FHWA Design Discipline Support Tool

Interchange Design (New Construction and Reconstruction)
Prompt-List for Assessing Key Geometric Features

Project: ____

Location: ____

Description: ____

Assessment made by: ________________________ Date of Assessment __________

1.0 Design Standards

1.1 What design standards are applicable to the project?

_____ AASHTO publication "A Policy on Design Standards Interstate System" - applicable for all projects on the Interstate System

_____ AASHTO publication "A Policy on Geometric Design of Highways and Streets" (commonly called the Green Book) – applicable to all freeways on the NHS including the Interstate System

_____ Additional standards of State DOT for geometric design, standard drawings, and standard specifications that meet or exceed the FHWA's adopted standards

1.2 Any design exceptions to the "13 controlling criteria" proposed? __________

2.0 Interchange and Ramp Spacing

2.1 Does the interchange spacing (based on crossroad to crossroad spacing) exceed 1 mi? __________

2.2 Does the spacing between successive entrances and exits meet or exceed AASHTO criteria? __________

3.0 Approach Alignment to Interchange

3.1 Is the grade of the freeway relatively flat through the interchange area? __________

3.2 Is the horizontal alignment of the freeway relatively straight through the interchange area? ___________

3.3 Is adequate sight distance (desirably decision sight distance) provided in advance of each exit? __________

4.0 Interchange Configurations

4.1 Was an appropriate array of interchange configurations and variations evaluated in the design study phase?
4.2 Is the selected interchange configuration appropriate for the operational needs, fits the topography and potential site conditions and constraints, and is consistent in exit pattern with other nearby interchanges?

4.3 Are all directional traffic movements provided for? __________

4.4 Are all the exits and entrances on the right side of the freeway mainline? ____________

4.5 Is there a weaving section within the interchange as proposed? _________

If YES, what is the distance between the physical merge and exit nose? ____________

4.6 Is the interchange configured with the crossroad over the freeway? ___________

4.7 Route Continuity - Is the interchange configured so that the priority route is the through facility? __________

5.0 Ramp Design

5.1 Is the design speed of the ramp proper at least 50 percent of the mainline design speed? __________

5.2 Is sufficient length for acceleration at entrance ramps provided? ____________

5.3 Is sufficient length for deceleration at exit ramps provided? ____________

5.4 Are the exit and entrance lanes balanced? _________

6.0 Signing

6.1 Is the proposed signing in accordance with the MUTCD and suggested limits on message units? _________

6.2 Evaluate the proposed signing from a driver's point of view. Assess the risk of driver confusion and strategies to simplify the signing.

7.0 Crossroad Design

7.1 Are sidewalks and bicyclist facilities provided along the interchange crossroad? Pedestrians and bicyclists are particularly vulnerable to high speed approach vehicles turning at ramp terminals. Are the crossings at interchange ramps controlled or uncontrolled?

7.2 Is sufficient control of access along the crossroad beyond the interchange being provided to ensure its integrity? (The AASHTO standard of a minimum of 100 ft in urban areas and 300 ft in rural areas is usually insufficient where additional development is likely).

7.3 Are adequate land development and access management measures in place for the interchange area?

7.4 Ensure elements of the ramp/crossroad intersection are properly designed, especially with regard to:

- turning radii for design vehicle
- capacity
- traffic control
- channelization
- intersection sight distance

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Interchange Design (New Construction and Reconstruction)
Supplemental Information to Prompt-List for Assessing Key Geometric Features

1.0 Design Standards

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Section 109(c) of Title 23 U.S.C. establishes standards for the design and construction of all projects on the National Highway System (NHS), including the Interstate System. These standards are applicable to any proposed improvement regardless of the funding source. Deviations from the standards must have approved design exceptions. FHWA has adopted the AASHTO publication “A Policy on Design Standards Interstate System” for all projects on the Interstate System, regardless of the funding for the proposed project. The Interstate Standards are not intended to be a “stand alone” document for all of the geometric design standards that are used in the development of projects on the Interstate System. Other publications, such as “A Policy on Geometric Design of Highways and Streets” (commonly called the Green Book) and the “Standard Specifications for Highway Bridges” are referenced in the Interstate Standards and used for all geometric design issues not specifically addressed in the Interstate Standards. Chapter 10 of the Green Book provides detailed information on the design concepts and standards that should be met as part constructed new or improved interchanges. Many state DOTs have also developed additional standards for geometric design, standard drawings, and standard specifications that meet or exceed the FHWA’s adopted standards.

1.2 Any design exceptions to the “13 controlling criteria” proposed? __________

The 23 CFR 625 provides that exceptions may be given on a project basis to designs which do not conform to the minimum criteria set forth in the standards, policies, and standard specifications for experimental features on projects and projects where conditions warrant that exceptions be made.

The FHWA has identified “*13 controlling criteria*” that require formal written approval if an exception from the standard is justified. These criteria are design speed, lane width, shoulder width, bridge width, horizontal alignment, superelevation, vertical alignment, grade, stopping sight distance, cross slope, vertical clearance, lateral offset to obstruction (formerly known as horizontal clearance), and structural capacity. A formal written design exception is required if design criteria on the NHS are not met for any of these 13 criteria. Divisions and States may supplement their design exception review procedures to include additional design elements and have exceptions to those additional elements handled by the same review and approval process, however, the 13 controlling criteria reflect the minimum FHWA requirements for formal written design exceptions on the NHS regardless of project funding.

1.2.1 Design Speed

Design speed is a selected value used to determine the various physical design elements and geometric features of the roadway. Design speed has a significant effect on the operation and safety of a highway because it is used to determine various individual design elements with specific dimensions such as stopping sight distance or horizontal curvature. The selected design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of the highway. The chosen design speed should equal or exceed the posted or regulatory speed in order to ensure that drivers operating at the legal speed limit can do so without exceeding the safe design speed of the highway. A design exception to "design speed" is rare because it is really an exception to individual physical design elements and accordingly should be justified on that basis.

The design standards for the Interstate System state that a minimum design speed of 70 mph should be used for rural areas, and where terrain is mountainous a design speed from 50 to 60 mph may be used. In urban areas, the design speed shall be at least 50 mph. Design speed is also applicable to the ramps within an interchange.

<u>1.2.2 Lane Width</u>
The design standards for the Interstate System state that all traffic lanes shall be at least 12 feet wide. In addition to the primary through travel lanes, the criteria also apply to lane widths for auxiliary lanes such as climbing lanes. There are also widths for special-purpose lanes such as on interchange ramps.

