (d) drainage and utility relocation needs; and (e) traffic control during construction.
7. The following is a summary of general application guidelines for use when considering whether to select a SPUI at a particular site. (a) SPUIs may be a good interchange candidate at sites having: restricted right-of-way, high volumes with major congestion, high left turn volumes, high volumes of large trucks, and/or high accident locations. (b) SPUIs may not be a good interchange candidate at sites having: severe skew angle, wide overcrossing roadway, adverse grade conditions on cross road, moderate to high pedestrian volumes across the cross road, and/or high through volumes and low turning volumes on the cross road.

Design Guidelines

1. Use the highest design standards possible. In SPUI design, every effort should be made to use design standards that exceed minimum levels at all times. As a design goal, desirable standards should be selected.
2. All SPUIs should be designed as if they were major interchanges and not expanded intersections, per se. Guide signing and roadway lighting should be upgraded to levels approaching freeway interchange standards.
3. Use dual left turn lanes on all approach legs of the SPUI.
4. For overpass SPUI3 designs, the first design alternative considered should be a multispan bridge having a 6-2-2-2 configuration with the bridge superstructure being readily expandable by at least two mainline through lanes at some future date without having to widen the abutments. The 6-2-2-2 SPUI3 configuration would have a six-lane (6) divided urban arterial cross section at grade to the SPUI with dual turn lanes for both the arterial (2) and off-ramp (2) left-turn movements, and dual right turn lanes for both (2) off-ramps. A single deceleration lane for right-turn operations from the cross street is acceptable even for relatively high volumes. However, an acceleration lane for off-ramp traffic onto the cross arterial is not necessarily recommended unless sufficient distance (greater than 1,200 ft) is available to the next downstream intersection. Direct entry merging for this maneuver provides good operation in restricted designs.
5. Generous storage length should be provided for left and right turn queueing at the off-ramp junction to the cross street. The queues of these turning operations should not be permitted to interact. Significant rear-end accidents may otherwise occur.
6. Signalizing the off-ramp right-turn operations should be avoided. Delayed-call right-turn queue detection should be provided for high-volume conditions having fairly balanced traffic patterns. Right-turn volumes from the off-ramp exceeding the complementary cross street volume by 100 vehicles per hour per lane, vphpl, should warrant this detector treatment when the right-turn volume exceeds 300 vphpl.
7. Pedestrians should be encouraged to cross the street at the first downstream signalized intersection at SPUI3s.
8. The option of using two horizontally mounted traffic signals should be considered, in the central area underneath the bridge of overpass SPUIs for off-ramp left turns, in place of vertically mounted signal clusters now often used over the central island. Horizontally mounted signals are commonly used in some parts of the country in urban conditions.
9. A high-quality lighting system should be provided for the SPUI, both along the roadways, and underneath any overpassing bridge structure. High-glare wall-pack lighting units should not be attached to any vertical wall of an overpass bridge abutment.

Economic Analysis

1. Some reliable form of economic analysis should be used to guide the design option formulations and the selection of the

most cost-effective design. The procedures herein are recommended for evaluating agency and user costs, and for selecting the most cost-effective design alternative.

2. As many proposed applications will be for capacity improvements of congested urban intersections, several types of grade separations would be expected to provide attractive benefit-to-cost ratios, on the order of 6:1 or greater, because of the large delay savings that would accrue over a 20-year period with nominal traffic growth rates. The analyst should not be misled by the initial attractiveness that any one design might offer until other design options are evaluated. During these studies, projected hourly traffic volumes on any movement should not be used that exceed the phase capacity by more than 20 percent because unreasonable delays may be predicted by the computer models.

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