Pavement markings which delineate lane lines on many highways may line up with longitudinal pavement joints, but do not always. For instance, the width of PC pavement is sometimes constructed wider than the lane widths to reduce stress at the pavement edge caused by heavy vehicles. However, the portion of the PC pavement outside of the painted lane line is considered part of the shoulder width, not the lane width. By definition, lane width is only the portion of the lane designed for use by vehicles traveling in the longitudinal direction and does not include shoulders, curbs, or on-street parking areas.

Lane width has an influence on the safety and comfort of the driver. As speed and volumes increase, adequate lane width is important to accommodate the variations in lateral placement of the vehicle within the lane. Adequate lane width is very important along horizontal curves where vehicles may tend to off-track and encroach into adjacent travel lanes. Lane width also has an impact on operations. When determining highway capacity, adjustments are made to reflect the effect of constricted cross sections on free-flow speeds. Lane widths less than 12 feet reduce travel speeds on high-speed roadways. Widths greater than 12 feet are not considered to increase speeds above the base level. The Highway Capacity Manual methodology for freeways and multi-lane highways reduces the estimated free flow speed for 11 or 10 foot wide lanes by 1.9 and 6.6 mph, respectively.

<u>1.2.3 Shoulder Width</u>
The design standards for the Interstate System state that the paved width of the right shoulder shall not be less than 10 feet and the paved width of the left shoulder shall be at least 4 feet on a four-lane section. On sections with six or more lanes, a 10 ft paved width for the left shoulder should be provided.

The adopted criteria for Interstates states that where truck traffic exceeds 250 DDHV, paved shoulder widths of 12 feet should be considered. A point of clarification is appropriate regarding the language “should be provided” and “should be considered” found in the AASHTO *Policy on Design Standards Interstate System.* All the shoulder widths mentioned become standards for the Interstate System by virtue of their adoption by FHWA and they are the minimum values for each condition described. Therefore, a project designed for the Interstate System which does not provide the applicable shoulder widths for the conditions mentioned in the AASHTO *Policy on Design Standards Interstate System* would require a design exception.

In situations where cross-sectional width is constrained, evaluating how that width can most effectively be distributed between the lane and shoulder should be evaluated. This evaluation is basically a consideration of trade-offs—taking some of the lane width to use for additional shoulder width or vice versa, depending on the location and the objectives. The optimal distribution will depend on site-specific characteristics. For example, on a rural two-lane roadway with no shoulders and a history of run-off-road crashes, an effective strategy may be to distribute some of the available width to accommodate a narrow paved shoulder and rumble strips, at the expense of narrower lanes. The objective would be to reduce the probability of run-off-road crashes. For a multilane highway with heavy truck volumes and a curvilinear alignment, maintaining full 12-foot lanes at the expense of some of the shoulder width may be a more optimal design. The objective would be minimizing truck off-tracking into adjacent lanes. The key is to look at the site specific characteristics such as highway type, traffic and truck volumes, geometry, crash history, and crash type. With this information various combinations of lane and shoulder width can be evaluated with the goal of optimizing safety and traffic operations at the design exception location.

Where shoulder width is limited, a possible mitigation strategy is to provide periodic “pull-off” areas in locations where additional space is available. Pull-off areas provide several advantages: 1) room to store disabled vehicles, particularly important for maintaining operations on high-volume freeways; and 2) they provide an area for law enforcement to pull over vehicles in areas with narrow shoulders. This increases safety for law enforcement personnel, the stopped driver, and passing motorists.

1.2.4 Bridge Width

Bridge width is the total width of all lanes and shoulders on the bridge, measured between the points on the bridge rail, curb, or other vertical elements that project the furthest onto the roadway. The design standards for the Interstate System state that the width of all bridges, measured between rails, parapets, or barriers shall equal the full paved width of the approach roadways. The approach roadway includes the width of paved shoulders. Long bridges (defined as having an overall length in excess of 200 ft) may have a lesser width and shall be analyzed individually. On long bridges, offsets to parapet, rail or barrier shall be at least 4 ft measured from the edge of the nearest traffic lane on both the left and the right.

1.2.5 Horizontal Alignment

The AASHTO *Policy on Design Standards Interstate System* states that curvature, stopping sight distance, superelevation, and allied features such as transition curves shall be correlated with the design speed in accordance with the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets*. In terms of the 13 controlling criteria, horizontal alignment refers only to the horizontal curvature of the roadway—the minimum radius for the selected design speed, which is determined from the maximum rate of superelevation and maximum allowable side friction factor. Other elements of horizontal alignment such as stopping sight distance and superelevation have separate criteria.

1.2.6 Superelevation

Maximum superelevation is affected by several variables such as climate, terrain, highway location (urban vs. rural), and frequency of very slow-moving vehicles. For example, northern states that experience ice and snow conditions may establish lower maximums for superelevation than states that do not experience these conditions. Due to these region specific variables that affect the rate of superelevation, State policy establishes maximum superelevation rates on the NHS within the ranges provided in the AASHTO Policy.

A point of clarification is that formal design exceptions are not required for superelevation transition lengths.

1.2.7 Vertical Alignment

In regard to the 13 controlling criteria, vertical alignment refers only to the vertical curvature of the roadway. Other elements of vertical alignment such as stopping sight distance and grade have separate criteria. In addition to stopping sight distance, vertical curvature is influenced by drainage, passenger comfort, and appearance.

1.2.8 Grade

The design standards for the Interstate System establish maximum grades as a function of the design speed and the type of terrain ranging from 3% to 6%. Grade affects vehicle speed and vehicle control, particularly for large trucks. A design exception is required if the maximum grade is exceeded. Minimum grades to achieve proper drainage are also provided and a design exception is required for highway segments that are flatter than the minimum grade.

1.2.9 Stopping Sight Distance

Stopping sight distance is required at all locations along the roadway, including horizontal and vertical curves. For horizontal curves this includes a sufficient horizontal sightline offset to an obstruction. For vertical stopping sight distance, this includes sight distance at crest vertical curves, headlight sight distance at sag vertical curves and sight distance at under crossings. On crest vertical curves the roadway itself limits the driver's sight distance. Sag vertical curves provide greater sight distance during daylight conditions, but severe sag vertical curves will limit the effective distance of the vehicle's headlights at night. Where lighting is provided on the roadway, a design to the driver comfort criteria may be adequate. The length of sag vertical curves to satisfy the comfort criteria over the typical design speed range results in minimum curve lengths about half of those based on headlight criteria.

Decision sight distance provides additional reaction time for more complex maneuvers that require speed, path or direction change, such as merging at a lane drop. It is desirable to provide decision sight distance at critical locations, but a formal design exception is not required for this criterion.

1.2.10 Cross Slope

Cross slope is an important design element because it drains water from the roadway laterally and helps prevent ponding of water on the pavement. Cross slopes that are too steep, however, can cause vehicles to drift, laterally skid when braking, and become unstable when crossing over the crown to change lanes. These conditions are exacerbated by icy, snowy, or windy conditions.

The design standards for the Interstate System states that on tangent sections the pavement cross slope shall be a minimum of 1.5 percent and desirably two percent. In areas of intense rainfall, the cross slope may be increased to 2.5 percent. Paved shoulders should have a cross slope in the range of two to six percent but not less than the cross slope of the adjacent pavement.

In addition to the cross slope of the lanes, the cross-slope break between the lane and shoulder on the high side of superelevated curves should not exceed 8%. A formal design exception is required when this condition cannot be met.

1.2.11 Vertical Clearance

The adopted standard for vertical clearance on all rural sections of the Interstate System is that the clear height of structures shall be not less than 16 ft over the entire roadway width, including the width of paved shoulder. In urban areas, 16 ft of clearance shall apply to at least a single interconnected interstate routing. On other interstate urban routes, the clear height shall be not less than 14 ft. An allowance should be made for future resurfacing to maintain the integrity of vertical clearance for national defense purposes. A design exception is required if this standard is not met. Exceptions for vertical clearance on the Interstate must also be coordinated with the Military Surface Development and Distribution Command Transportation Engineering Agency (SDDCTEA) of the Department of Defense.

1.2.12 Horizontal Clearance (Lateral Offset to Obstruction)

The lateral offset distance is defined from the edge of traveled way, shoulder, or other designated point to a vertical roadside element. Some examples of these elements include walls, barriers, bridge piers, sign and signal supports, trees, and utility poles. Lateral offset can be thought of as an operational offset—vertical roadside elements offset to the extent that they do not affect a driver's speed or lane position. Adequate clearance should be provided for mirrors on trucks and buses and for opening curbside doors where on-street parking is provided.

Lateral offset should not be confused with the clear zone—a clear recovery area, free of rigid obstacles and steep slopes, which allows vehicles that have run off the road to safely recover or come to a stop. While lateral offset can be thought of as an operational offset, the clear zone serves a safety function. Lateral offset to obstructions is one of the 13 controlling criteria that require a design exception. Clear zone is not. The AASHTO *Roadside Design Guide* provides ranges for clear zone based on speed, traffic, and roadside slopes. The Guide states that "the values suggest only the approximate center of a range to be considered and not a precise distance to be held as absolute." Designers need to exercise judgment in selecting an appropriate clear zone, taking into account the variables listed above as well as the location (urban vs. rural), the type of construction (new construction/reconstruction/3R), and the context. Chapter 10 of the *Roadside Design Guide* provides guidance on roadside safety in urban and restricted environments and emphasizes the need to look at each location and its particular site characteristics individually. Even though clear zone is not one of the controlling criteria that requires a design exception if not met, its importance should still be recognized. Even though it is variable and dependent on many site specific issues, a clear zone should be established for projects or project segments. Once a clear zone has been established, decisions to deviate from it for particular roadside obstacles should be documented.

1.2.13 Structural Capacity

With regard to design exceptions, structural capacity refers to the load-carrying capacity of the bridge. Although identified as one of the controlling criteria, structural capacity is typically not thought of as an element of geometric design. The adopted standard for the Interstate System is that all new bridges have at least an MS 18 (HS 20) structural capacity.

The bridge rail (i.e. type or condition) is not part of the 13 controlling criteria. However, bridge rail is an important safety consideration and should be structurally sound and meet current crash test standards. Updating substandard barrier is an important safety improvement and should be included as part of a project if needed.

2.0 Interchange and Ramp Spacing

2.1 Does the interchange spacing (based on crossroad to crossroad spacing) exceed 1 mi? __________ If no, consider the following improvement alternatives: • Provide collector-distributor roads • Add auxiliary lanes at one or more entrance ramps to facilitate merging.

Interchange and ramp spacing are related terms, but not synonymous. Interchange spacing is a distance measured along the freeway between the centerlines of the intersecting crossroads. Ramp spacing values are a byproduct of individual ramp design and operational requirements. Both are very important considerations in the planning and design of new or modified interchanges.

In urban areas, a rule-of-thumb is that there should be a one-mile minimum spacing between interchanges to allow for the ability to provide proper advance guide signing and to provide sufficient space for entrance and exit maneuvers. Closer spacing may be allowed, but might necessitate the use of collector-distributor roads or the "braiding" (grade-separation) of ramps to facilitate smooth traffic flow.

In rural undeveloped areas, the interchange spacing rule-of-thumb is spaced no closer than three miles apart. There is no specific guidance for areas between urban and rural contexts. These spacing guidelines are intended to minimize the disruption of entering and exiting traffic to the freeway and to prevent insufficient sign spacing. The risk is greatest with regard to urban spacing values of less than one-mile.

2.2 Does the spacing between successive entrances and exits meet or exceed AASHTO criteria? __________ If no, consider the following improvement alternatives: • Relocate one or more ramps to achieve minimum spacing (Note: check spacing to adjacent interchanges if ramps are significantly relocated). • Provide collector-distributor roads to consolidate closely spaced ramps. • Consolidate separate entrance ramps; provide one rather than two merge points to the mainline. • Add auxiliary lanes at one or more entrance ramps to facilitate merging.

For guidance on minimum spacing between individual ramps, most agencies utilize Exhibit 10-68 from the AASHTO *"Policy on Geometric Design of Highways and Streets"* (Green Book). An NCHRP research project will be completed in 2010 to provide supplemental guidance to Exhibit 10-68 and explain important considerations for determining appropriate ramp spacing. This supplemental guidance is very important since some design practitioners simply default to the stated minimum values in Exhibit 10-68 and fail to examine the key considerations of their specific project conditions. Such considerations include: ramp volumes, truck volumes, acceleration and deceleration length needs created by grade and ramp configuration. The spacing values in Exhibit 10-68 are also not indicative of the needs to accommodate two-lane entrance and exit ramps and the lengths needed to properly form auxiliary lanes for such ramps.

3.0 Approach Alignment to Interchange

3.1 Is the grade of the freeway relatively flat through the interchange area? __________ 3.2 Is the horizontal alignment of the freeway relatively straight through the interchange area? __________

It is desirable to locate a proposed new interchange on a relatively flat gradient. Freeway gradients on approaches to interchanges should be limited to 3% in areas with a 70 mph design speed and up to 5% for a 50 mph design speed.

It is desirable to locate a proposed new interchange on a relatively straight alignment. The horizontal curve radius of the freeway approaching a proposed interchange should be limited to 2600 ft for a 70 mph design speed (1900 ft for 60 mph design speed).

3.3 Is adequate sight distance (desirably decision sight distance) provided in advance of each exit? __________

It is highly desirable to provide *decision sight distance* along the freeway mainline in advance of an exit. Decision sight distance is discussed in Chapter 3 of the AASHTO Green Book.

If full decision sight distance values cannot be provided, assess the risk of the deficiency. A deficiency of less than 10 mi/h is generally a low to moderate risk. Decision sight distance deficiencies greater than 10 mi/h represent higher risk and in such cases the following improvement alternatives should be considered:
• Revise the mainline geometry to provide adequate sight distance
• Relocate the exit ramp to lengthen available sight distance
• Incorporate enhanced advance signing strategies

4.0 Interchange Configurations

4.1 Was an appropriate array of interchange configurations and variations evaluated in the design study phase?

There are a variety of interchange configurations and variations available for the design of new and reconstructed facilities depending on the conditions encountered. The selection of an interchange configuration is influenced by factors such as topography, the number of intersecting legs, right-of-way availability, operational needs on the mainline and cross street, potential site impacts, and cost. Each interchange must be designed to fit individual site needs, conditions and constraints.

Interchanges are broadly classified into two functional categories – "service interchanges" and "system interchanges". The term "service interchange" applies to interchanges that connect a freeway to lesser facilities (non-freeways) such as arterials or collector roads. Most service interchange forms have at-grade intersections of the ramp terminals and the non-freeway cross-road. These intersections generally have some type of traffic control (stop signs, traffic signals, or yield conditions at roundabout intersections) that may require drivers to either stop or yield to other traffic or pedestrians. An interchange that connects two or more freeways is generally termed a "system interchange". Generally, the traffic movements within system interchanges are intended to be free-flowing without stopping (except in special cases where toll plazas or ramp metering may be present).

4.1.1 Diamond interchanges
Diamonds are the most common type of service interchange configuration and are generally applicable for a wide range of conditions. Diamond configurations have one-way diagonal ramps in each quadrant. As a result of the common usage of the diamond interchange, they have a high degree of driver familiarity. Traffic maneuvers at a diamond interchange are relatively uncomplicated. From a human factors perspective, an important desirable characteristic of the diamond interchange is that the turn movements from the crossroad and from the freeway exit ramps are "true" to the intended change in direction of travel. In other words, a driver makes a left turn at the interchange when desiring to make a left turn in travel direction. This desirable characteristic is consistent with driver expectancy. In contrast, interchanges that utilize loop ramp configurations may confuse unfamiliar drivers since loop ramps require making a right turn at the interchange for a movement that would normally be considered as a left turn in their intended direction of travel. Diamond interchanges can be further categorized based upon the ramp separation distance, ramp terminal control strategy, and the crossroad cross-section.

4.1.2 Conventional diamond
Applicable mostly to rural conditions where space allows, a conventional diamond is typically characterized by an intersection spacing of 800 to 1200 ft (centerline to centerline) between where the two sets of ramp terminals intersect on the crossroad. Several options may be utilized for the traffic control at the two ramp terminal intersections with the

crossroad. Lower volume ramps may simply be stop controlled. Adequate sight distance based on unsignalized intersection criteria must be provided and can play a key factor in the bridge design at the interchange. If higher volumes exist, actuated traffic signals or roundabouts may be appropriate. To accommodate potential future traffic growth, consideration should be given for coordination of the signals and for needed lengths of left-turn bays on the crossroad. The bridge width is typically the significant factor influencing the cost of a conventional diamond interchange. If the two intersections of the ramp terminals are spaced far enough apart, then typically the bridge width need only accommodate the crossroad through lanes (plus any median) since the left-turn lanes on the crossroad can be formed beyond the bridge structure.

4.1.3 Compressed diamond
A "compressed" diamond interchange is typically characterized by having the ramp terminal intersections spaced 400 ft to 800 ft apart along the crossroad. This form of the diamond interchange is sometimes used where right-of-way is restricted. In some instances, only one side of the diamond is "compressed" (i.e. the nearest ramp terminal is 200-400 feet from the freeway centerline). Under higher volume conditions, obtaining traffic signal progression becomes challenging in the compressed diamond configuration. Also, because the spacing between the two intersections is less than at a conventional diamond, it may be necessary for the bridge width to also include the left turn lanes on the crossroad. The compressed diamond is best suited to rural or suburban areas where traffic demands are low to moderate. Under higher volumes, the inability to achieve efficient crossroad signal coordination makes the compressed diamond much less operationally efficient than the tight diamond or the single-point diamond.

4.1.4 Tight diamond
The tight diamond has the same form as the conventional diamond, with the spacing between the two at-grade intersections usually between 250 and 400 feet. Because of the close spacing between the intersections, both must be signalized, and the signals must be coordinated to allow through traffic to pass through both intersections with at most one stop. For maximum operational efficiency, special treatment of channelization and traffic control is required: left turns from the minor road must store in advance of the first intersection (not between the two ramps). This feature may increase the risk of wrong-way movements (improper left-turns from the crossroad onto the freeway exit ramp) and therefore enhanced wrong-way warning signing and marking strategies may be appropriate. This form also allows easy accommodation of pedestrian crossings of the minor road.

4.1.5 Single-point diamond
The single-point diamond interchange (SPDI) consolidates all the left-turn movements to and from entrance and exit ramps into a single intersection in the center of the interchange. All four left-turning moves are controlled by a single multi-phase traffic signal system and opposing left turns keep to the left of each other. The advantages of a SPDI include:

- The operation of only one signalized intersection on the crossroad, as opposed to two in conventional diamond interchanges, typically offers improved operations and reduced delay through the intersection area.
- Right-turn movements may be signalized to allow for a signalized pedestrian crossing or they may be free-flow movements.
- Curve radii for left-turn movements through the intersection are significantly flatter than at conventional intersections, and, therefore, the left turns discharge more efficiently and better accommodate trucks.

The primary disadvantages of the SPDI are its higher costs because of the need for a larger structure and the need for a careful design of channelization for the left turns to minimize driver confusion (overlapping turn paths and wrong-way maneuvers). Also, SPDIs with a skewed angle between the two roadways increases the signal clearance intervals and adversely affects delay.

Single-point diamond interchanges may be designed such that the crossroad either passes over or under the freeway. Constructing the crossroad intersection over the freeway allows the structure columns to be located in the freeway median thus reducing the clear span of the structure and substantially reducing costs associated with girder depth. Also, when the at grade-intersection is located on the top level it is exposed to an even lighted surface, thus not requiring the driver to go from sunlight into shade and back into sunlight.

4.1.5 Split diamond

Split diamonds serve multiple crossroads connected by frontage roads that are usually one-way. In addition to the ability to serve multiple crossroads, split diamonds offer the advantages of reducing conflicts by handling traffic at four, rather than two, intersections and at each intersection the number of left -turn movements is reduced from two to one. This form typically is more costly due to the need for two or more bridges. The split diamond form is commonly used near central business districts. This form allows easy accommodation of pedestrian crossings of the minor road.

4.1.6 Cloverleaf Interchanges

Cloverleaf interchanges use loops to accommodate some movements. Interchanges with loops in all four quadrants are referred to as "full cloverleafs" and others with loops in one or more quadrants are referred to as "partial cloverleafs" or "parclos."

4.1.7 Full Cloverleaf

With full cloverleafs, because all the left turn movements are made via loops, there is no need for intersections on the crossroad. This typically decreases the delay encountered by these movements and increases the efficiency of operations on the crossroad. A major disadvantage of the full cloverleaf is the weaving that must occur between the loop ramps. Weaving is very frequently a problem in all but very low volume conditions. The AASHTO Green Book recommends that when the sum of traffic on two consecutive loops approaches 1000 vph, that either another interchange form be used or that a collector-distributor (C-D) system separated from the mainline traffic be added to accommodate the weaving traffic.

Full cloverleaf interchanges require more right-of-way than most other forms depending on the design of the loop radii. The speed of travel on a loop may be increased by using larger loop radii. On the other hand, tighter radii may be more susceptible to run of the road crashes.

4.1.8 Partial Cloverleafs (ParClos)

Parclos use one, two, or three loops to handle certain movements. Typically, the heavier left turn movements are the ones accommodated via loops. Parclos are highly adaptable and can accommodate high traffic volumes. Parclo configurations are generally most applicable in situations where a specific left-turn movement pair has a comparatively high volume that would be operationally problematic on the ramp terminals of a diamond interchange. They are also advantageous when one or more quadrants must be avoided due to right of way restrictions.

There are a variety of forms of parclos and common terminology describes them based on the location of the loops and if ramps are in four, three, or two quadrants.

In Parclo A interchanges, entrances to the freeway are made via loop ramps. This provides for improved operations on the crossroad by eliminating the left turns onto the freeway entrance ramps. It also eliminates the need for providing those left turn lanes on the crossroad and therefore typically allows for reduced structure costs. Exits off the freeway are made via direct connection ramps to the crossroad and the intersection at the crossroad requires either signalization or stop control.

A parclo A may also have ramps in only two quadrants and eliminate the two direct freeway entrance ramps from the crossroad. Whereas in a four-quad parclo A all traffic entering the freeway is made via a right turn off the crossroad, in a two-quad parclo A two entry movements are made via a left turn from the crossroad onto the loop ramps. With either form of Parclo A, there are two intersections and minor road through traffic may have to stop twice. Each stop is usually controlled by a 2-phase signal.

In Parclo B interchanges, the loop ramps accommodate traffic exiting the freeway. In a four-quad parclo B, the loops eliminate the need for the traffic exiting the freeway from having to make a left turn at the crossroad. Although the parclo B configuration requires two intersections, the through traffic on the crossroad would only have to stop once at most. If the intersections are signalized, the signals can be designed such that the crossroad through traffic receives a continuous green indication. Another major advantage of the four-quad Parclo B is that because the movements exiting the freeway are unsignalized, there is a lower risk of traffic queues on the exit ramp. The ramp terminal design of the four-quad parclo B interchange also makes wrong-way ramp entry movements highly unlikely.

In Parclo AB interchanges, all ramps are located on one side of the crossroad. This form is mainly used where the right-of-way is restricted on one side of the mainline because of a stream or railroad.

In full cloverleafs, Parclo AB, and 3-loop Parclos, because loops are present in adjacent quadrants, weaving is a problem that may lead to a breakdown in traffic operation and more crashes.

4.1.9 Directional Interchanges

Directional interchanges allow for all high speed direct movements from one facility to another and are particularly applicable for system interchanges. Directional interchanges may also incorporate loop ramps to accommodate traffic of lower-volume directional movements. The volume on a tight loop ramp (30-40 mph design speed) is limited to approximately 1,200 DHV. Several agencies have constructed loop ramps with two-lanes.

The entrance to loop ramps should be designed with consistent radii, without compound curves entering the loop from a high-speed condition. Compound curve design is acceptable when leaving the loop and entering the acceleration lane.

4.1.10 T and Y Interchanges

Interchanges having three legs are commonly referred to as “T” and “Y” interchanges and are used where a freeway or major highway begins or terminates. Three-leg interchanges should only be considered when future expansion to the unused quadrant is unlikely since they are difficult to expand, modify, or otherwise retrofit as a four leg facility. The trumpet type (with a single structure) has three of the turning movements accommodated with direct or semi-direct ramps and one movement by a loop ramp. In general, the semi-direct ramp should favor the heavier left-turn movement and the loop the lighter volume. Where both left turning movements are fairly heavy, the design of a directional T-type interchange is best-suited.

4.2 Is the selected interchange configuration appropriate for the operational needs, fits the topography and potential site conditions and constraints, and is consistent in exit pattern with other nearby interchanges?

While interchanges should be custom designed to fit specific site conditions and traffic operational needs, it is desirable that the overall pattern of exits along the freeway have some degree of uniformity. An inconsistent arrangement of exits between successive interchanges may cause driver confusion and result in drivers slowing down on high-speed lanes and making unexpected maneuvers. From the standpoint of driver expectancy, it is desirable that all interchanges have one point of exit located in advance of the crossroad wherever practical. Exhibit 10-45 of the AASHTO Greenbook presents examples of inconsistent and uniform exit patterns.

4.3 Are all directional traffic movements provided for?

The AASHTO Interstate Standards Policy states: *“Each interchange shall provide for all traffic movements.”*

Unless demonstrated to be impractical, all interchanges should provide for all movements even if the anticipated demand volume for that movement is low. The omission of the ability to make full movements between the freeway and crossroad or between two freeways can create confusion for unfamiliar drivers looking for the connection. When drivers exit the freeway, there is an expectation that they can re-enter in the same direction of travel at the same interchange or within a short distance on a frontage road. In addition to creating driver confusion and frustration, omitting movements at service interchanges may contribute to increased wrong-way movements as confused drivers attempt to re-enter the freeway via the ramp they exited from. Even proposals to omit connections for very low volume movements should be highly scrutinized. As a minimum, the right-of-way should be obtained to construct any missing connections in the future. Future land use changes and development may significantly increase the demand for the maneuver. Considerations of the risk of not providing for all movements should include the amount of travel misdirection required for a driver to make the movement via an adjacent interchange and the ease to reach the adjacent interchange.

4.4 Are all the exits and entrances on the right side of the freeway mainline?

It is highly preferable to use right-hand entrance and exit ramps in the design of new interchanges. Entrance and exit ramps on the left-side of the freeway are contrary to driver expectation and studies indicate that crashes may be reduced as much as 25-70 percent with the use of right-off, right-on ramps as compared to left hand ramps. Traffic speeds are typically faster in the left-most lanes of the freeway, and therefore speed differentials between entering and exiting traffic and through traffic is usually greater with left-hand ramps.

If possible, existing left hand entrance/exit ramps should be replaced with right hand ramps when reconstructing an interchange. If this is impracticable because of unacceptable economic, environmental or social impacts then such reasons should be well documented and justified. Such justification should include a crash data analysis showing that the existing left hand ramp is not a substantial safety hazard.

If it is not feasible to eliminate left-side ramps, consider the following mitigation measures:

- Extend auxiliary lanes in advance of exits and beyond entrances to reduce the speed differential conflicts
- Provide full decision sight distance in advance of a left-side exit
- Providing supplemental advance signing for left-side exit ramps
- Provide ramp geometry near the point of physical merge or diverge that accommodates a high design speed (provide at least 75 percent of mainline design speed)

4.5 Is there a weaving section within the interchange as proposed?

Weaving sections on freeways involve the crossing of traffic streams created by merging and diverging maneuvers. This may occur within an interchange or between two closely spaced interchanges. Full cloverleaf interchanges have weave sections occurring between the loop ramps (a freeway entrance from a loop is immediately followed by an exit onto a loop). The entrance and exit are joined by a continuous auxiliary lane.

Considerable traffic turbulence occurs throughout weaving sections. Interchange designs should avoid creating weaving sections or at least have the weaving section placed on collector-distributor lanes. Designs that incorporate collector-distributor lanes and/or grade-separate closely spaced ramps by “braiding” are typically more costly. Evaluation of the total interchange cost and the expected traffic operational benefits of improved design alternatives is needed to reach a sound decision between design alternatives.

Traffic operations within a freeway weaving segment are greatly dependent upon the volumes of weaving traffic and the length of the weaving segment. Heavy weaving volumes (particularly with high truck volumes) require longer lengths to allow vehicles to change lanes safely and at reasonable speeds. Key risk factors such as the volume of weaving and non-weaving traffic, the free-flow speed of the freeway, the weave configuration, and the length of weaving segment should be considered in evaluating design alternatives.

As the length of a weaving segment increases, the effects of the weaving maneuvers diminish and the merging and diverging maneuvers themselves mostly contribute to disruptions within the traffic stream. Under most typical conditions, weaving lengths of 2500 ft or more are of low risk. Weaving lengths between 1600 ft and 2500 ft should be evaluated closely and may or may not operate acceptably depending on specific volumes and site conditions. Weaving lengths of less than 1600 ft may be appropriate if volumes are low, however, they should be considered a high risk for operational failure during times of higher volume conditions.

For weaving segments that may be problematic, consider the following design alternatives:

- Relocating one or both ramps to eliminate the weave.
- Constructing a collector-distributor road on which the weaving could occur at lower speed.
- Redesigning the interchange to lengthen the weave.
- Continuing an auxiliary lane beyond the weaving section to aid entering drivers.

4.6 Is the interchange configured with the crossroad over the freeway?

At service interchanges it is desirable to design the interchanges with the crossroad above the freeway due to:
- The crossroad above the freeway results in longer sight distances to the exit ramp and gore area.
- The crossroad above the freeway allows gravity to assist the operation of both accelerating vehicles (the on-ramp has a down-grade) and decelerating vehicles (the off-ramp has an up-grade). In addition, the resulting grades generally provide longer sight distances.

4.7 Route Continuity - Is the interchange configured so that the priority route is the through facility?

The concept of route continuity is applicable to system interchanges and is based on a driver's expectation that through travel on a primary route should be provided without a need to make excessive lane changes or an exit type of maneuver. The principle of route continuity is an extension of the principle of operational uniformity coupled with the application of proper lane balance and the principle of maintaining a basic number of lanes as described in Chapter 10 of the AASHTO Greenbook. Designs that adhere to the principle of route continuity will greatly simplify the driving task by reducing forced lane changes and simplify directional signing.

Desirably, the through driver should be provided a continuous through route on which changing lanes is not necessary to continue on the through route. In maintaining route continuity, interchange configuration may not always favor the heavy traffic movement, but rather the through route. In this situation, heavy movements can be designed on flat curves with reasonably direct connections and auxiliary lanes.

On existing interchanges where this principle is violated and it is not practical to reconfigure the interchange to provide route continuity, consider the following mitigation strategies:

• Provide enhanced advance guide signing and gore signing
• Provide auxiliary lane(s) to minimize lane changing.

5.0 Ramp Design

5.1 Is the design speed of the ramp proper at least 50 percent of the mainline design speed?

The design speed of the ramp proper must be at least 50% of the design speed of the mainline freeway. It is desirable that the design speed of the ramp proper be 70%-85% of the design speed of the freeway mainline (see Exhibit 10-56 of AASHTO Greenbook). This is particularly important for interchanges in rural settings where operating speeds tend to be higher and congestion levels lower than in urbanized areas. Rural Interstates also carry a higher percentage of truck volumes and a higher percentage of unfamiliar drivers. Drivers tend to become accustomed to high travel speeds and the transitions between design speed changes should be at the desirable range rather than the minimum standards.

Directional ramps and diamond interchanges should be designed in the upper range (within 85% of the mainline design speed). Loop ramps in cloverleaf or partial cloverleaf interchanges are typically in the lower range (within 50% of the mainline design speed). The minimum design speed on ramps or turning roadways associated with interchanges is normally 30 mph. A minimum design speed of 25 mph may be used on loop ramps when the mainline design speed is 50 mph or less. Because of the increased lengths and large areas required, in many cases the upper practical design speed on loop ramps is 30 mph. Connections between freeways in a system interchange are generally free-flow and should also be made via high design speed (85% of mainline) connections.

If the design speed of the ramp proper is not consistent with the desired middle and upper ranges, consider the following design alternatives or improvements:

• Increase the ramp radius of curve
• Increase the ramp superelevation
• Widen the ramp cross section

• Improve the roadside on the approach to and on the low-speed curve
• Provide transition curvature between the high-speed mainline and low-speed ramp

5.2 Is sufficient length for acceleration at entrance ramps provided?

An entrance ramp requires sufficient length to transition between the elevation differences of the freeway and crossroad over a reasonable grade. It is desirable that grades on ramps not exceed five percent. A maximum grade of eight percent should only be used if the length of such grade is relatively short. In addition, the ramp also serves to facilitate transitions in vehicle speeds (acceleration). Some acceleration may occur on the ramp proper depending on the grade and curvature of the ramp. When the ramp lane joins with the freeway mainline, additional length may be needed to achieve further acceleration. Also, a "gap acceptance" length should be provided to allow entering vehicles to adjust speed and safely maneuver into the freeway mainline. Freeways with higher volumes and/or high truck volumes typically warrant longer gap acceptance lengths at entrances to provide safe and efficient merging maneuvers.

The two general forms of entrance ramps are the parallel type and the taper type (see AASHTO Greenbook Chapter 10). The operational and safety benefits of long acceleration lanes provided by parallel type entrances are well recognized. The parallel type entrance ramp is recommended for new interchange construction or for the reconstruction or reconfiguring of existing interchanges. An acceleration lane length of at least 1200 ft is desirable (longer if on up grades exceeding 2%). Merge tapers at the downstream end of parallel-type entrance ramps should have a minimum taper length of 300 ft. The parallel type entrance ramp is particularly advantageous when the geometrics of the ramp proper limit the ability of vehicles to accelerate to near freeway operating speeds. Desirably, a curve with a radius of 1000 ft or more and a length of approximately 200 ft should be provided in advance of the parallel ramp. If the approach curve has a short radius, drivers tend to drive directly onto the mainline without using the acceleration lane.

Some agencies use, or have previously used, taper-type entrance ramps where the entrance is merged into the freeway with a long uniform taper (70:1 or greater desired). When using a taper style entrance, it is important that the geometrics of the ramp proper be such that vehicles may attain a speed within 5 mph of the operating speed of the freeway by the time they reach the point where the left edge of the ramp joins the traveled way of the freeway. If properly designed, the taper-type entrance ramp is an acceptable alternative. However, parallel entrance ramps are generally preferred and studies have shown that parallel entrance ramps are typically safer than tapered. In particular, the parallel design offers advantages to older drivers. With the tapered entrance, the driver has poorer angles in which to use side/rear-view mirrors to monitor surrounding traffic prior to merging. Taper-type entrance ramps can also cause confusion in mainline horizontal curve situations when the driver may have difficulty identifying the mainline alignment.

Entrance ramps and merging areas should be visible to approaching main line traffic for a minimum distance equivalent to the design stopping sight distance and desirably to decision sight distance values.

5.3 Is sufficient length for deceleration at exit ramps provided? ____________

The appropriate length of the deceleration lane varies depending on the design speed of the mainline and the design speed of the first geometric control on the exit ramp (usually a horizontal curve but could be the stopping sight distance on a vertical curve or the back of an anticipated traffic queue). Exhibit 10-73 of the AASHTO Greenbook provides the minimum lengths of deceleration lanes for exit ramp terminals. When the average grade of the deceleration lane exceeds 2%, the deceleration length should be adjusted by the factor obtained from Exhibit 10-71.

The two general forms of exit ramps are the parallel type and the taper type (see AASHTO Greenbook Chapter 10). A well-designed taper-type exit fits the direct path preferred by most drivers, permitting them to follow a natural exit path within the diverging area. The divergence angle should normally be between 2 and 5 degrees. At ramp terminals on curves, the parallel type of exit ramp is preferred because it provides increased "target" value of the diverge point and reduces the steering demands on the exiting driver. Exit ramps should diverge in such a way that the vertical curvature will not restrict visibility along the ramp to a value less than the stopping sight distance for the ramp design speed. Ramps that "drop out of sight" create a definite problem in driver recognition of queuing at the crossroad intersection and should be avoided.

Consider the queue storage requirements along the exit ramp (influenced by the traffic control device operations such as signals, roundabouts or ramp meters at the ramp termini) when determining appropriate deceleration length needs on the ramp proper. It is desirable to provide decision sight distance to the back of any stopped queue along a ramp. It is also suggested to use ninety percentile queue lengths when considering ramp length needs.

5.4 Are the exit and entrance lanes balanced?

The principle of lane balance involves providing an operationally balanced arrangement of lanes in conjunction with exiting and entering traffic. At exits, lane balance simply means the provision of one more lane going away (the combined number of lanes on the freeway and ramp after the exit should be one more than on the freeway preceding the exit). Compliance with this principle essentially avoids having a “trap” lane or lane drop situation with an exit-only lane. Redesigning exit-only ramp diverges to continue the right lane at least 600 ft past the physical diverge has been a successful strategy used in many states.

At entrance terminals, the sum of lanes before the merge (on freeway and ramp) is equal to the total number on the freeway after the merge (or one more than the total if a lane is being added).

It may be necessary to obtain lane balance by adding an auxiliary lane upstream from the diverging nose. The length of each additional lane should be 2,500 ft. and should be introduced using a 0 to 12 ft. taper with a length of at least 300 ft.

There may be conditions off the mainline, such as on collector-distributor roads, where lane balance and lane continuity are less important.

6.0 Signing

6.1 Is the proposed signing in accordance with the MUTCD and suggested limits on message units? __________

The Manual of Uniform Traffic Control Devices (MUTCD) provides guidance on Interstate signing standards and criteria. The concept signing plan should provide for simple signs that can be accommodated within the interchange layout. Avoid using diagrammatic signs when simpler signing will suffice. Other opportunities to address human factors within interchanges areas include the following strategies.

- Use pavement markings and signs to assist the driver with simplifying decisions, but the number of signs or special markings should be used judiciously.
- Consider the effect of intelligent transportation systems on driver workload and decision making. Avoid providing too much information to drivers in too short of a drive time.

Sign designs should strive to provide the necessary information with consideration of practical driver comprehension limits of message units.

6.2 Evaluate the proposed signing from a driver’s point of view. Assess the risk of driver confusion and strategies to simplify the signing.

The complexity of the freeway guide signing should be a major consideration in concept development and the early design stages of an interchange project. The need to provide clear and simple signing that an unfamiliar driver can understand while traveling at freeway speeds is a critical design consideration. Signing needs may directly influence design choices such as interchange spacing, ramp locations, and interchange layouts.

7.0 Crossroad Design

7.1 Are sidewalks and bicyclist facilities provided along the interchange crossroad? Pedestrians and bicyclists are particularly vulnerable to high speed approach vehicles turning at ramp terminals. Are the crossings at interchange ramps controlled or uncontrolled?

Pedestrian and bicycle accommodations (such as sidewalks, bicycle lanes, and shoulders) should be maintained on the crossroad through the interchange area. Pedestrians and bicyclists are particularly vulnerable to high speed approach vehicles turning at ramp terminals. In areas with pedestrian usage, avoid channelization designs at the crossroad/ramp intersection that provide free-flow movements and consider providing accessible pedestrian signals across all crossings.

The PEDSAFE Guide (www.walkinginfo.org/pedsafe) is a comprehensive guide to the wide range of treatments available to enhance pedestrian safety and mobility. PEDSAFE includes diagnostic software which allows a user to find appropriate treatments taking into account the location, goal of the treatment, types of pedestrian crash, and site characteristics.

The design of accessible pedestrian facilities is required and is governed by implementing regulations under the Rehabilitation Act of 1973 and the Americans with Disabilities Act (ADA) of 1990. These two Acts reference specific design and construction standards for usability. Other reference material for pedestrian and bicyclist facilities are AASHTO's "Guide for the Planning, Design, and Operation of Pedestrian Facilities, 1st Edition," and FHWA's "Designing Sidewalks and Trails for Access Part D of II: Best Practices Design Guide."

Providing a dedicated grade-separated freeway crossing for pedestrians and bicyclists away from the interchange area may be preferable and appropriate in some instances. Another potential strategy is to include connecting pedestrian and bicyclist facilities to a route parallel to the one crossing at the interchange, such as a smaller street without an interchange.

7.2 Is sufficient control of access along the crossroad beyond the interchange being provided to ensure its integrity? (The AASHTO standard of a minimum of 100 ft in urban areas and 300 ft in rural areas is usually insufficient where additional development is likely).

7.3 Are adequate land development and access management measures in place for the interchange area?

Poor and inadequate access management along the interchange crossroad is the most likely cause of operational failure at an interchange. Proper control of access must be maintained within and near an interchange in order to ensure its integrity. This is accomplished by acquiring sufficient right of way, and restricting the proximity of public and private access to the ramp/minor road at-grade intersection. The AASHTO standard of a minimum of 100 ft in urban areas and 300 ft in rural areas is usually insufficient where additional development is likely. The values suggested in the TRB Access Management Manual Tables 9-14 and 9-15 should be obtained when new interchanges are proposed and the ability to obtain right of way and access control rights are more practical.

For improvements to existing interchanges, consideration should be given to extending control of access limits if possible. Also, implementing strategies such as using raised medians to restrict turning movements in the interchange area should be considered. Projects that will expand the capacity of the interchange should closely evaluate the effect of the spacing/separation of traffic signals within the crossroad interchange area and the interrelated effects on queue storage and progression through the intersections.

Entrance and exit ramps should not be allowed to have side road or private driveway connections on the ramps. Such access within interchange ramps is counter to driver expectancy and may also contribute to wrong-way ramp movements.

7.4 Ensure elements of the ramp/crossroad intersection are properly designed, especially with regard to:

7.4.1 Turning radii for design vehicle

The intersection turning radii should be appropriate for the number and type of trucks. An AASHTO WB-62 or WB-67 vehicle is recommended as the minimum design vehicle for all turning movements for interchanges on the Interstate System.

7.4.2 Capacity

Consideration of capacity requires assessing the necessary traffic control devices and certain physical geometric design elements such as number of turning lanes, angle of intersection, grade, and channelization. In urban areas where traffic volumes may be high, inadequate capacity of the ramp/cross road intersection can adversely affect the operation of the ramp/freeway junction. In a worst case situation the safety and operation of the mainline may be impaired by a back-up onto the freeway. Therefore, special attention should be given to providing sufficient capacity and storage for the at-grade intersection with the cross road. This could lead to the addition of lanes at the intersection or on the ramp proper, or it could involve traffic signalization timing modifications where the ramp traffic will be given priority. The analysis must also consider the operational impacts of the traffic characteristics on the intersecting road and signal timing for pedestrians.

7.4.3 Traffic control

The ramp/cross road intersection is typically controlled by stop signs, roundabouts, or signals. Use of roundabouts at ramp terminals has been used very successfully in several states and offers many advantages in regard to safety, capacity, user delay, and accommodating nearby frontage roads.

7.4.4 Channelization

Most wrong-way movements originate at the ramp/cross road intersection. This intersection must be properly signed and designed to minimize the potential for a wrong-way movement.

7.4.5 Intersection Sight Distance

Intersection sight distance needs are dependent upon the type of traffic control used. Addressing sight distance issues at at-grade intersections is a critical design concern. Special attention must be given to the location of the bridge pier or abutment because these may present major sight distance obstacles. The combination of a bridge obstruction and the needed sight distance may result in relocating the ramp/cross road intersection to provide the needed sight distance.